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P. N. KHANNA

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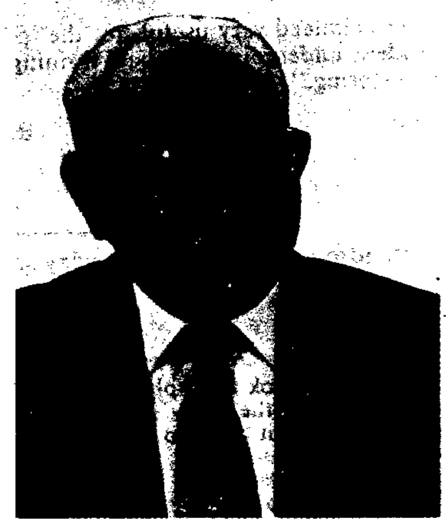
An Encyclopaedia of Technical Information



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PREFACE

In the planning and execution of Civil Engineering works India is making rapid strides to keep pace with the present requirements. As such, to keep abreast with the latest advances made in this field the book has been subjected to further revision.

I have again drawn freely from all available sources from a wide field and have made all efforts to incorporate all matters of fundamental importance and the relevant latest IS Codes of Practice, and also the specifications issued by the various PWDs and the IRC, and other valuable literature produced by the Institution of Engineers (India), as regards their practical application, have been taken into account. Formulae and equations which have become, more or less, mainly of historic interest have been omitted.

I have to repeat again that the material discussed in the various Sections is by no means conclusive or the last word on the subject.

P.N. Khanna

As AN ABSOLUTE ACCURACY OR FREEDOM FROM ERRORS OF A WORK containing such a mass of figures and data cannot be hoped although greatest care has been exercised in its preparation, figures for all important design works should be checked with the theory.

THE RECEPTION OF THE LAST EDITION OF THIS BOOK WAS FAR WIDER than I had anticipated and this edition also was exhausted witnin a very short time of its publication as were the previous ones and the book had to be out of market for a considerable time disappointing innumerable friends who were eager to have a copy of their own. The book was bought right from Chief Engineers down to subordinates and not only from all corners of India but also from abroad.

The task of compiling and revising such a voluminous book dealing with so many subjects has been a very arduous one and has now taken almost ten years of hard and ceaseless labour which involved the review of virtually all current literature, within the scope of the book, in the field whatever I could gather, and study of construction methods using the latest techniques, wherever I could manage to go. I have tried to consult the best available sources of information; great help has been derived from the latest publications of outstanding authors, works of the various research institutions, and from the technical instructions and works specifications issued from time to time by the various Public Works Deptts. in India and also abroad where I had had the opportunities of working and learning. The book attempts, therefore, to present the collective experience of a large body of experts and I have to acknowledge my indebtedness to these great masters from whom I have borrowed so profusely, for all what has been reproduced.

Everyday practice may be variable or even controversial, still this book gives the basic principles, a knowledge of which is essential for a Civil Engineer. In view of the demand for the book from engineers of all grades, the subject matters have been presented in such a manner and sequence so as to cater for a wide range of readers. The aim has been to make the information sufficient for all normal work likely to be encountered, and to be as definite as possible in every case.

The present edition has been fully revised, a lot more of useful data, whatever I could gather from the various sources, has been added and the scope of the book considerably widened. Suggestions received from several readers for enhancement of its usefulness have also been incorporated as far as feasible.

P. N. KHANNA

From the PREFACE TO THE FIRST EDITION

THIS BOOK HAS BEEN COMPILED PRIMARILY FOR THE "PRACTICAL man" and should prove a most useful work of referenee to the young engineers of the various Public Works Departments. The object of this volume is to give a fairly complete but concise account of the various subjects to serve as a ready reference for everyday-work problems which constantly confront the engineers, whether in the office or in the field, without having to wade through numerous books and notes.

All possible efforts have been made to make the book comprehensive and complete by itself, packed with as many details as possible, elucidating in simple and plain language the engineering principles in sufficiently practicable and most easily applicable form free from advanced mathematics.

My grateful thanks are due to my numerous colleagues and friends for the valuable help given me in making available the various details required for my work. Particular appreciations are expressed to: Mr. Percy M. Otway, M.AM. Soc.C.E., M.I.Mech.E., M.I.Struct.E., F.G.S., etc., of the Ministry of Transport (Roads Organization), London, for the most valuable help rendered in my collecting the data for the modern methods of road building and taking me round personally on works to show the various field processes as used in that country; Col. W.P. Andrews, M.C., of the Cement and Concrete Association, London, for similar help given as regards concrete works and making available their latest literature on the subject; the Secretary, Indian Roads Congress for his kind permission to reproduce the various Standards and other useful information produced by them.

Very little originality, no finality or perfection is claimed. I shall gratefully appreciate the readers who will kindly call attention to any errors of ommission or commission or give valuable suggestions for improvement of the book to enhance its usefulness.

P. N. KHANNA

P.W.D., PATIALA.

SI Units and their Practical Application

In SI units dimensions are expressed in mm, area in mm^2; stress in N/mm^2; bending moment in N. mm units. The relationships in general with metric units are as follows :

1 kg/cm^2 is taken as 0.1 N/mm^2	10 kg/cm^2 is taken as 1 N/mm^2
1 kg-cm is taken as 100 N-mm	0.01 kg-cm is taken as 1 N-mm
1 kg (f) is taken as 10 N	0.1 kg (f) is taken as 1 N
1 t/m is taken as 10 kN/m	100 kg/m is taken as 1 kN/m

$1 \text{ MPa} = 1 \text{ N/mm}^2 = 0{\cdot}102\,0 \text{ kg/mm}^2$.

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WEIGHTS & MEASURES

General Abbreviations

ap	Apothecaries' wt.	galls.	gallons	pt.	pint
av.	average	grs.	grains	qr.	quarter
avoir, avp	Avoirdupois wt.	hr.	hour	sec.	second
		imp.	Imperial	sq.	square
Br.	British	ins.	inches	sr.	seer
cu., c.	cube, cubic	lbs.	pounds	st.	stone
cwt.	hundredweight	liq.	liquid	tr	Troy wt.
dr.	dram, drachm	md.	maund	US	United States
fl.	fluid	min.	minute	wt.	weight
ft.	foot, feet	oz.	ounce	yds.	yards

Metric System Symbols

a.	are	dm.	decimetre	km.	kilometre
cg.	centigram	g., gm.	gram	l.	litre
cl.	centilitre	ha.	hectare	m.	metre
cm., c.	centimetre	hg.	hectogram	mg.	milligram
dg.	decigram	hl.	hectolitre	ml.	millilitre
dkg.	dekagram	hm.	hectometre	mm.	millimetre
dkm.	dekametre	kg., kgm., kilo.	kilogram	q.	quintal
dkl.	dekalitre				
dl.	decilitre	kl.	kilolitre	t., T.	metric ton

The Metric System

The metric system is a system of weights and measures which was first adopted by France and is now almost universally used in the sciences and arts. The greater part of the population of the world works on the metric system. The metric is so called because it uses the 'metre' as the primary unit.

In the metric system there are three principal or primary units: the *Metre* —the unit of length; the *Litre* (pronounced lee-ter)—the unit of capacity (usually liquid), and the *Gram*—the unit of weight or mass. Multiples of these units are obtained by pre-fixing to the names of the principal units the Greek words *deka* or *deca* (10—ten times), *hecto* (100—hundred times), *kilo* (1000—thousand times), *myria* (10,000—ten thousand times), and *mega* (1,000,000 times). The sub-multiples or divisions are obtained by pre-fixing the Latin words *deci* (1/10—one-tenth), *centi* (1/100—one hundredth), and *milli* (1/1000—one-thousandth). These pre-fixes form the key to the entire system.

1 metre / gram / litre = 10 Deci metres / grams / litres = 100 Centi metres / grams / litres = 1000 Milli metres / grams / litres

1 Kilo metre / gram / litre = 10 Hecto metres / grams / litres = 100 Deka metres / grams / litres = 1000 metres / grams / litres

Metric numbers are written with the decimal point (.) at the right of the figures denoting the units; thus, the expression 18 metres 5 centimetres, is written —18.05 m., 56.746 is read —56 metres and 746 millimetres. 1 cm. 2.25 mm. is written as 1.225 cm. (The English system is a 'fractions' system while the Metric system is a 'decimal' system)

Basic Standards: The standard relationship between English measure and Metric measure is as follows :—

1 inch = 25.4 millimetres = 2.54 cm. (exact)
1 metre = 39.37 inches = 1.09 yards
1 litre = 1.76 pints (imp.) = 0.22 galls. (imp.)
1 gallon (imp.) = 4.546 litres
1 gallon (US) = 3.785 litres
1 pound avp = 0.4536 kilograms
1 kilogram = 2.2046 pounds avp

The following fundamental standards or units are used in the metric system :—

For length	metre
For mass	kilogram or gram
For volume	litre
For force	dyne on dyn
For work & energy	erg or dyne-cm.
For heat	gram-calorie or calorie
For power	watt

The metric system of units called the centimetre-gram-second (CGS) system is commonly used in scientific circles, and metre-kilogram-second (MKS) or metre-tonne-second (MTS) system is more generally used in engineering.

COMMERCIAL WEIGHTS OR UNITS OF MASS

British Units

Avoirdupois Units (avp)
Ordinary Commercial Weights

1 dram	= 27.344 grains
16 drams	= 437.5 grains
	= 1 ounce (oz.)
	= 0.911 oz. tr
16 ounces	= 7000 grains
	= 1 pound (lb.)
	= 1.215 lbs. ap, tr
	= 14.583 ozs. tr
14 pounds	= 1 stone
28 pounds	= 1 quarter (qr.)
4 quarters	= 1 cwt.
	= 112 lbs.
20 cwts.	= 1 ton
	= 2240 lbs.

Troy Units (tr)—partly obsolete
For gold and silver etc.

480 grains	= 1 oz. (tr)
	= 1.097 ozs. avp
12 ounces	= 5760 grains
	= 1 lb. (tr)
	= 13.166 ozs. avp
	= 0.823 lb. avp
175 lbs. troy	= 144 lbs. avp
100 ,, ,,	= 82 ,, ,,

An avp pound is heavier than a tr or ap pound, but an avp ounce is lighter than a tr or ap ounce which are of the same weight.

The only recognised pound is the avoirdupois (avp) pound. US and British pound are of the same weight.

Apothecaries' Units (ap)—Dry—Used for medicine

20 grains	= 1 scruple (scr)	The apothecaries' weights and measures are used by chemists in compounding medicines, but drugs and medicines are bought and sold by avoirdupois weight.
3 scruples	= 60 grains	
	= 1 drachm	
8 drachms	= 480 grains	
	= 1 ounce	
12 ounces	= 5760 grains.	
	= 1 pound	

1 pound avp	= 7000 grains	The weight of the "grain" is the same in all systems.
1 pound tr or ap	= 5760 grains	

1 drachm (ap) = 2.194 drams avp
1 dram (avp) = 0.456 drachms ap

American Weights and Measures. The British systems of weights and measures are used in the United States and the following measures are different in capacity or weight although called by the same name: Ton; Hundredweight; Fluid Ounce; Gill; Pint; Quart; Gallon (wet and dry); Bushel; Peck; Dram.

American Units

1 US cwt.	= 100 pounds (avp)	= $^1/_{20}$ US ton
1 US ton (short ton)	= 2000 pounds (avp)	= 20 US cwts.
1000 pounds (avp)	= 1 kip	
1 quintal US	= 0.4536 quintals metric	

Ton & Tonne explained

British ton is called "long ton"; US ton is called "short ton"; and "tonne" is metric ton, (also called tonneau or millier).

1 Br. ton	= 1.120 US tons	1 US ton	= 0.8929 Br. tons
	= 1.016 tonnes		= 0.9072 tonnes
	= 2240 lbs. avp		= 2000 lbs. avp
	= 20 cwts. Br.		= 17.857 cwts. Br.
	= 22.4 cwts. US		= 20 cwts. US
	= 1016.06 kilograms		= 907.18 kilograms
1 tonne	= 0.9842 Br. tons		
	= 1.1023 US tons		
	= 2204.6 lbs. avp		
	= 19.68 cwts Br.		
	= 22.05 cwts. US		
	= 1000 kilograms		

1 shipping ton (merchandise) is 42 cu. ft. in UK and 40 cu. ft. in US

Metric Units of Weights

1 tonne	= 1,000,000 grams	= 1000 kilograms	
1 quintal (q)	= 100,000 ,,	= 100 ,,	
1 myriagram (mg)	= 10,000 ,,	= 10 ,,	
1 *kilogram* (kg)	= 1,000 ,,	= 1 ,,	
1 hectogram (hg)	= 100 ,,	= 1/10 ,,	
1 dekagram (dkg)	= 10 ,,	= 1/100 ,,	
1 *gram*(g)	= 1 ,,	= 1/1000 ,,	= 1000 mg.
1 decigram (dg)	= 1/10 ,,	= 10 cg.	
1 centigram (cg)	= 1/100 ,,	= 10 mg.	
1 milligram (mg)	= 1/1000 ,,		
1 microgram	1/100,000 ,,		

The gram is the weight of one cubic centimetre or one millilitre of water; the kilogram is the weight of one litre of water; and the tonne is the weight of one cubic meter of water.

The gram is used in weighing gold, silver, letters and small quantities of things. The kilogram is used by grocers and is the primary unit of mass. The tonne is used for heavy articles. (dekagram = decagram; gram = gramme; kilogram = kilogramme; milligram = milligramme. Metric ton is called tonne.)

Grams = grains	× 0.065	Grams ×	15.43	=	grains
= drachms ap	× 3.9	×	0.032	=	oz. ap or tr
= drams avp	× 1.77	×	0.035	=	oz. avp
= oz. ap	× 31.10	Kg. ×	2.2	=	lbs. avp
= oz. avp	× 28.35				
= lbs. avp	× 453.6				

(Usually 1 grain is taken=60 mg. and 1 oz. avp = 30 grams)

Carat Weight—for precious stones

1 carat (metric) = 3.086 grains
= 0.200 or $^1/_5$ gram
= 200 milligrams

A carat has different weights at different places.

(*i*) Some of the measures are wholly or partly obsolete, although not mentioned as such, but have been given for clarification to avoid confusion.

(*ii*) Figures for some of the conversion factors may differ slightly in value (due to rounding off), but that will make no difference for practical purposes.

"Avoirdupois" literally means 'things or goods that sell by weight'. "Troy" is derived from Troyes, a town in France where this weight was first used. "Apothecary" means a 'Druggist'.

The term carat is also used to express the fineness of gold, each carat meaning a twenty-fourth part. Pure gold or fine gold is described as "24-carat fine". Standard gold is "22-carat fine" since it contains 22 parts of pure gold out of 24. A "14-carat gold" means that in 24 parts there are 14 parts (by weight) of pure gold.

Conversion Factors

1 grain	=	64.8 milligrams	1 milligram	=	0.0154 grains
	=	0.0648 grams	1 gram	=	15.43 ,,
1 oz. avp	=	28.35 ,,		=	0.035 oz. avp
1 oz. tr or ap	=	31.103 ,,		=	0.032 ,, tr
1 lb. avp	=	453.592 ,,	1 kilogram	=	35.27 ,, avp
	=	0.454 kilograms		=	32.15 ,, tr
1 stone	=	6.35 ,,		=	2.2 lbs. avp
1 cwt. Br.	=	50.802 ,,	1 quintal	=	220.46 ,, ,,
	=	0.508 quintals		=	1.968 cwt. Br.
1 cwt. US	=	45.352 kilograms		=	2.2 quintals US
1 ton Br.	=	1016.06 ,,	1 tonne	=	2204.6 lbs. avp
	=	1.016 tonnes		=	0.9842 tons Br.
1 ton US	=	907.2 kilograms		=	1.1023 ,, US
	=	0.907 tonnes		=	19.68 cwts. Br.

MEASURES OF LENGTH

British Units

12 inches	=	1 foot
3 feet	=	1 yard
5½ yards	=	1 rod, pole or perch
220 yards	=	1 furlong
8 furlongs	=	1 mile
	=	1760 yards
5000 feet	=	1 canal mile

Gunter's Surveying Chain

7.92 inches	=	1 link
25 links	=	1 rod or pole
100 links	=	1 chain* = 66ft.
10 chains	=	1 furlong
80 chains	=	1 mile
10 sq. chains	=	1 acre

*Engineer's chain is 100 ft.

1 mil or milli-inch = one-thousandth of an inch
1 micro-inch = one-millionth of an inch

Nautical Measures. A nautical or sea mile is the distance on the earth's surface at the sea level of one minute of arc (1/60 of a degree) of longitude of earth at the equator. A nautical mile is taken equal to 6080.26 ft. or 1.1515 statute (or land) miles—(5280×1.1515) by the British Admiralty, and 6086.07 ft. or 1.152664 statute miles by the US Coast Survey Deptt. (The statute or land miles being 5280 ft.) The International nautical miles is 6076.12 ft. or 1852 metres, i.e., 1.15078 miles or 1.852 kilometres per hour.

"Knot" is a rate and not a distance and is used for expressing ship's rate of travel : 1 knot is one nautical mile or 6080 ft. per hour=1.1515 land miles (British Admiralty) per hour (or 1.853 kilometres per hour.

Ropes and Cables

6 feet = 1 fathom = 1.83 metres
120 fathoms = 1 cable or cable length=0.22 km.
3 nautical miles = 1 league

Metric Units of Length

1 megametre	=	1,000,000 metres	
1 hectokilometre	=	100,000 ,,	
1 myriametre	=	10,000 ,,	
1 *kilometre* (km)	=	1,000 ,,	
1 hectometre (hm)	=	100 ,,	
1 dekametre (dkm)	=	10 ,,	
1 *metre* (m)	=	1 ,,	=10 dm.
1 decimetre (dm)	=	1/10 ,,	=10 cm.
1 centimetre (cm)	=	1/100 ,,	=10 mm.
1 millimetre (mm)	=	1/1000 ,,	
1 micron or micrometre	=	1/1000 millimetre=0.00039 inch	
1 millimicron	=	one--millionth of a millimetre	

The metre is used in ordinary measurements, the centimetre or millimetre in reckoning very small distances or measurements, and the kilometre for roads or great distances.

Conversion Factors

			1 millimetre	=	0.03937 inch
			1 centimetre	=	0.3937 or 2/5 inch
1/25 inch	=	1 millimetre	1 decimetre	=	3.937 inches
1 inch	=	25.4 ,,		=	0.328 feet
	=	2.54 cm. (exact)	1 *metre*	=	39.37 inches
1 foot	=	30.48 centimetres		=	3.281 feet
	=	0.3048 metres		=	1.094 yards
1 yard	=	0.9144 ,,	1 dekametre	=	32.81 feet
	=	91.44 centimetres		=	10.94 yards
1 rod	=	5.029 metres	1 hectometre	=	328 ft. 1 in.
1 furlong	=	0.201 kilometres	1 *kilometre*	=	3280 ft. 10 ins.
	=	201.168 metres		=	1093.63 yards
1 mile	=	1.609 kilometres		=	4.97 furlongs
	=	1609 metres		=	5/8 or 0.621 mile

Metric Equivalents in Millimetres of Fractions of an Inch

Inch	1/16	1/8	3/16	1/4	5/16	3/8	7/16	1/2
mm.	1.587	3.175	4.762	6.350	7.937	9.525	11.112	12.70
Inch	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1
mm.	14.29	15.87	17.46	19.05	20.64	22.22	23.81	25.40

Inch	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09	0.10
mm.	0.254	0.508	0.762	1.016	1.270	1.524	1.778	2.032	2.286	2.540

SQUARE MEASURES OR MEASURES OF SURFACE

British Units

144 sq. inches	=	1 sq. foot
9 „ feet	=	1 „ yard
$30\frac{1}{4}$ „ yards	=	1 „ rod, pole or perch = $272\frac{1}{4}$ sq. ft.
40 „ rods	=	1 rood = 1210 sq. yds.
484 „ yards	=	1 sq. chain
4 roods	=	4840 sq. yards=160 sq. rods=10 sq. chains
	=	1 acre = 43,560 sq. ft.
640 acres	=	1 sq. mile

An acre is the area of a square whose side is 208.71 ft. long.

Metric Units

1 sq. kilometre (km²)	=	1,000,000 sq. metres	=100 hectares (ha)	
1 „ hectometre	=	10,000 „ „	=100 ares=1 ha.	
1 „ dekametre	=	100 „ „	= 1 are	
1 „ metre (m²)	=	1 „ „	=100 sq. dm.	
1 „ decimetre (dm²)	=	1/100 „ „	=100 „ cm.	
1 „ centimetre (cm²)	=	1/10,000 „ „	=100 „ mm.	

sq. metre = centare or centiare; hectare = sq. hectometre
are = sq. dekametre.

The sq. metre is the primary unit of ordinary surfaces or small areas. The are or sq. dekametre, hectare or sq. hectometre are the units of land measures.

Conversion Factors

1 sq. inch	= 6.45 sq. centimetres = 645.2 sq. millimetres	1 sq. millimetre	= 0.00155 sq. ins.
1 sq. foot	= 929.0 sq. centimetres = 0.093 sq. metres	1 sq. centimetre	= 0.155 sq. ins.
1 sq. yard	= 0.836 sq. metres = 0.836 centares	1 sq. decimetre	= 15.50 sq. ins.
1 acre	= 4046.86 sq. metres = 40.47 ares = 0.4047 hectare	1 sq. metre or 1 centare	= 10.76 sq. ft. = 1.196 sq. yds.
1 sq. mile	= 2.590 sq. kilometres = 259 hectares = 640 acres	1 sq. dekametre or 1 are	= 119.6 sq. yds.
		1 sq. hectometre or 1 hectare	= 11,959.85 sq. yds. = 2.471 acres
		1 sq. kilometre	= 247.10 acres = 0.3861 sq. miles

CUBIC MEASURES

British units			Metric Units		
1728 cu. ins.	=	1 cu. ft.	1000 cu. mm.	=	1 cu. cm.
27 cu. ft.	=	1 cu. yd.	1000 cu. cm.	=	1 cu. dm.
			1000 cu. dm.	=	1 cu. metre

Conversion Factors

1 cu. in.	= 16387 cu. mm. = 16.387 cu. cm.	1 cu. cm. (cm³)	= 0.061 cu. ins.
1 cu. ft.	= 28,317 cu. cm. = 28.317 cu. dm. = 0.028 cu. metre	1 cu. dm. (dm³)	= 61.02 cu. ins. = 0.035 cu ft.
1 cu. yd.	= 0.764 cu. metre	1 cu. metre (m³)	= 35.315 cu. ft. = 1.308 cu. yds.

Cm.⁴ × 0.024 = in.⁴	In.⁴ × 41.623 = cm.⁴	ft.⁴ × 0.00863 = m.⁴
Cm.³ × 0.061 = in.³	In.³ × 16.387 = cm.³	m.⁴ × 115.862 = ft.⁴
Cm.² × 0.155 = in.²	In.² × 6.45 = cm.²	ft.³ × 0.0283 = m.³
Cm. × 0.3937 = in.	In. × 2.54 = cm.	m.³ × 35.3147 = ft.³

VOLUME OR CAPACITY

Apothecaries' Fluid Measure—for medicine

60 minims = 1 fl. dram = 3.55 cu. centimetres or millilitres
8 fl. drams = 1 fl. ounce = 28.4 cu. centimetres or millilitres

1 teaspoonful	= 1/8 fl. ounce	= 1 fl. dram
1 desertspoonful	= 2 fl. drams	
1 tablespoonful	= 1/2 fl. ounce	= 4 fl. drams
1 wineglass	= 4 tablespoonfuls	= 2 fl. ounces
1 teacup	= 4 fl. ounces	
1 tumblerful	= 8 fl. ounces	
1 glassful	= 12 fl. ounces	

1 cu. centimetre or 1 millilitre	= 0.035 fl. oz. Br. = 0.034 „ „ US = 0.28 „ drams = 16.9 minims	1 cu. dm. or 1 litre	= 1.76 pints Br. = 0.88 quarts „ = 0.22 galls. „

The following approximate measures are usually taken:

1 minim = 0.06 cu. cm.		1 teaspoonful	= 4 cu. cm.
$1\frac{1}{2}$ „ = 0.1 „ „		1 tablespoonful or ½ fl. ounce	= 15 „ „
15 „ = 1.0 „ „			

Weight of 1 pint of water	=	$1\frac{1}{4}$ lb. or 20 ozs. avp
„ 1 fl. oz. „	=	1 oz. avp
„ 1 „ dram „	=	54.69 grains
„ 1 cu. in. „	=	252.5 „
„ 1 cu. cm. „	=	15.43 „
„ 1 gall. imp. „	=	10 lbs. avp
„ 1 „ US „	=	8.33 „ „

LIQUID OR FLUID MEASURES

British Units

1 fl. oz.	= 1.732 cu. ins.
	= 0.96 fl. oz. US
5 ,, ,,	= 8.669 cu. ins.
	= 1 gill
4 gills	= 34.68 cu. ins.
	= 20 fl. ozs.
	= 1 pint
	= 1.20 pints US
	= 1/8 galls.
	= 3/4 bottles
2 pints	= 69.366 cu. ins.
	= 1 quart
4 quarts	= 277.42 cu. ins.
	= 160 fl. ozs.(Br.)
	= 8 pints
	= 1 gallon
	= 6 bottles

American Units

1 fl. oz.	= 1.805 cu. ins.
	= 1.04 fl. oz. Br.
4 ,, ,,	= 7.220 cu. ins.
	= 1 gill (US)
4 gills	= 28.875 cu. ins.
	= 16 fl. ozs.(US)
	= 1 pint (US)
	= 0.833 pints Br.
2 pints	= 57.75 cu. ins.
	= 1 quart (US)
4 quarts	= 231 cu. ins.
	= 128 fl. ozs. (US)
	= 8 pints (US)
	= 1 gall. (US)
	= 0.8327 galls(imp)

1 barrel = 31½ or 42 gallons US

Dry Measures (partly obsolete)

British Units

2 gallons = 1 peck = 554.8 cu. ins. = 0.321 cu. ft.
4 pecks = 1 bushel = 2219.3 cu. ins. = 1.284 cu. ft. = 8 galls.
= 1.03 bushels US = 0.036 kl.
8 bushels = 1 quarter
1 cu. ft. = 6.24 galls. = 3.1144 pecks = 0.7786 bushels
1 cu. yd. = 168.178 galls. = 84.089 pecks = 21.02 bushels
1 Br. bushel = 1.032 US bushels
1 US ,, = 0.969 Br. ,,

American Units

2 pints = 1 quart = 67.20 cu. ins.
4 quarts = 8 pints = 1 gall. = 268.8 cu. ins. = 1.164 US liq. galls.
8 quarts = 1 peck = 2 galls. = 537.6 cu. ins.
4 pecks = 1 bushel = 8 galls. = 2150.42 cu. ins. = 1.244 cu. ft.
1 cu. ft. contains 7.48 liq. US galls. and 6.428 dry US galls.
1 pint dry US = 1.164 pints liq. US = 0.97 pints Br. = 33.6 cu. ins.
1 pint liq. US = 0.86 pints dry US = 0.833 pints Br. = 28.875 cu. ins.
1 pint Br. (liq.) = 1.03 pints dry US = 1.2 pints liq. US = 34.68 cu. ins.

Metric Units of Liquid Measures

1 kilolitre (kl)	=	1000 litres			
1 hectolitre (hl)	=	100 ,,			
1 dekalitre (dkl)	=	10 ,,			
1 *litre* = 1000 ml.	=	1 ,,	= 10 dl.	= 35.30 fl. oz. Br.	
1 decilitre (dl)	=	1/10 ,,	= 10 cl.	= 35.53 ,, ,,	
1 centilitre (cl)	=	1/100 ,,	= 10 ml.	= 0.353 ,, ,,	
1 millilitre (ml)	=	1/1000 ,,			

1 kilolitre = 1 cu. metre
1 litre = 1 cu. decimetre
1 millilitre = 1 cu. centimetre

Conversion Factors

1 cu. in.	= 16.387 millilitres or cu.cm.
	= 0.577 fl. ounces Br.
1 cu. ft.	= 28.317 litres or cu. dm.
	= 0.028 cu. metres
	= 6.24 galls. imp.
	= 7.48 ,, US
1 cu. yd.	= 764.53 litres
	= 0.764 cu. metres
	= 168.178 galls. imp.
	= 202 ,, US

1 millilitre	= 0.061 cu. in.
1 centilitre	= 0.61 cu. in.
1 decilitre	= 6.10 cu. ins.
1 dekalitre	= 2.2 galls. imp.
	= 2.64 ,, US
1 kilolitre or cu. metre or 1000 litres	= 35.317 cu. ft.
	= 1.308 cu. yds.
	= 219.97 galls. imp.
	= 264.17 ,, US

1 fl. dram Br.	=	3.55 millilitres or cu. cm.	
1 ,, ,, US	=	3.697 ,, ,, ,,	
1 ,, ounce Br.	=	28.413 ,, ,, ,,	= 0.0284 litres
1 ,, ,, US	=	29.574 ,, ,, ,,	= 0.0296 ,,
1 pint Br.	=	0.568 litres or cu. dm.	= 34.68 cu. ins.
1 ,, US liq.	=	0.473 ,, ,,	= 28.875 ,,
1 quart Br.	=	1.136 ,, ,,	= 69.366 ,,
1 ,, US liq.	=	0.946 ,, ,,	= 57.75 ,,

Dry Measures (partly obsolete)

1 pint US	= 0.55 cu. dm.	= 33.6 cu. ins.	
1 quart US	= 1.100 ,, ,,	= 67.20 ,, ,,	= 2 pints
1 peck Br.	= 9.092 ,, ,,	= 554.8 ,, ,,	
1 ,, US	= 8.827 ,, ,,	= 537.6 ,, ,,	= 8 quarts
1 bushel Br.	= 36.374 ,, ,,	= 1.284 ,, ft.	
1 ,, US	= 35.238 ,, ,,	= 1.244 ,, ,,	= 4 pecks

1 cu. metre = 27.49 bushels Br. = 28.38 bushels US

Gallon explained

1 imp. gall.	= 1.20 US galls. liq.
	= 1.03 ,, ,, dry
	= 277.42 cu. ins.
	= 0.16 cu. ft.
	= 4546 cu. cm.
	= 4.546 litres
	= 160 fl. ozs.
	= 6 bottles

1 US gall.	= 0.833 imp. galls.
	= 0.859 US dry ,,
	= 231 cu. ins.
	= 0.134 cu. ft.
	= 3785 cu. cm.
	= 3.785 litres
	= 8 pints liq. US

Cu. cm. = minims × 0.06
= fl. drams × 3.55
= fl. oz. Br × 28.4

Cu. cm. × 16.9 = minims
× 0.28 = fl. drams
× 0.035 = fl. oz. Br.

Litre: Is the volume occupied by one kilogram of water (or one litre of water weighs one kilogram). It is one cu. decimetre or one thousand cu. centimetres.

1 litre = 1.76 pints Br.
= 2.113 „ US
= 0.22 galls. imp.
= 0.264 „ US
= 61.025 cu. ins.
= 0.0353 „ ft.
= 0.88 quarts imp.
= 1.057 „ US
= 35.30 fl. oz. Br.

Litres = fl. oz. × 0.028
= pints × 0.568
= galls. imp. × 4.55

Litres × 35.30 = fl. oz.
× 1.76 = pints Br.
× 0.22 = galls. imp.

Grams / cu. cm. = kg. / cu. dm. = tonnes / cu. metre
Grams / cu. metre = milligrams / litre or 1 cu. decimetre

Equivalents of Moments of Inertia and Section Moduli

Moment of inertia in cm. units
= moment of inertia in inch units × 41.648
Moment of inertia in inch units
= moment of inertia in cm. units × 0.024
Section modulus in cm. units
= section modulus in inch units × 16.397
Section modulus in inch units
= section modulus in cm. units × 0.061

Sundry Units

12 articles or units = 1 dozen
12 dozens = 1 gross
1 score = 20 articles or units

1 quire = 25 sheets of paper
20 quires = 1 ream of paper
= 500 sheets

INDIAN MEASURES & WEIGHTS

8 ratties = 1 masha
12 mashas = 1 tola = wt. of rupee coin
5 tolas = 1 chhatak
4 chhataks = 1 pau = 20 tolas
4 pau = 1 seer = 80 tolas
40 seers = 1 maund

Conversion Factors

1 ratti = 1.875 grains
1 masha = 15 grains
= 1 gram (approx)
1 tola = 180 grains
= 0.41 oz. avp
= 0.375 oz. ap
= 3 drachms ap
= 6.583 drams avp
= 11.66 grams
1 chhatak = 2.057 ozs. avp
= 58.30 grams
1 pau = 8.23 ozs. avp
1 seer = 2.057 lbs. avp
= 32.9 ozs. avp
= 0.933 kilograms
= 933 grams
1 maund = 0.0367 Br. tons
= 0.04114 US tons
= 0.03732 tonnes
= 82.28 lbs. avp
= 0.73 cwts. (Br.)
= 37.324 kilograms
= 0.373 quintals
9 lbs. avp = 350 tolas (exact)
1 oz. avp = 2.43 tolas
1 oz. tr or ap = 2.67 tolas

1 lb. avp = 0.486 seers
1 cwt. Br. = 1.361 maunds
or 1 md. 14.4 srs.
1 ton Br. = 27.222 maunds
or 27 mds. 8.8 srs.
1 ton US = 24.3 maunds

1 gram = 1 masha (approx)
1 kilogram = 1.072 seers
= 86 tolas
1 quintal = 2.68 maunds
= 2 mds. 27.2 seers
1 tonne = 26.79 maunds

Liquid Measures

1 chhatak = 2.057 fl. ozs.
1 pau = 8.23 fl ozs.
= 0.234 litres
3.3 drams = 1 tola
= 11.7 ml. or cu.cm.
1 fl. oz. = 2.43 tolas
1 litre = 1.08 seers
1 seer = 0.93 litres

To convert	× *by*
Seers to lbs. avp	72/35
Maunds to cwts.	36/49
Tons to maunds	$27^1/_2$

POWER, WORK, ENERGY & HEAT

The work done in unit time is called the power. The unit of power in the technical system of measurement is the horse-power. In the British engineering units 1 h.p. = 550 ft.-lbs./sec. In the metric units horse-power is called Cheval-Vapeur (CV) and is equal to 75 kilogram-metres/sec. The fundamental metric unit of power is the watt. 1 watt = 1 joule/sec. = 10^7 ergs/sec.

The fundamental metric units of work and energy are the *erg* or *dyne-cm. and joule.*

Units of Heat

The fundamental unit of heat in the British system is the British Thermal Unit (B.Th U.) which is the quantity of heat required to raise the temperature of 1 lb. of water by 1 deg. F. The unit of heat in the metric system is the *gram-calorie* or *calorie*, which is the quantity of heat required to raise the temperature of one gram of water 1 deg. C. One B.Th.U. is equal to 252 calories. The kilogram-calorie or kilo-calorie (kg.-cal. or kilo-cal.) which is equal to 1000 gram calories is in more frequent practical use, which is the heat required to raise the temperature of one kilogram of water by 1 deg. C. 1 kilo-calorie/kg. = calorie/gram = centigrade heat unit/sec. A practical unit of energy, usually applied to heat is the *joule*.

FORCE

The unit of force in the British system is the poundal (pdl). The dyne or dyn, newton and sthene are the units of force in the metric system.

1 newton = 10^5 dynes ; 1 sthene = 10^8 dynes.

Poundal is the force required to produce an acceleration of 1 foot/sec./sec. in a mass of 1 pound. Dyne is the force required to produce an acceleration of 1 centimetre/sec./sec. in a mass of 1 gram.

For practical purposes the value of g (acceleration due to gravity) is usually taken at 32.2 ft./sec./sec. or 981 cm./sec./sec. (The international recognised exact standard value of g is 32.174 ft./sec./sec. or 980.665 cm./sec./sec.)

Cm./sec./sec. = 0.03281 ft./sec./sec.

Ft./sec./sec. = 30.48 cm./sec./sec. or 0.305 metres/sec./sec.

Conversion Factors

1 h.p. = 1.014 metric h.p.
= 745.7 watts
= 0.7457 kilowatts
= 10.70 kg.-cals./min.
= 76.040 kg.-m./sec.
= 42.44 B.Th.U./min.
= 550 ft.-lbs./sec.
= 33,000 ft.-lbs./min.

1 h.p.-hr. = 0.7457 kw.-hr.
= 641.19 kg.-cals.
= 273,745 kg.-m.
= 63,705,000 ft. poundals
= 2,684.52 kilojoules
= 2.68452 megajoles
= 2,544 B.Th.U.
= 1,980,000 ft.-lbs.

1 metric h.p. or Cheval Vapeur = 0.9863 h. p.
= 735.50 watts
= 0.7355 kilowatts
= 75 kg.-m./sec.
= 542.48 ft.-lbs./sec.

1 metric h.p.-hr.
= 270,000 kg.-m.
= 632.527 kg.-cals.
= 0.7355 kw-hr.

1 B.Th.U. = 1,055 joules
= 252 calories
= 0.252 kg.-cal.
= 0.293 watt hr.
= 0.000293 kw.-hr.
= 107.586 kg.-m.
= 0.000393 h. p.-hr.
= 778 ft.-lbs.

1 watt = 1 joule/sec.
= 6.1156 kg.-m./min.
= 0.01434 kg -cal./min.
= 0.2388 cal. sec.
= 3.414 B.Th.U./hr.
= 44.26 ft.-lbs./min.

1 watt-hr. = 3600 joules
= 3,415 B.Th.U.
= 367.1 kg.-m.
= 0.8605 kg.-cal.
= 0.00134 h.p.-hr.
= 2655 ft.-lbs.

1 kilowatt-hr.
= 1.341 h.p.-hr.
= 3412.14 B.Th.U.
= 859.85 kg. cals.
= 3.600 kilo joules
= 367.098 kg.-m.
= 2,654,200 ft.-lbs.
= 85,429,000 ft.-poundals

1 kilowatt = 1000 watts
= 10 hectowatts
= 1.341 h.p.
= 1.36 metric h.p.
= 101.97 kg.-m./sec.
= 14.34 kg.-cals./min.
= 56.92 B.Th.U./min.
= 737.56 ft.-lbs./sec.

1 kg.-cal./min.
= 0.06972 killowatts
= 0.09351 h. p.
= 51.43 ft.-lbs./sec.

1 B.Th.U./min.
= 17.57 watts
= 0.01757 kilowatts
= 0.02356 h.p.
= 12.96 ft.-lbs./sec.

1 B.Th.U./sq. ft.	=	2.71 kg.-cals./sq. metre
1 B.Th.U./cu. ft.	=	8.89 kg.-cals./cu. metre
1 kg.-cal./sq. metre	=	0.369 B.Th.U./sq. ft.
1 gm.-cal./sq. cm.	=	3.69 B.Th.U./sq. ft.
1 kg.-cal./cu. metre	=	0.1125 B.Th.U./cu. ft.
1 B.Th.U./sq. ft./hr./deg. F.	=	4.88 kg.-cals./sq. metre/hr./deg. C.
1 kg.-cal./sq. metre/hr./deg. C.	=	0.205 B.Th.U./sq. ft./hr./deg. F.

B.Th.U./lb. = 2.326 joules/gram
= 0.5556 kg.-cals./kg.

1 calorie = 4.19 joules
= 4.19 watt-sec.

1 kg.-cal. = 3.968 B.Th.U.
= 427 kg.-m.
= 4,190 joules
= 0.00156 h.p.-hr.
= 0.00116 kw.-hr.
= 3,087 ft.-lbs.

1 kg.-cal./kg. = 1.8 B.Th.U./lb.
= 4.1868 joules/gram

1 joule = 107 ergs
= 0.2388 calories
= 1 watt-sec.
= 0.0002778 watt-hr.
= 0.2778 kilowatt-hr.
= 0.1019 kg.-m.
= 0.737 ft.-lbs.
= 0.00095 B.Th.U.
= 23.73 foot poundals

1 kilo-joule = 0.9478 B.Th.U.

1 mega-joule = 0.3725 hp.-hr.

1 poundal = 13,826 dynes
= 0.03108 lb.-force
= 0.0141 kg.-force

1 lb.-force = 444,822 dynes
= 32.174 poundals
= 0.45359 kg.-force

1 ft.-poundal = 0.04214 joules
= 0.031 ft.-lb. (force)

1 kg.-metre = 9.8145 joules
= 0.0093 B.Th.U
= 2.342 calories
= 0.0027 watt-hr.
= 7.233 ft.-lbs.
= 232.715 ft.-poundals

1 ft.-lb = 1.3569 joules
= 0.1383 kg.-m.
= 32.174 ft.-poundals
= 0.3766 milliwatt-hr.
= 0.324 calories
= 1.356×107 dyne-cm.
= 0.001286 B.Th.U

1 kg.-m./sec. = 0.0131 h.p.
= 0.0133 metric h.p.
= 0.0098 kilowatts
= 7.233 ft.-lbs./sec.

1 ft.-lb./sec. = 1.356 watts.
= 0.00184 metric h.p.
= 0.001818 h.p.
= 0.0717 B.Th.U./min.
= 0.01945 kg.-cal./min.
= 0.13825 kg.-m./sec.

1 mega-dyne = 72.330 poundals
= 1.01972 kg.-force
= 2.24809 lb.-force

1 kg.-force = 980,665 dynes
= 70.93 poundals
= 2.2046 lb.-force

Specific Gravity is the ratio between the weight in air of any given net volume of a substance and the weight of an equal volume of pure water. The weight of any substance is its specific gravity × weight of water per unit volume.

Specific Volume is defined as the volume per unit mass of the substance.

MISCELLANEOUS CONVERSION FACTORS

To Convert	Into	× by
Acre-feet	Cu. feet	43,560
,,	,, metres	1233.48
,,	Galls. imp.	271,327
,,	,, US	325,851
,,	Hectare-metre	0.1234
,,	Kilolitres	1233.48
Ares	Sq. dekametres	1
,,	,, metres	100
,,	,, yards	119.6
Atmosphere	Kg./sq. cm.	1.034
,,	Metres of water	10.342
,,	Feet of water	33.93
,,	Ins. of mercury	29.92
,,	Lbs./sq. in.	14.69
Centigrams	Grains	0.154
,, /litre	Grs./gall. imp.	0.7015
,, ,,	Grams./cu.metre	10
Cm./sec.	Ft./sec.	0.0328
,, ,,	Km./hour	0.036
,, ,,	Metres/min.	0.6
,, ,,	Miles/hour	0.0224
Cu. ft./min.	Cu. cm./sec.	472.0
,, ,,	Galls US/sec.	0.1247
,, ,,	,, imp./min.	6.24
,, ,,	,, ,, /day	9000
,, ,,	Litres/sec.	0.4720
,, /sec.	Galls. imp./min.	374
,, ,,	,, ,, /day	538,176
,, ,,	,, US/min.	448.83
,, ,,	,, ,, /day	646,315
Cu. m./kg.	Cu. ft./lb.	16.018
Ft. of water	Lbs./sq. ft.	62.4
,, ,,	,, /sq. in.	0.4335
,, ,,	Kg./sq. metre	304.8
Feet/min.	Feet/sec.	0.0167
,, ,,	Cm./sec.	0.5080
,, ,,	Km./hour	0.0183
,, ,,	Metres/min.	0.3048
,, ,,	Miles/hour	0.0114

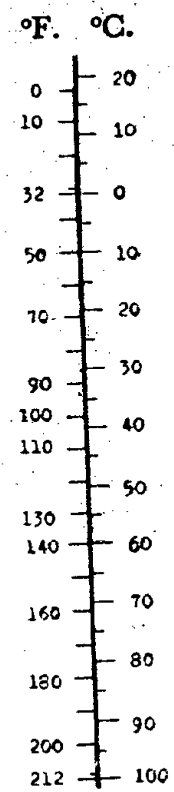
To Convert	Into	× by
Feet/sec.	Cm /sec.	30.48
,, ,,	Km /hour	1.0973
,, ,,	Metres/min.	18.29
,, ,,	Metres/sec.	0.3048
,, ,,	Miles/hour	0.6818
,, ,,	,, /min.	0.0114
Foot-lbs.	Kg -metres	0.1383
Foot-tons	Kg -metres	309.69
,, ,,	Tonnes-metres	0.31
Galls. imp.	Galls. US	1.20
,, /sq. ft.	Litres/sq. metre	48.905
,, /hour	,, /min.	0.076
,, /min.	,, /hour	272.758
,, / ,,	Cu. ft./sec.	0.0027
,, / ,,	Galls./day	1440
Galls.-US of water	Lbs. of water	8.3453
,, ,, /min.	Cu. ft./hour	8.0208
,, ,, ,,	,, ,, /sec.	0.0022
,, ,, ,,	Litres/sec.	0.0631
,, ,, sq. yd.	,,/sq. metre	5.44
Grains/cu. ft.	Grains/gall.	0.1605
,, / cu. in.	Grams/cu. cm.	0.0039
,,/gall. imp.	Parts/million	14.286
,, ,,	Mg /litre	1.4254
,, ,,	Grains/cu. ft.	6.229
,, ,, US	Parts/million	17.118
Grams/cm.	Lbs./foot	0.0672
,, /sq. cm.	,, /sq. in.	0.0142
,, /cu. cm.	Grains/cu. in.	252.01
,, ,,	Lbs./cu. foot	62.43
,, ,,	,, /cu. inch	0.0361
,, ,,	Oz./cu. inch	0.578
,,/cu. metre	Grains/cu. ft.	0.437
,,/litre	,, /gall. imp.	70.12
,, ,,	Lbs./cu. foot	0.0624
,, ,,	Oz./gall. imp.	0.1603
,, ,,	Parts/million	1000
,,/millilitre	Lbs /gall. imp.	10.022

To Convert	Into	× by
Cwt./cu. yd.	Quintals/cu. m.	0.664
Inch-lbs.	Kg -metres	0.0115
,, ,,	Kg -cm.	1.152
,, -tons	,, ,,	25.803
Inch/sec.	Metres/min.	1.5240
Kilograms	Pounds (avp)	2.2046
Kg./cu. cm.	Lbs./cu. inch	36.125
,, / ,, metre	,, / ,, yard	1.6855
,, / ,, ,,	,, / ,, foot	0.062
,, / litre	,, / ,, ,,	62.426
,, / ,,	,, / gall. imp.	10.0221
Kg.-metres	Foot-lbs.	7.233
,, - ,,	,, tons	0.0032
,, - ,,	Inch-lbs.	86.796
,, - ,,	,, - tons	0.0387
Kg./metre	Lbs./ft.	0.6720
,,/sq. metre	,, /sq. ft.	0.2048
,,/ ,, ,,	,, / ,, in.	0.00142
,,/ ,, cm.	,, / ,, ,,	14.2233
,,/ ,, ,,	,, / ,, ft.	2048.17
,,/ ,, ,,	Tons/,, ,,	0.914
Kilometres	Miles	0.6214
,,/hour	Cm./sec.	27.78
,,/ ,,	Feet/min.	54.68
,,/ ,,	,, /sec.	0.9113
,,/ ,,	Metres/sec.	0.278
,,/ ,,	,, /min.	16.67
,,/ ,,	Miles/hour	0.6214
Knots	Miles/hour	1.1515
,,	Metres/sec.	0.5148
,,	Km./hour	1.8532
Litres/min.	Galls./hour	13.1985
,,/sq. metre	,,/sq. ft.	0.0204
,,/sec.	,,/min. imp.	19.00
,,/ ,,	,,/ ,, US	15.84
,,/ ,,	Cu. ft./min.	2.119
Metres	Feet	3.281
,,/min.	Cm./sec.	1.667
,,/ ,,	Feet/sec.	0.0547
,,/ ,,	Km./hour	0.06
,,/ ,,	Miles/hour	0.0373
,,/sec.	Feet/min.	196.85
,,/ ,,	,, /sec.	3.281
,,/ ,,	Km./hour	3.6
,,/ ,,	,, /min.	0.06
,,/ ,,	Knots	1.9435
,,/ ,,	Milles /hour	2.237
,,/ ,,	,, /min.	0.0373

To Convert	Into	× by
Metres of water	Lbs./sq. in.	1.4223
Miles	Kilometres	1.609
,,/ hour	Cm./sec.	44.704
,, ,,	Feet/min.	88
,, ,,	,, /sec.	1.467
,, ,,	Km./hour	1.609
,, ,,	Metres/min.	26.82
,, ,,	,, / sec.	0.4470
,, / min.	Cm./sec.	2682
,, ,,	Feet/sec.	88
,, ,,	Km./min.	1.609
Milliers	Kilograms	1000
Mg./litre	Parts/million	1
Million imp. galls./day	Cu. ft./sec.	1.8568
,, US ,,	,, ,, /sec.	1.5472
,, ,, ,,	Litres / ,,	52.6
,, imp. ,,	,, ,,	63.12
Oz. avp/ft.	Grams/cm.	0.930
,, /sq. yd.	,,/sq. metre	33.91
Oz. liq. imp.	Cu. inches	1.7339
,, /cu. in.	Grams/cu. cm.	1.73
,,/gall. imp.	,, /litre	6.236
,, ,,	Lbs./cu. ft.	0.389
Parts/million	Grains/cu. ft.	4.37
,, ,,	,, /gall. imp.	0.0702
,, ,,	,, / ,, US	0.0584
Pounds/foot	Grams/cm.	14.882
,, ,,	Kg./metre	1.488
,, / inch	Grams/cm.	178.58
,, ,,	Kg./metre	17.858
,, / yard	Grams/cm.	4.961
,, ,,	Kg./metre	0.4961
,,/gall. imp.	Grams/litre	99.779
,, ,,	Kg./litre	0.0998
Pounds/sec.	Tonnes/hour	1.633
,, /cu. ft.	Grams/cu. cm.	0.0160
,, ,, ,,	,, /litre	16.019
,, ,, ,,	Kg./cu. metre	16.019
,, ,, ,,	Oz./gall. imp.	2.569
,, /1000 ft.	Kg./km.	1.4882
,, /cu. inch	Grams/cu. cm.	27.68
,, ,, ,,	Kg./cu cm.	0.0277
,, ,, yard	,, /cu. metre	0.5933
,, /sq. foot	Feet of water	0.0160
,, ,, ,,	Grams/sq. cm.	0.4882
,, ,, ,,	Kg./sq. metre	4.8824
,, /sq. inch	Feet of water	2.3068

To Convert	Into	× by	To Convert	Into	× by
Pounds sq. in.	Grams/sq. cm.	70.307	Tons/sq. ft.	Tonnes/sq. m.	10.937
,, ,,	Metres of water	0.70	,, ,, ,,	Kg./ sq. cm.	1.0937
,, ,,	Kg./sq. cm.	0.0703	,, ,, ,,	,, /sq inch	15.556
,, ,,	,, / sq.metre	703.07	,, ,, yard	Tonnes/sq. m.	1.215
,, ,,	Tons/sq. ft.	0.0643	,,US/ sq. ft.	Tonnes/sq. m.	9.7648
,, ,,	Tonnes/sq. m.	0.7031	,, /cu. yd.	Kg. /cu. metre	1186.5
,, /sq. yd.	Kg./sq. metre	0.543	,, ,, ,,	Tonnes /,, ,,	1.1865
,, of water	Galls. imp.	0.1000	Tonnes/cu.m.	Lbs./cu. inch	0.036
,, ,,	,, US	0.1198	,, ,, ,,	,, /cu. foot	62.43
,, ,,	Cu. feet	0.0160	,, ,, ,,	,, / cu. yard	1685.6
,, ,,	,, inches	27.68	,, ,, ,,	Grams/cu. cm.	1
Tons Br./ft.	Kg./metre	3333.33	,, /sq. ,,	Tons/sq. foot	0.0914
,,/cu. yard	,,/cu. ,,	1329	,, ,,	Lbs./ ,, inch	1.4223
,,/ ,, ,,	Tonnes/cu. m.	1.329	,, /hour	,, / sec.	0.6124
,,/sq. inch	Kg./sq.mm.	1.575	,,-kilometre	Ton-miles Br.	0.6115
,, ,, ,,	,, /sq. cm.	157.488	,, - metres	Foot-tons Br.	3

THERMOMETRIC SCALES



Two thermometric scales are in common use, Fahrenheit and Centigrade or Celsius. Temperature is measured in degrees Fahrenheit (°F.) or degrees Centigrade (°C).

Freezing point of pure water or melting point of ice = 32°F.=O°C.

Boiling point of pure water = 212°F. = 100°C.

Human temperate = 98.4°F. = 37°C.

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Cold water temperature is taken = 45°F.= 7°C.
Cool ,, ,, ,, = 66°F.=19°C.
Temperate ,, ,, ,, = 79°F.=26°C.
Tepid ,, ,, ,, = 86°F.=30°C.
Warm ,, ,, ,, = 99°F.=37°C.
Hot ,, ,, ,, =104°F.=40°C.

O°F. is the melting point of a mixture of equal parts of salt and snow.

To convert °F. to °C.: $°C. = \frac{5}{9}(°F. - 32)$.

,, ,, °C. to °F.: $°F. = \frac{9}{5}\,°C. + 32$.

For most purposes mercury in glass thermometres are used. For recording the interior temperature of a dam, thermocouples are used.

Steam as compared with water occupies 1646 times as much space. It is generally assumed that one cubic inch

	kg/cu. metre
Alum	1700
Aluminium, cast	2580-2700
,, wrought	2640-2800
	kg/sq. metre
,, sheets, per mm thick	2.8
Asbestos cement sheets	
6 mm thick, flat	11
,, ,, corrugated	16
	kg/cu. metre
Ashes and cinders, loose	500-700
Asphalt, solid	2200-2300
	kg/sq. metre
,, mastic as laid, 10 mm thick	22
	kg/cu. metre
Bajri (see Shingle)	— —
Ballast	
Brick	930-1260
Stone,	
dry well shaken	1600-1840
,, ,, ,, loose	1400-1600
,, wet fully	1920-2240
,, consolidated	1920-2080
Basalt	2850-2960
Beaswax	960
Bitumen (av.)	1000
,, cutback	1120
straight-run	1070
Brass	8550
Bricks, common, burnt	1600-2000
,, pressed	1760-1840
,, fire bricks	1760-2000
,, engineering	2160
Brick ballast	930-1260
Brickwork, common	1800-1950
,, machine cut bricks	2400
,, reinforced	1850-2000
,, sundried	1600-1700
	kg/sq. metre
,, 10 cm thick	192

	kg/cu. metre
Carborundum	1600
Cast iron (av.)	7200
,, steel	7840
Castor oil	960
Cement, common	
(grey) loose	1300-1400
tightly packed	1700
one bag	50 kg
rapid hardening	1200
mortar	1920
Chalk stone (av.)	2200
Charcoal wood	320-500
Cinders or clinker	700-880
Clay	
dry, lumps	1040
dry, compact	1440
damp, ,,	1760
dry, rammed	1920-2080
dry, gravelly	2080
wet, compact	2080
Coal	
loose	800-900
dust	960
steam (Bengal)	880
heavy quality	1200-1500
Coal tar	1010
Coke, (Coal)	1000
Concrete, cement	
plain stone ballast	2300
,, reinforced (2% steel)	2400-2500
,, ,, (5% steel)	2580-2700
aerated or cell	260
sawdust	1120
slag foamed	1280
brick aggregate	1850
cinders	1760
coke breeze	1100-1400
clinker	1500-1700
,, lime aggregate	1920-2250
stone aggregate	2250
	kg/sq. metre
Copper sheets, per mm thick	8.69

	kg/cu. metre
,, cast	8790-8940
,, wrought	8840-8940
Cork	240
Crosote	1070
Diesel oil	960
Dolomite	2880
Earth	
dry, loose	1280-1500
moist, loose	1440-1600
dry, rammed	1600
moist, rammed	1760-1840
with sand mixed	2100
Fire clay	2240
,, bricks	1760-2000
	kg/sq. metre
Glass	
rolled, plate, 6 mm thick	17
sheet, per 1 mm thick	2.5
	kg/cu. metre
Grains	560-770
Granite	2640-2800
Gravel, loose	1600-1800
,, rammed	1920-2080
Gun-metal	8640
Gun powder	900
Gutta parcha	970
Gypsum	2240-2400
Gypsum powder	1600
Hemp	320-560
Ice	900
Iron	
ore	2400-3700
pig	7200
grey cast	7030-7130
white cast	7580-7720
wrought	7500-7700
Kankar (stone)	1360-1470
,, lime unslaked	1190
,, ,, slaked	1025
Kerosene	820
Laterite	2080-2400
Lead, solid, cast	11350
,, liquid	10710
	kg/sq. metre
,, sheets per mm thick	11

	kg/cu. metre
Lime-stone	2400-2640
,, in lumps, unslaked	1280-1440
,, freshly burnt in pieces	880-960
,, ground-quicklime	960
,, white, slaked, fresh	580-640
,, after 10 days	800
,, mortar	1760
Linseed oil	330
Loam	
dry, loose	1200
,, , compact	1600
wet, compact	1920
Macadam, bitumen premixed	2200
rolled	2560
Marble	2560-2720
,, grit	1600
Mercury	13600
Mild steel	7850
Mortar	
cement	2080
lime	1760
Mud, river, wet	1760-1920
Oil crude	880
fuel and lubricating	900
castor	960
linseed	930
turpentine	865
Paints, ready mixed	
aluminous	1120
bituminous	1120
chocolate	2500
red lead	3200
white lead	2800
white zinc	2400
Paraffin (wax)	800-960
Peat	560-880
dry	1100
wet	675-690
Petrol	1010
Petroleum	1010
Pitch	
Plaster (see Mortar)	
Plaster of Paris	1760-2400

	kg/cu. metre
Plastics	1060-1600
Porcelain	2350
Pudlo	670
Pumice stone	800-1120
Quartz rock	2650
Read lead and litharge	
dry	2110
paste	8900
Red oxide, dry	1030
Resin	1090
Rip Rap	1280-1440
Roofs (see under "Roofs")	—
Rubber	940
Salt, powder, common	990
,, , rock	1080
Sand, dry, clean	1450-1600
,, river	1840
,, wet	1760-2000
Sand-stone	2240-2400
Shale	2650
Shellac gum	610
Shingle	
aggregate, 3 to 38 mm	1460
fine, dry	1600
,, saturated	2080
Silt, wet	1760-1920
Slag, broken, 12 mm	1450-1520
	kg/sq. metre
Slate, 25 mm thick	72
	kg/cu. metre
Snow,	
freshly fallen	125-190
well compacted	250-800
Soda, caustic	1280
,, silicate	880

	kg/cu. metre
Steel, mild	7850
,, rolled	7840
Stone	
granite	2640-2800
lime-stone	2400-2640
marble	2720
pumice	800-1120
sand-stone	2240-2400
shale	2300-2500
Stone Masonry	
mortar rubble	2500
dry rubble	2080
dry random rubble	2100-2200
granite, ashlar	2640
,, rubble	2400
lime-stone ashlar	2560
marble dressed	2700
sandstone dressed	2240
,, ashlar	2400
Sulphur	2050
Surkhi	1010-1120
Talc	2800
Tallow	930
Tar (av.)	1080
Terra-cotta	1870-2370
Timber (See Section ,, Timber Structures")	
Turpentine	865
Varnishes	960
Water, fresh	1000
,, 1 litre	1 kg
,, sea	1026
Wheat	770-800
White lead dry	1380
Wood-fuel	350
Wood (See Timber)	
Wrought iron	7700
	kg/sq. metre
Zinc sheets, per mm thick	7.1

Note: The values given in the above table for granular materials such as cement, earth, gravel, sand, are really the bulk densities and not the weights of the solid materials. Density of material in bulk is affected by the voids between the particles. True weight of a granular material is its specific gravity × weight of water.

Density is defined as the mass per unit volume.

TEMPERATURE CONVERSION

Centigrade to Fahrenheit

$$°F = \frac{9\ °C}{5} + 32 \qquad °C = \frac{5\ (°F - 32)}{9}$$

C°	F°	C°	F°	C°	F°	C°	F°
0	32	51	123.8	103	217.4	155	311
1	33.8	52	125.6	104	219.2	156	312.8
2	35.6	53	127.4	105	221	157	314.6
3	37.4	54	129.2	106	222.8	158	316.4
4	39.2	55	131	107	224.6	159	318.2
5	41	56	132.8	108	226.4	160	320
6	42.8	57	134.6	109	228.2	161	321.8
7	44.6	58	136.4	110	230	162	323.6
8	46.4	59	138.2	111	231.8	163	325.4
9	48.2	60	140	112	233.6	164	327.2
10	50	61	141.8	113	235.4	165	329
11	51.8	62	143.6	114	237.2	166	330.8
12	53.6	63	145.4	115	239	167	332.6
13	55.4	64	147.2	116	240.8	168	334.4
14	57.2	65	149	117	242.6	169	336.2
15	59	66	150.8	118	244.4	170	338
15.5	60	67	152.6	119	246.2	171	339.8
16	60.8	68	154.4	120	248	172	341.6
17	62.6	69	156.2	121	249.8	173	343.4
18	64.4	70	158	122	251.6	174	345.2
19	66.2	71	159.8	123	253.4	175	347
20	68	72	161.6	124	255.2	176	348.8
21	69.8	73	163.4	125	257	177	350.6
22	71.6	74	165.2	126	258.8	178	352.4
23	73.4	75	167	127	260.6	179	354.2
24	75.2	76	168.8	128	262.4	180	356.0
25	77	77	170.6	129	264.2	181	357.8
26	78.8	78	172.4	130	266	182	359.6
27	80.6	79	174.2	131	267.8	183	361.4
28	82.4	80	176	132	269.6	184	363.2
29	84.2	81	177.8	133	271.4	185	365
30	86	82	179.6	134	273.2	186	366.8
31	87.8	83	181.4	135	275	187	368.6
32	89.6	84	183.2	136	276.8	188	370.4
33	91.4	85	185	137	278.6	189	372.2
34	93.2	86	186.8	138	280.4	190	374
35	95	87	188.6	139	282.2	191	375.8
36	96.8	88	190.4	140	284	192	377.6
37	98.6	89	192.2	141	285.8	193	379.4
38	100.4	90	194	142	287.6	194	381.2
39	102.2	91	195.8	143	289.4	195	383
40	104	92	197.6	144	291.2	196	384.8
41	105.8	93	199.4	145	293	197	386.6
42	107.6	94	201.2	146	294.8	198	388.4
43	109.4	95	203	147	296.6	199	390.2
44	111.2	96	204.8	148	298.4	200	392
45	113	97	206.6	149	300.2	201	393.8
46	114.8	98	208.4	150	302	202	395.6
47	116.6	99	210.2	151	303.8	203	397.4
48	118.4	100	212	152	305.6	204	399.2
49	120.2	101	213.8	153	307.4	204.4	400
50	122	102	215.6	154	309.2	205	401

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SECTION 2

MATHEMETICS

Page

Mathematical Signs and Abbreviations

$+$ plus, positive
$-$ minus, negative
$\pm$ plus or minus
$\mp$ minus or plus
$=$ equal to; equals
$\neq$ not equal to
$>$ greater than
$<$ less than
$\geqq$ $\geqslant$ equal to or greater than
$\leq$ equal to or less than
$\times$ or . multiplied by
$\div$ or / divided by
$\angle$ any angle
$\llcorner$ right angle
$\perp$ perpendicular to
$\because$ since; because
$\therefore$ therefore; hence
() [] parentheses, brackets
: is to, : : so is, : to (proportion) 2 :4 : : 3 :6, 2 is to 4 as 3 is to 6

: ratio; divided by
2 :4, ratio of 2 to 4=2/4
n° *n* degree
n′ *n* minutes
n″ *n* seconds
Δ triangle; delta, difference
∞ infinity
$\parallel$ parallel to
$\nparallel$ not parallel to
$\sqrt{\ }$ square root
$\sqrt[3]{\ }$ cube root
Σ sum of finite quantities
ϕ Phi, θ Theta } any angle
π Pi
$\propto$ varies as; proportional to
a_1, a_2 *a* sub one, a sub two
a^2, a^3 *a* squared, *a* cubed
a^n *a* raised to the power of n

Greek Alphabets or Letters

α Alpha	Δ δ Delta	μ Mu	Ω ω Omega
β Beta	ϕ Phi	π Pi	O o Omicron
γ Gamma	θ Theta	Σ σ Sigma	ζ Zeta

Roman and Arabic Numerals

I	1	IX	9	XVII	17	LX	60	CCCC	400
II	2	X	10	XVIII	18	LXX	70	CD	400
III	3	XI	11	XIX	19	LXXX	80	D	500
IV	4	XII	12	XX	20	XXC	80	DC	600
V	5	XIII	13	XXI	21	XC	90	DCC	700
VI	6	XIV	14	XXX	30	C	100	CM	900
VII	7	XV	15	XL	40	CC	200	M	1000
VIII	8	XVI	16	L	50	CCC	300	MM	2000

1960 = 1000 + 900 + 60 — MCMLX

Functions of π

π = ratio of circumference to diameter of circle $= 22/7 = 3.14159$

$\pi^2 = 9.8696$; $\pi^3 = 31.0063$; $\frac{1}{\pi} = 0.3183$; $\sqrt{\pi} = 1.7724$

LOGARITHMS

$\log 1 = 0$

$$\log_n m = \frac{\log m \text{ to any base}}{\log n \text{ to the same base}}$$

$\log mn = \log m + \log n$; $\log \frac{m}{n} = \log m - \log n$;

$\log x^m = m \log x$; $\log \sqrt[m]{x} = \frac{\log x}{m}$

If $a^x = b$, then $x \log a = \log b$, and $x = \frac{\log b}{\log a}$

$\log 52.23 = 1.7180$ $\log .005223 = \bar{3}.7180$

$\log .5223 = \bar{1}.7180$ $\log .00005223 = \bar{5}.7180$

$\log .05223 = \bar{2}.7180$ $5 \times \bar{2}.7180 = \bar{7}.5900$

$\bar{1}.5223 = -1 + .5223 = -.4777$

MENSURATION

Areas

Square

Diagonal = side $\times \sqrt{2} = 1.414 \times$ side

Area = side^2 = $1.2732 \times$ area of inscribed circle
= $0.6366 \times$ area of circumscribing circle

Dia. of a circle equal in area to square
= $1.1284 \times$ side of square

Dia. of a circle of equal periphery as square
= $1.273 \times$ side of square

Dia. of a circle circumscribed about square
= $1.414 \times$ side of square

Circumference of a circle circumscribing a square
= $4.443 \times$ side of square.

Triangle (See also page 2/15)

Area = $\frac{1}{2} \times$ base $\times$ perpendicular height

$$= \sqrt{s(s-a)(s-b)(s-c)} = \frac{abc}{4R} = rs$$

$$r = \frac{\text{Area}}{s}; \quad R = \frac{abc}{4rs} = \frac{abc}{4 \text{ area}}$$

a b c are sides; $s = \frac{1}{2}(a+b+c)$

R = radius of circumscribed or escribed circle;
r = radius of inscribed circle.

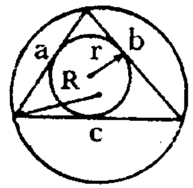
Equilateral Triangle

$$h = \frac{a\sqrt{3}}{2}; \quad a = \frac{2h}{\sqrt{3}}; \quad r = \frac{a}{2\sqrt{3}} = \frac{a\sqrt{3}}{6}; \quad R = \frac{a}{\sqrt{3}} = 2r$$

$$\text{Area} = \frac{a^2\sqrt{3}}{4} = 0.433\, a^2$$

h = height; a = side

Isosceles Triangle



Area $= \frac{c\sqrt{(4a^2 - c^2)}}{4}$ c = base; a = sides

Circle (See also under "Road Curves".)

Area $= \frac{1}{4}\pi \times \text{dia.}^2 = \pi \times r^2 = 0.7854 \times \text{dia.}^2$
$= 3.1416\ r^2 = 0.07958\ \text{cir.}^2 = \frac{1}{4}\ \text{cir.} \times \text{dia.}$

Circumference $= \pi d = r \times 6.283 = 3.5449\ \sqrt{\text{area}}$

Dia. $= \text{cir.} \times 0.3183 = 1.1283\ \sqrt{\text{area}} = \sqrt{(\text{area} \div 0.7854)}$

Side of a square equal in area = dia. × 0.8862 = cir. × 0.285

Side of an inscribed square = dia. of circle × 0.707
= cir. × 0.225

Side of an inscribed equilateral triangle = dia. × 0.86

Side of a square of equal periphery as circle = dia. × 0.785

Quadrilateral inscribed in a circle:

Area $= \sqrt{(s-a)(s-b)(s-c)(s-d)}$
a b c d are sides ; $s = \frac{1}{2}(a+b+c+d)$

Area of a circle varies as the square of its diameter: any circle whose diameter is double that of another contains four times the area of the other.

The area of a circle is equal to the area of a triangle whose base equals the circumference and perpendicular equals the radius.

Semi-circle.

Centre of gravity lies at a distance of $\frac{4r}{3\pi}$ from the diametre.

Arc of a Circle. An arc is any part of a circumference BCD.

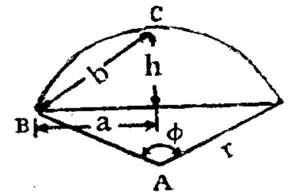
Length of an arc $= \frac{\phi}{360} \times 2\pi r = \frac{\phi \times r}{57.3}$

$= \phi \times r \times 0.01745 = \frac{2 \times \text{area of sector}}{r}$ or $\frac{8b - 2a}{3}$ (approx.)

ϕ = central angle of the arc in degrees; r = radius;
b = chord of half arc; a = $\frac{1}{2}$ chord of arc.

A chord is a straight line connecting two points on the circumference: BD.

Sector of a Circle. A sector is the space included between an arc and two radii drawn to the centre: ABCD.



Area $= \frac{\phi}{360} \times \pi r^2 = \frac{1}{2}$ radius × L

L is length of arc BCD $L = \frac{\phi \pi r}{180}$

ϕ is in degrees.

Centre of gravity of sector of a circle (ABCD) lies at a distance of $2/3\ r \times \frac{(2a)}{L}$ from A on the line AC $= 2/3\ r \times \frac{\sin \phi}{\phi}$

Segment of a Circle. A segment is that part of the circle contained between the arc and its chord (BCD).

Area $= 4/3\ h\ \sqrt{(a^2 + 2/5\ h^2)} = \frac{(L \times r) - 2a(r-h)}{2}$

$= 2/3\ L \times h = 4/3\ a \times h$ (approx.)

h = rise; a = $\frac{1}{2}$ BD.

Centre of gravity of segment of a circle (BCD) lies at a distance of $\frac{(2a)^3}{12 \times \text{Area}}$ or $\frac{BD^3}{12 \times \text{Area}}$ from A on the line AC.

$r = \frac{h^2 + a^2}{2h} = \frac{b^2}{2h}$; $h = r - \sqrt{(r^2 - a^2)} = \frac{b^2}{2r}$

$a = \sqrt{h(2r-h)} = \sqrt{r^2 - (r-h)^2} = \frac{8b - 3L}{2}$

$b = \sqrt{2rh} = \frac{3L + 2a}{8}$

Fillet. Area $= 0.215 r^2 = 0.1075\ c^2$

Distance of centre of area or centre of gravity of a semi-circle arc from diameter $= \frac{2r}{\pi}$

Distance of centre of area of surface of a hemisphere from diameter $= \frac{r}{2}$

Rhombus. Area $= \frac{1}{2}\ d_1 d_2$ $d_1 d_2$ are diagonals

Ellipse. Area $= \frac{1}{4}\pi Dd = 0.7854 \times Dd$ D = major or long axis;
(Long axis $\times$ 0.7854 = short axis) d = minor or short axis.

Perimeter or circumference $= \pi \times \sqrt{D^2+d^2 - \frac{(D-d)^2}{8.8}}$

Or approximately $= 3.1416\sqrt{2(a^2+b^2)}$ $b = \frac{1}{2}d;\ a = \frac{1}{2}D$
Or $3.1416\,(a+b)$ Or $1.82 \times D + 1.315d$

Parabola. Area = base $\times$ 2/3 height

Polygons. Area of any regular polygon = radius of inscribed circle (perpendicular drawn from the centre of the figure to the centre of side) $\times\ \frac{1}{2}$ sum of all sides.

Sum of interior angles of any polygon, regular or irregular = $180^\circ \times$ (number of sides—2).

A regular polygon has equal sides and equal angles.

Hexagon. Area $= 0.866\ d^2 = 0.649\ D^2$
Octagon. Area $= 0.828\ d^2 = 0.707\ D^2$
d = dia. of inscribed circle or short dia.; D = dia. of circumscribed circle or long dia.

TABLE OF REGULAR POLYGONS

Name of Polygon	No. of Sides	Angle Deg.	Area = $S^2 \times$	R = S $\times$	Side = R $\times$	Side = r $\times$	r = S $\times$
Triangle	3	60°	0.433	0.577	1.732	3.464	0.289
Tetragon	4	90°	1.000	0.707	1.414	1.000	0.500
Pentagon	5	108°	1.721	0.851	1.176	1.454	0.688
Hexagon	6	120°	2.598	1.000	1.000	1.155	0.866
Octagon	8	135°	4.828	1.307	0.765	0.828	1.207
Decagon	10	144°	7.694	1.618	0.618	0.650	1.538
Dodecagon	12	150°	11.196	1.932	0.517	0.543	1.866

Angle = angle contained between two sides; S = side of polygon; R = radius of circumscribed circle; r = radius of inscribed circle.

Trapezium. Area = sum of parallel sides $\times\ \frac{1}{2}$ height

Irregular Figures. **Simpson's Rule:** Divide the area or figure into an even number (n) of parallel strips by means of (n+1) ordinates, spaced equal distances, d.

Area $= \frac{1}{3}d$ [first ordinate + last ordinate + 2(sum of all intermediate odd — numbered ordinates) + 4(sum of all intermediate even — numbered ordinates)]

Volumes and Surfaces

Diagonal of a *Cube* = edge of cube $\times \sqrt{3}$
Diagonal of a *rectangular solid* $= \sqrt{(\text{length}^2 + \text{breadth}^2 + \text{depth}^2)}$

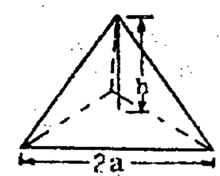
Pyramids. A pyramid is a solid whose base is a polygon and whose sides are triangles uniting at a common point called the vertex.

Circular Cones. A cone is a solid whose base is a circle and whose convex surface tapers uniformly to a point called the vertex.

Volume $= \frac{1}{3} \times$ area of base $\times$ vertical height
Convex area $= \frac{1}{2} \times$ perimeter of base $\times$ slant height
$= \pi r\sqrt{(r^2+h^2)}$ r is radius and h = vertical height
Centre of gravity $= \frac{1}{4}$ vertical height above base.

Regular Tetrahedron or Pyramid. It has four equal sides.

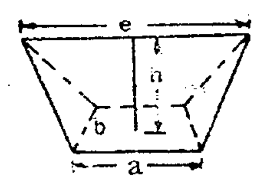
Volume $= \frac{2a^3\sqrt{2}}{3}$; $h = (2a) \times \sqrt{2}/3$
Surface area $= (2a)^2 \times \sqrt{3}$



Wedge on rectangular base

Volume $= bh/6\,(2a+e) = A/3\,(2a+e)$
or Area of base $\times \frac{1}{2}h$—(approx.)

If the side opposite to a is of different size than a, take a+ the other side instead of 2a.



Sphere

Surface area $=\pi\times$ dia.$^2=4\pi r^2=$ cir. $\times$ dia. $=$ cir.$^2\times 0.3183$

Volume $=$ surface area $\times 1/6$ dia. $=1/6\times\pi d^3=0.5236\ d^3$.
$=4/3\pi r^3$

Side of an equal cube $=$ dia. of sphere $\times 0.806$

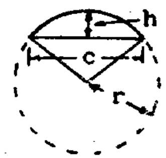
Side of inscribed cube $=1.1547\times$ radius

Length of an equal cylinder $=$ dia. of sphere $\times 0.6667$

Hemisphere—half-sphere

Total surface area $=3\pi r^2$; Volume $=2/3\pi r^3$

Centre of gravity of half-sphere $=3/8$ r above spherical centre.



Spherical Sector

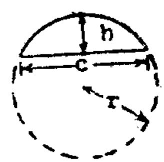
Volume $=2/3\ \pi r^2 h=2.094 r^2 h$; $S=\pi r(2h+\frac{1}{2}c)$

$c=2\sqrt{h\ (2r-h)}$

Centre of gravity above centre of sphere $=\frac{3}{4}\ (r-\frac{1}{2}h)$

S = total area of conical and spherical surface.

Spherical Segment



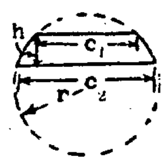
Volume $=\pi h^2(r-h/3)=\pi h\ (c^2/8+h^2/6)$

Area of spherical surface
$=2\pi\ rh=\pi\ (c^2/4+h^2)$

$c=2\sqrt{h\ (2r-h)}$ $r=\dfrac{c^2+4h^2}{8h}$

Centre of gravity above base of segment $=\dfrac{h(4r-h)}{4(3r-h)}$

Spherical Zone. A spherical zone is the part of a sphere inculded between two parallel zones.



$$\text{Volume}=\frac{\pi h}{6}\left(\frac{3c_1^2}{4}+\frac{3c_2^2}{4}+h^2\right)$$

Area of spherical or convex surface only $=2\pi rh$

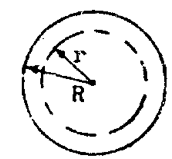
Spherical Wedge

$$\text{Volume}=\frac{a^\circ}{360^\circ}\times\frac{4\pi r^3}{3}$$

$$\text{Area of spherical surface}=\frac{a^\circ}{360^\circ}\times 4\pi r^2$$

Hollow Sphere or Spherical Shell

Volume $=4/3\ \pi\ (R^3-r^3)$

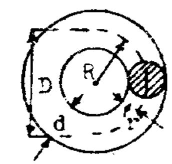


When the thickness of the shell is very small compared with outer diameter :

Volume $=\pi D^2 h$ (approx) :

D is outer dia., h is thickness.

Cylindrical Ring or Torus



Volume $=2\pi^2 Rr^2=\frac{1}{4}\pi^2 Dd^2=2.47\times Dd^2$

Area of spherical surface
$=4\pi^2 Rr=\pi^2 Dd=9.8696\ Dd$.

Prismoids

Volume $=h/6\ (A_1+A_2+4A)$

End faces are in parallel planes.

$A_1\ A_2$ are the areas of ends, top and bottom;
A is the area of mid section parallel to ends,
h is the length between ends, *or* height.

Frusta of Pyramids and Cones. If a pyramid or cone is cut by a plane parallel to the base, so as to form two parts, the lower part is called the frustum of the pyramid or cone.

Volume $=h/3(A_1+A_2+\sqrt{A_1A_2})$

Area of spherical surface
$=s/2\ (P+p)$

s is slant height,
P, p, are perimeter of ends.

$$\text{Centre of gravity above base}=\frac{h}{4}\left(\frac{3A_1+A_2+2\sqrt{A_1A_2}}{A_1+A_2+\sqrt{A_1A_2}}\right)$$

ALGEBRIC FORMULAE

a^n = a raised to the nth power

$a^{2/3} = \sqrt[3]{a^2}$; $a^{3/2} = \sqrt{a^3}$; $a^{-1} = \frac{1}{a}$; $a^{-2} = \frac{1}{a^2}$

$8^2 = 4^3 = 2^6 = 64$

$x^{1/2} = \sqrt{x}$; $x^{1/3} = \sqrt[3]{x}$

$x^{-m} = \frac{1}{x^m}$; $x^m = \left(\frac{1}{x}\right)^{-m}$; $\frac{1}{x^m} = \sqrt[m]{x}$

$a^m \times a^n = a^{m+n}$; $a^m \times b^m = (ab)^m$; $(a^m)^n = a^{mn}$

$a^m \times a^n \times a^x = a^{m+n+x}$

$a^m \div b^m = \left(\frac{a}{b}\right)^m$; $a^m \div a^n = a^{m-n}$

$10^0 = 1$; $10^1 = 10$; $10^2 = 100$; $10^3 = 1000$, etc.

$10^{-1} = \frac{1}{10} = 0.1$; $10^{-2} = \frac{1}{100} = 0.01$; $10^{-3} = \frac{1}{1000} = 0.001$

$n + (-m) = n - m = -(m - n)$

$n - (+m) = n + (-m)$; $n - (-m) = n + (+m) = n + m$

$(-n) \times (-m) = +mm$; $(+n)(-m) = -nm$

$(-a) \div (-b) = a \div b = + \frac{a}{b}$

$(-a) \div (+b) = (+a) \div (-b) = -\frac{a}{b}$

$(a+b)^2 = a^2 + b^2 + 2ab$; $(a-b)^2 = a^2 + b^2 - 2ab$

$(a+b)(a-b) = a^2 - b^2$; $a^2 + b^2 = (a+b)^2 - 2ab$

$(a+b)^2 - (a-b)^2 = 4ab$

$\left(\frac{a+b}{2}\right)^2 - \left(\frac{a-b}{2}\right)^2 = ab$

$(x+a)(x+b) = x^2 + (a+b)x + ab$

$(x-a)(x-b) = x^2 - (a+b)x + ab$

$a^3 - b^3 = (a-b)(a^2+b^2+ab)$; $a^3 + b^3 = (a+b)(a^2+b^2-ab)$

$(a+b+c)^2 = a^2+b^2+c^2+2ab+2bc+2ac$

$(a+b-c)^2 = a^2+b^2+c^2+2ab-2bc-2ac$

$(a+b)^3 = a^3+3a^2b+3ab^2+b^3 = a^3+b^3+3ab(a+b)$

$(a-b)^3 = a^3-3a^2b+3ab^2-b^3 = a^3-b^3-3ab(a-b)$

$(a+b)(a^2-ab+b^2) = a^3+b^3 = (a+b)^3-3ab(a+b)$

$(a-b)(a^2+ab+b^2) = a^3-b^3 = (a-b)^3+3ab(a-b)$

$(x+a)(x+b)(x+c) = x^3+(a+b+c)x^2+(bc+ac+ab)x+abc$

Quadratic Equations

If $ax^2+bx+c=0$, then $x = \frac{-b \pm \sqrt{b^2-4ac}}{2a}$

If $x^2+ax=b$, then $x = -\frac{a}{2} \pm \sqrt{b + \left(\frac{a}{2}\right)^2}$

If $x^{2n}+ax^n=b$, then $x = \sqrt[n]{-\frac{a}{2} \pm \sqrt{b + \left(\frac{a}{2}\right)^2}}$

Cubic Equations If $x^3+ax+b=0$,

$$x = \left(-\frac{b}{2} + \sqrt{\frac{a^3}{27} + \frac{b^2}{4}}\right)^{\frac{1}{3}} + \left(-\frac{b}{2} - \sqrt{\frac{a^3}{27} + \frac{b^2}{a}}\right)^{\frac{1}{3}}$$

If $x+y=s$, and $xy=p$, then $x = \frac{s+\sqrt{s^2-4p}}{2}$, $y = \frac{s-\sqrt{s^2-4p}}{2}$

TRIGONOMETRY

Trigonometry is that branch of mathematics which treats of the properties of angular functions and their application to the solution of triangles.

Radian. A radian is the angle subtended at the centre of a circle by an arc whose length is equal to the radius of the circle, and is a constant angular measurement.

1 radian $= 57° - 17' - 44'' = 180/\pi$ deg. $= 57.296$ deg.

1 deg. $= 0.0175$ radians. π radians $= 180$ deg.

Sine=sin; Cosine=cos; Tangent=tan; Cotagent=cot;
Secant=sec; Cosecant=cosec or csc;
Perpendicular=perp.; Hypotenuse=hyp.

$$\text{Sin } A = \frac{\text{perp.}}{\text{hyp.}} = \frac{a}{c} = \frac{1}{\text{cosec } A} = \cos B$$

$$\text{Cos } A = \frac{\text{base}}{\text{hyp.}} = \frac{b}{c} = \frac{1}{\sec A} = \sin B$$

$$\text{Tan } A = \frac{\text{perp.}}{\text{base}} = \frac{a}{b} = \frac{1}{\cot A} = \cot B$$

$$\text{Cot } A = \frac{\text{base}}{\text{perp.}} = \frac{b}{a} = \frac{1}{\tan A} = \tan B$$

$$\text{Sec } A = \frac{\text{hyp.}}{\text{base}} = \frac{c}{b} = \frac{1}{\cos A} = \text{cosec } B$$

$$\text{Cosec } A = \frac{\text{hyp.}}{\text{perp.}} = \frac{c}{a} = \frac{1}{\sin A} = \sec B$$

Versine A $= 1 - \cos A$

Coversine A $= 1 - \sin A$

$$(\sin A)^{-1} = \frac{1}{\sin}$$

$\sin A \text{ cosec } A = 1$

$\cos A \sec A = 1$

$\tan A \cot A = 1$

$\sin^2 A + \cos^2 A = 1$

$\sec^2 A - \tan^2 A = 1$

$\text{cosec}^2 A - \cot^2 A = 1$

$1 + \tan^2 A = \sec^2 A$

$1 + \cot^2 A = \text{cosec}^2 A$

$$\tan A = \frac{\sin A}{\cos A} = \sqrt{\sec^2 A - 1}$$

$$\cot A = \frac{\cos A}{\sin A} = \sqrt{\text{cosec}^2 A - 1}$$

$$\sin A = \sqrt{1-\cos^2 A} = \sqrt{\tan A \cos A} = 2 \sin \tfrac{1}{2}A \cos \tfrac{1}{2}A = \sqrt{\tfrac{1}{2}(1-\cos 2A)}$$

$$\cos A = \sqrt{1-\sin^2 A} = \cot A \sin A = 2\cos^2 \tfrac{1}{2}A - 1 = 1 - 2\sin^2 \tfrac{1}{2}A = \cos^2 \tfrac{1}{2}A - \sin^2 \tfrac{1}{2}A$$

Function of the Sum and Difference of Two Angles

$$\sin(A \pm B) = \sin A \cos B \pm \cos A \sin B$$

$$\cos(A \pm B) = \cos A \cos B \mp \sin A \sin B$$

$$\tan(A \pm B) = \frac{\tan A \pm \tan B}{1 \mp \tan A \tan B}; \quad \cot(A \pm B) = \frac{\cot B \cot A \mp 1}{\cot B \pm \cot A}$$

Functions of ½ A

$$\sin \tfrac{1}{2}A = \sqrt{\tfrac{1}{2}(1-\cos A)} = \tfrac{1}{2}\sqrt{1+\sin A} - \tfrac{1}{2}\sqrt{1-\sin A}$$

$$\cos \tfrac{1}{2}A = \sqrt{\tfrac{1}{2}(1+\cos A)} = \tfrac{1}{2}\sqrt{1+\sin A} + \tfrac{1}{2}\sqrt{1-\sin A}$$

$$\tan \tfrac{1}{2}A = \sqrt{\frac{1-\cos A}{1+\cos A}} = \frac{1-\cos A}{\sin A} = \frac{\sin A}{1+\cos A} = \frac{\tan A}{1+\sec A} = \text{cosec } A - \cot A$$

$$\cot \tfrac{1}{2}A = \sqrt{\frac{1+\cos A}{1-\cos A}} = \frac{1+\cos A}{\sin A} = \frac{\sin A}{1-\cos A} = \frac{1}{\text{cosec } A - \cot A}$$

The complement of an angle A=90°—A
The supplement of an angle A=180°—A

sin A	=cos(90°—A)	cot A	=tan(90°—A)
	=sin (180°—A)		=—cot (180°—A)
sin(—A)	=—sin A	cot(—A)	=—cot A
sin(90°+A)	=+cos A	cot(90°+A)	=—tan A
sin(90°—A)	=+cos A	cot(90°—A)	=+tan A
sin(180°+A)	=—sin A	cot(180°+A)	=+cot A
sin(180°—A)	=+sin A	cot(180°—A)	=—cot A
cos A	=sin(90°—A)	sec A	=cosec (90°—A)
	=—cos(180°—A)		=—sec(180°—A)
cos(—A)	=+cos A	sec(—A)	=+sec A
cos(90°+A)	=—sin A	sec(90°+A)	=—cosec A
cos(90°—A)	=+sin A	sec(90°—A)	=+cosec A
cos(180°+A)	=—cos A	sec(180°+A)	=—sec A
cos(180°—A)	=—cos A	sec(180°—A)	=—sec A
tan A	=cot(90°—A)	cosec A	=sec (90°—A)
	=—tan(180°—A)		=cosec(180°—A)
tan(—A)	=—tan A	cosec(—A)	=—cosec A
tan(90°+A)	=—cot A	cosec(90°+A)	=+sec A
tan(90°—A)	=+cot A	cosec(90°—A)	=+sec A
tan(180°+A)	=+tan A	cosec(180°+A)	=—cosec A
tan(180°—A)	=—tan A	cosec(180°—A)	=+cosec A

Functions of 2 A

$$\sin 2A = 2 \sin A \cos A = \frac{2 \tan A}{1+\tan^2 A}$$

$$\cos 2A = \cos^2 A - \sin^2 A = 2 \cos^2 A - 1 = 1 - 2 \sin^2 A$$

$$\tan 2A = \frac{2 \tan A}{1-\tan^2 A} \qquad \cot 2A = \frac{\cot^2 A - 1}{2 \cot A}$$

Products and Powers of Functions

$$\sin A \cos B = \tfrac{1}{2}\,[\sin(A+B)+\sin(A-B)]$$

$$\cos A \sin B = \tfrac{1}{2}\,[\sin(A+B)-\sin(A-B)]$$

$$\cos A \cos B = \tfrac{1}{2}\,[\cos(A+B)+\cos(A-B)]$$

$$\sin A \sin B = \tfrac{1}{2}\,[\cos(A-B)-\cos(A+B)]$$

$$\sin^2 A = \tfrac{1}{2}(1-\cos 2A); \qquad \cos^2 A = \tfrac{1}{2}(1+\cos 2A)$$

Sums and Differences of Functions

$$\sin A + \sin B = 2 \sin \tfrac{1}{2}(A+B) \cos \tfrac{1}{2}(A-B)$$

$$\sin A - \sin B = 2 \cos \tfrac{1}{2}(A+B) \sin \tfrac{1}{2}(A-B)$$

$$\cos A + \cos B = 2 \cos \tfrac{1}{2}(A+B) \cos \tfrac{1}{2}(A-B)$$

$$\cos A - \cos B = 2 \sin \tfrac{1}{2}(A+B) \sin \tfrac{1}{2}(B-A)$$

$$\tan A + \tan B = \frac{\sin(A+B)}{\cos A \cos B}; \qquad \cot A + \cot B = \frac{\sin(A+B)}{\sin A \sin B}$$

$$\tan A - \tan B = \frac{\sin(A-B)}{\cos A \cos B}; \qquad \cot A - \cot B = -\frac{\sin(A-B)}{\sin A \sin B}$$

$$\sin^2 A - \sin^2 B = \sin(A+B) \sin(A-B) = \cos^2 B - \cos^2 A$$

$$\cos^2 A - \cos^2 B = \sin(A+B) \sin(B-A)$$

$$\cos^2 A - \sin^2 B = \cos(A+B) \cos(A-B)$$

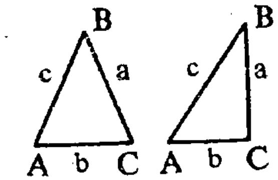
Solution of Triangles

(See also under Mensuration)

Relations between Angles and Sides of Plane Triangles:

Let a, b, c, = sides of triangle and A, B, C, = angles opposite a, b, c, respectively. $A+B+C=180°$.

Law of Sines : $\frac{a}{\sin A} = \frac{b}{\sin B} = \frac{c}{\sin C} = \frac{abc}{2\,\text{Area}} = 2R$



R = radius of escribed or circumscribing circle;

r = radius of inscribed circle.

$$R = \frac{a}{2 \sin A} = \frac{b}{2 \sin B} = \frac{c}{2 \sin C}$$

$$r = \frac{\text{Area}}{s} = (s-a)\tan \tfrac{1}{2}A = (s-b)\tan \tfrac{1}{2}B = (s-c)\tan \tfrac{1}{2}C$$

Area $= \tfrac{1}{2}(ab \sin C) = \tfrac{1}{2}(bc \sin A) = \tfrac{1}{2}(ac \sin B)$

$= \sqrt{s(s-a)(s-b)(s-c)} = rs \; ; \; s = \tfrac{1}{2}(a+b+c)$

Given two angles, A and B, and a side b, then

$$a = b\frac{\sin A}{\sin B}; \qquad c = b\frac{\sin C}{\sin B}$$

Law of Cosines: Given two sides and the included angle

$$a = \sqrt{b^2+c^2-2\,bc \cos A}$$

$$c = \sqrt{a^2+b^2-2\,ab \cos C} \; ; \; b = \sqrt{c^2+a^2-2\,ca \cos B}$$

Given three sides a, b and c, then

$$\cos A = \frac{b^2+c^2-a^2}{2\,bc}; \quad \cos B = \frac{c^2+a^2-b^2}{2\,ca}; \quad \cos C = \frac{a^2+b^2-c^2}{2\,ab}$$

$a = b \cos C + c \cos B$; $b = c \cos A + a \cos C$; $c = a \cos B + b \cos A$

Right angled triangles

$a = c \sin A = b \tan A$; $b = c \cos A = a \cot A$;

$c = a \operatorname{cosec} A = b \sec A$

n	n^2	n^3	$\sqrt{n}$	$\sqrt[3]{n}$	$\sqrt{10n}$	$\sqrt[3]{10n}$	$\sqrt[3]{100n}$	$1/n$
1	1	1	1	1	3·162	2·154	4·642	1
2	4	8	1·414	1·260	4·472	2·714	5·848	·5000
3	9	27	1·732	1·442	5·477	3·107	6·694	·3333
4	16	64	2	1·587	6·325	3·420	7·368	·2500
5	25	125	2·236	1·710	7·071	3·684	7·937	·2000
6	36	216	2·449	1·817	7·746	3·915	8·434	·1667
7	49	343	2·646	1·913	8·367	4·121	8·879	·1429
8	64	512	2·828	2·000	8·944	4·309	9·283	·1250
9	81	729	3·000	2·080	9·487	4·481	9·655	·1111
10	100	1000	3·162	2·154	10·000	4.642	10·000	·1000
11	121	1331	3·317	2·224	10·488	4·791	10·323	·09091
12	144	1728	3·464	2·289	10·954	4·932	10·627	·08333
13	169	2197	3·606	2·351	11·402	5·066	10·914	·07692
14	196	2744	3·742	2·410	11·832	5·192	11·187	·07143
15	225	3375	3·873	2·466	12·247	5·313	11·447	·06667
16	256	4096	4·000	2·520	12·649	5·429	11·696	·06250
17	289	4913	4·123	2·571	13·038	5·540	11·935	·05882
18	324	5832	4·243	2·621	13·416	5·646	12·164	·05556
19	361	6859	4·359	2·668	13·784	5·749	12·386	·05263
20	400	8000	4·472	2·714	14·142	5·848	12.599	·05000
21	441	9261	4·583	2·759	14·491	5·944	12·806	·04762
22	484	10648	4·690	2·802	14·832	6·037	13·006	·04545
23	529	12167	4·796	2·844	15·166	6·127	13·200	·04348
24	576	13824	4·899	2·884	15·492	6·214	13·389	·04167
25	625	15625	5·000	2·924	15·811	6·300	13.572	·04000
26	676	17576	5·099	2·962	16·125	6·383	13·751	·03846
27	729	19683	5·196	3·000	16·432	6·463	13·925	·03704
28	784	21952	5·292	3·037	16·733	6·542	14·095	·03571
29	841	24389	5·385	3·072	17·029	6·619	14·260	·03448
30	900	27000	5·477	3·107	17·321	6·694	14·422	·03333
31	961	29791	5·568	3·141	17·607	6·768	14·581	·03226
32	1024	32768	5·657	3·175	17·889	6·840	14·736	·03125
33	1089	35937	5·745	3·208	18·166	6·910	14·888	·03030
34	1156	39304	5·831	3.240	18·439	6·980	15·037	·02941
35	1225	42875	5·916	3·271	18·708	7·047	15·183	·02857
36	1296	46656	6·000	3·302	18·974	7·114	15·326	·02778
37	1369	50653	6·083	3·332	19·235	7·179	15·467	·02703
38	1444	54872	6·164	3·362	19·494	7·243	15·605	·02632
39	1521	59319	6·245	3·391	19·748	7·306	15·741	·02564
40	1600	64000	6·325	3·420	20·000	7·368	15·874	·02500
41	1681	68921	6·403	3·448	20·248	7·429	16·005	·02439
42	1764	74088	6·481	3·476	20·494	7·489	16·134	·02381
43	1849	79507	6·557	3·503	20·736	7·548	16·261	·02326
44	1936	85184	6·633	3·530	20.976	7·606	16·386	·02273
45	2025	91125	6·708	3·557	21·213	7·663	16·510	·02222

See page 1/22

n	n^2	n^3	$\sqrt{n}$	$\sqrt[3]{n}$	$\sqrt{10n}$	$\sqrt[3]{10n}$	$\sqrt[3]{100n}$	$1/n$
46	2116	97336	6·782	3·583	21·448	7·719	16·631	·02174
47	2209	103823	6·856	3·609	21·679	7.775	16·751	.02128
48	2304	110592	6·928	3·634	21·909	7·830	16·869	·02083
49	2401	117649	7·000	3·659	22·136	7·884	16·985	·02041
50	2500	125000	7·071	3·684	22·361	7·937	17·100	·02000
51	2601	132651	7·141	3·708	22·583	7·990	17·213	·01961
52	2704	140608	7·211	3·733	22·804	8·041	17·325	·01923
53	2809	148877	7·280	3·756	23·022	8·093	17·435	·01887
54	2916	157464	7·348	3·780	23·238	8·143	17·544	·01852
55	3025	166375	7·416	3·803	23·452	8·193	17·652	·01818
56	3136	175616	7·483	3·826	23·664	8·243	17·758	·01786
57	3249	185193	7·550	3·849	23·875	8·291	17·863	·01754
58	3364	195112	7·616	3·871	24·083	8·340	17·967	·01724
59	3481	205379	7·681	3·893	24·290	8·387	18·070	·01695
60	3600	216000	7.746	3·915	24·495	8·434	18·171	·01667
61	3721	226981	7·810	3·936	24·698	8·481	18·272	·01639
62	3844	238328	7·874	3·958	24·900	8·527	18·371	·01613
63	3969	250047	7·937	3·979	25·100	8·578	18·469	·01587
64	4096	262144	8·000	4·000	25·298	8·618	18·566	·01562
65	4225	274625	8·062	4·021	25·495	8·662	18·663	·01538
66	4356	287496	8·124	4·041	25·690	8·707	18·758	·01515
67	4489	300763	8·185	4·062	25·884	8·750	18·852	·01493
68	4624	314432	8·246	4·082	26·077	8·794	18·945	·01471
69	4761	328509	8·307	4·102	26·268	8·837	19·038	·01449
70	4900	343000	8·367	4·121	26·458	8·879	19·129	·01429
71	5041	357911	8·426	4·141	26·646	8·921	19·220	·01408
72	5184	373248	8·485	4·160	26·833	8·963	19·310	·01389
73	5329	389017	8·544	4·179	27·019	9·004	19·399	·01370
74	5476	405224	8·602	4·198	27·203	9·045	19·487	·01351
75	5625	421875	8·660	4·217	27·386	9·086	19·574	·01333
76	5776	438976	8·718	4·236	27·568	9·126	19·661	·01316
77	5929	456533	8·775	4·254	27·749	9·166	19·747	·01299
78	6084	474552	8·832	4·273	27·928	9·205	19·832	·01282
79	6241	493039	8·888	4·291	28·107	9·244	19.916	·01266
80	6400	512000	8·944	4·309	28·284	9·283	20·000	·01250
81	6561	531441	9·000	4·327	28·460	9·322	20·083	·01235
82	6724	551368	9·055	4·344	28·636	9·360	20·165	·01220
83	6889	571787	9·110	4·362	28·810	9·398	20·247	·01205
84	7056	592704	9·165	4·380	28·983	9·435	20·328	·01190
85	7225	614125	9·220	4·397	29·155	9·473	20·408	·01176
86	7396	636056	9·274	4·414	29·326	9·510	20·488	·01163
87	7569	658503	9·327	4·431	29·496	9·546	20·567	·01149
88	7744	681472	9·381	4·448	29·665	9·583	20·646	·01136
89	7921	704969	9·434	4·465	29·833	9·619	20·724	·01124
90	8100	729000	9·487	4·481	30·000	9·655	20·801	·01111

n	n^2	n^3	$\sqrt{n}$	$\sqrt[3]{n}$	$\sqrt{10n}$	$\sqrt[3]{10n}$	$\sqrt[3]{100n}$	$1/n$
91	8281	753571	9·539	4·498	30·166	9·691	20·878	·01099
92	8464	778688	9·592	4·514	30·332	9·726	20·954	·01087
93	8649	804357	9·644	4·531	30·496	9·761	21·029	·01075
94	8836	830584	9·695	4·547	30·659	9·796	21·105	·01064
95	9025	857375	9·747	4·563	30·822	9.830	21·179	·01053
96	9216	884736	9·798	4·579	30·984	9·865	21·253	·01042
97	9409	912673	9·849	4·595	31·145	9·899	21·327	·01031
98	9604	941192	9·899	4·610	31·305	9·933	21·400	·01020
99	9801	970299	9·950	4·626	31·464	9·967	21·472	·01010
100	10000	1000000	10·000	4·642	31.623	10·000	21·544	·01000

FUNCTIONS OF ANGLES AT 1 Deg. INTERVALS

∠	Sine	Tan	Cot	Cos	∠	∠	Sine	Tan	Cot	Cos	∠
0	0	0	∝	1	**90**	23	·3907	·4245	2·3559	·9205	67
1	·0175	·0175	57·2900	·9998	89	24	·4067	·4452	2·1445	·9135	66
2	·0349	·0349	28·6363	·9994	88	**25**	·4226	·4663	2·1445	·9063	**65**
3	·0523	·0524	19·0811	·9986	87	26	·4384	·4877	2·0503	·8988	64
4	·0698	·0699	14·3007	·9976	86	27	·4540	·5095	1·9626	·8910	63
5	·0872	·0875	11·4301	·9962	**85**	28	·4695	·5317	1·8807	·8829	62
6	·1045	·1051	9·5144	·9945	84	29	·4848	·5543	1·8040	·8746	61
7	·1219	·1228	8·1443	·9925	83	**30**	·5000	·5774	1·7321	·8660	**60**
8	·1392	·1405	7·1154	·9903	82	31	·5150	·6009	1·6643	·8572	59
9	·1564	·1584	6.3138	·9877	81	32	·5299	·6249	1·6003	·8480	58
10	·1736	·1763	5·6713	·9848	**80**	33	·5446	·6494	1·5399	·8387	57
11	·1908	·1944	5·1446	·9816	79	34	·5592	·6745	1·4826	·8290	56
12	·2079	·2126	4·7046	·9781	78	**35**	·5736	·7002	1·4281	·8192	**55**
13	·2250	·2309	4·3315	·9744	77	36	·5878	·7265	1·3764	·8090	54
14	·2419	·2493	4·0108	·9703	76	37	·6018	·7536	1·3270	·7986	53
15	·2588	·2679	3·7321	·9659	**75**	38	·6157	·7813	1·2799	·7880	52
16	·2756	·2867	3.4874	·9613	74	39	·6293	·8098	1·2349	·7771	51
17	·2924	·3057	3·2709	·9563	73	**40**	·6428	·8391	1·1918	·7660	**50**
18	·3090	·3249	3·0777	·9511	72	41	·6561	·8693	1·1504	·7547	49
19	·3256	·3443	2·9042	·9455	71	42	·6691	·9004	1·1106	·7431	48
20	·3420	·3640	2·7475	·9397	**70**	43	·6820	·9325	1.0724	·7314	47
21	·3584	·3839	2·6051	·9336	69	44	·6947	·9657	1·0355	·7193	46
22	·3746	·4040	2·4751	·9272	68	**45**	·7071	1·0000	1·0000	·7071	**45**
							Cos	Cot	Tan	Sine	∠

SECTION 3

APPLIED MECHANICS & BENDING MOMENTS IN BEAMS

DEFINITIONS OF TERMS

Applied Mechanics is the branch of science that treats of the action of forces on engineering structure.

Statics is that branch of Mechanics which treats of the conditions under which a structure is in equilibrium under the action of forces.

Kinetics treats of the relations between forces acting on bodies in motion.

Stress is internal resistance to an external force or load applied. *A unit stress* is the stress produced per unit of area. The three basic stresses are—Tension, Compression and Shear.

Tensile stress is produced when the external forces tend to stretch a body or pull the particles away from one another.

Ultimate tensile stress is the maximum load that a specimen of the material can sustain under tension divided by the original cross sectional area of the specimen.

Tensile strength (per unit area) is the minimum load required to fracture a specimen of a material divided by the original area of the cross section of the material.

Ultimate strength is the maximum load that a specimen of the material can sustain divided by the original cross sectional area of the specimen.

Compressive stress is produced when the forces tend to compress the body or push the particles closer together.

Shearing stress is produced when the forces tend to cause the particles in one section of a body to slide over those of the adjacent section.

Torsion—When a bar is firmly secured at one end and a force is applied to the other end so that the bar tends to twist, the stresses developed in the bar are torsional stresses—the force does not bend the bar, it tends to twist it. The twisting moment is called torque.

Elasticity is the property of a solid material to return to its original size and shape on removal of the force, provided the stress (or the force) has not exceeded a certain limit called *elastic limit*.

Plasticity is the opposite quality of elasticity. A perfectly plastic material is a material which does not return to its original shape when the loading causing deformation is removed. Modelling clay and lead are examples of plastic materials.

Elastic limit is the greatest stress which a material is capable of developing without a permanent deformation remaining upon complete release of the stress and is that stress beyond which the ratio of stress to strain ceases to be constant, (stress is proportional to strain up to a certain limit only) and the deformation caused begins to increase in a faster ratio than the applied loads.(Also explained in

Modulus of Elasticity is a measure of the elastic property of a material and is a ratio between the applied load and the resulting strain, which disappears on removal of the load, and is $= \frac{\text{stress}}{\text{strain}}$ and is generally designated by letter "E". The stress is proportional to the strain (Hook's law) and the modulus of elasticity of steel is constant up to the elastic limit or "yield point". Tensile modulus is called *Young's Modulus* and is the ratio of stress to strain in tension. Modulus of elasticity is required in all calculations involving loading of structures. Modulus of elasticity of steel (in both tension and compression) is 2.1×10^6

Poisson's Ratio: When an elastic material is subjected to an axial stress in the direction of longitudinal axis of the member, the member is deformed not only in the direction of the axial stress but also in the transverse direction. The transverse contraction is proportional to the longitudinal extension and the ratio between the two is called Poisson's ratio and is $= \frac{\text{transverse contraction}}{\text{longitudinal extension}}$

Poisson's ratio for steel is 0.30, and for concrete 0.15 (av.)
Young's modulus, shear modulus and Poisson's ratio are called "Elastic Constants."

Shear modulus or modulus of rigidity $= \frac{\text{shear stress}}{\text{shear strain}}$

Resilience or strain energy is the energy in an elastic material for which it can be repeatedly strained without fracture.

Modulus of Rupture is the maximum unit stress produced in the material when a beam is loaded to failure (See Section 8.)

Rupture stress is the unit stress at the time of failure.

The max. BM is sometimes called the *Moment of Rupture.*

Fatigue is the diminishing resistance to fracture caused by continued application of varying or alternating stresses. see under "Reversal of Stresses."

Yield Point Is the point where the elongation of a bar under tension increases without increase of load and

Yield Stress is the lowest stress in tension at which yield point is reached.

(See Section 5 for detailed descriptions)

Impact. The sudden application of a load to a structure producing stresses in excess of those arising from the static loading.

Static loading is in which loads are applied slowly and they remain stationary.

Dynamic loading is the load applied by the blows of a falling weight.

Load factor. The value by which the load causing failure of the structure to unserviceability is divided to give the permissible working load on the structure.

The *work done* by a force is equal to the product of the force and the distance travelled by its point of application in the direction of the force.

Effective lateral Restraint. Restraint which will produce sufficient resistance in a plane perpendicular to the plane of bending to restrain a loaded beam from buckling to either side at its point of application.

Moment of Inertia (also called Second Moment) of any section is the sum of the products obtained by multiplying the area of each elementary area in the cross section by the square of its centre of gravity distance from the neutral axis, or $I = A\ r^2$

The moment of inertia of a section about an axis other than through its centre of gravity is equal to its moment of inertia about the neutral axis (passing through its centre of gravity) plus the area of section multiplied by the square of the distance between the two axis,

or $I_{XX} = I_{AA} + Ay^2$.

The bending stresses in beams should be calculated on the moment of inertia of the net cross-section of the beam which is the area of the section less deductions for holes for

rivets and bolts etc. In making deductions for rivet and bolt holes, the diameter should be assumed to be 1.5 mm in excess of the nominal diameter.

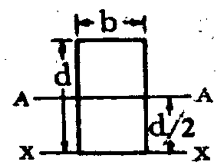
Effective span. For calculating the bending moments in beams, the effective span of a beam should be taken to be the length between the centres of the supports, except where the point of application of the reaction is taken as eccentric to the support, then the effective span may be the length between the assumed points of application of the reactions.

GENERAL FORMULAE FOR THE FLEXURE OF BEAMS

(Flexure means bending stresses or beam strength)

A	=Area of section
I_{AA}	=Moment of Inertia about the neutral axis (passing through the centre of gravity of the section).
I_{XX}	=Moment of Inertia about an axis parallel to the neutral axis—xx.
Z	=Section Modulus.
E	=Young's Modulus of Elasticity.
r	=Radius of Gyration.
f	=Stress in extreme fibres of beams.
R	=Radius of curvature.
y	=Distance of outermost fibre from neutral axis=d/2
B M	=Bending Moment or Moment of Resistance.

$$BM = \frac{fI}{y} = fZ;\quad f = \frac{BMy}{I} = \frac{BM}{Z};\quad Z = \frac{I}{y}$$



$$\frac{BM}{I} = \frac{f}{y} = \frac{E}{R} \qquad I = Ar^2,\ A = bd$$

$$I_{AA} = \frac{bd^3}{12};\quad Z_{AA} = \frac{bd^2}{6};\quad r_{AA} = \sqrt{\frac{I}{A}} = \frac{d}{\sqrt{12}}$$

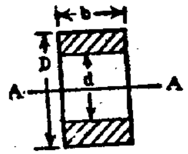
$$= 0.2887d = \frac{d}{3{\cdot}45}$$

$$I_{XX} = I_{AA} + A\left(\frac{d}{2}\right)^2 = \frac{bd^3}{3};\quad Z_{XX} = \frac{bd^2}{3};$$

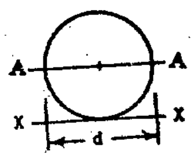
$$r_{xx} = \frac{d}{\sqrt{3}} = 0.5774d$$

When b is the least breadth of section, least $r = \frac{b}{3 \cdot 45}$

The stiffness of a beam depends on its moment of inertia and inversely on its length.



$$I_{AA} = \frac{b\,(D^3 - d^3)}{12}; \quad Z_{AA} = \frac{b\,(D^3 - d^3)}{6D}$$



$$A = \frac{\pi d^2}{4} = .785d^2; \quad I_{AA} = \frac{\pi d^4}{64} = \cdot 0491d^4$$

$$Z_{AA} = \frac{\pi d^3}{32} = .0982d^3: \quad r = \frac{d}{4}$$

$$I_{xx} = \frac{5}{64}\pi d^4$$

In the case of I beams, where the section is not symmetrical about the neutral axis, the maximum compression and tensile stresses are not equal and Z has two values, 'compression modulus of section' Zc, and 'tension modulus of section' Zt, when

$$f_c = \frac{BM}{Z_c} \quad \text{and} \quad Z_c = \frac{I}{y_c}$$

$$f_t = \frac{BM}{Z_t} \quad \text{and} \quad Z_t = \frac{I}{y_t}$$

If the section is symmetrical about the neutral axis, then the section modulus has only one value and $f_c = f_t = BM/Z$.

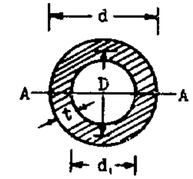
For R.S. Joists, the minimum radius of gyration is about 0.21 of the breadth of the joist and the maximum radius of gyration about 0.42 of the depth of the joist.

Solid semi-circle or half round (resting on flat bottom).

Neutral axis from bottom=0.4244 R

$$I_{AA} = 0.1098R^4; \quad Z_{AA} = 0.1907R^3; \quad r_{AA} = 0.2643R$$

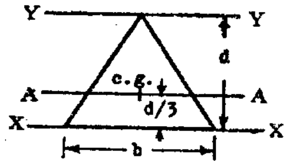
$A = \frac{\pi R^2}{2}$ R is radius of the semi-circle.



$$A = \frac{\pi\,(d^2 - d_1^2)}{4} = .785\,(d^2 - d_1^2)$$

$$I_{AA} = \frac{\pi\,(d^4 - d_1^4)}{64} = .0491\,(d^4 - d_1^4)$$

$$Z_{AA} = \frac{\pi\,(d^4 - d_1^4)}{32d}; \quad r_{AA} = \frac{\sqrt{(d^2 + d_1^2)}}{4}$$



$$I_{AA} = \frac{bd^3}{36}; \quad Z_{AA} = \frac{bd^2}{24};$$

$$r_{AA} = \frac{d}{\sqrt{18}} = 0.236d$$

$$I_{xx} = \frac{bd^3}{12}; \quad Z_{xx} = \frac{bd^2}{12}; \quad r_{xx} = \frac{d}{\sqrt{6}} = 0.4083d$$

$$I_{YY} = \frac{bd^3}{4}; \quad Z_{YY} = \frac{bd^2}{4}; \quad r_{YY} = \frac{d}{\sqrt{2}} = 0.707d$$

Thin Tubes, Cylinders or Shells of thickness "t"

$$A = \pi Dt; \quad I = \frac{\pi D^3 t}{8}; \quad Z = .70D^2 t; \quad r = 0.35D$$

Hoop stress in the material of the cylinder $= \frac{pd}{2t}$; Longitudinal stress due to the pressure on the ends of the cylinder which is also the stress in the material of a shell $= \frac{pd}{4t}$; p=internal intensity of fluid pressure.

(A cylinder may be regarded as thin when thickness of walls is less than 1/48 of diameter.)

ECONOMIC SECTIONS OF BEAMS

For equal depths and weights per foot the comparative strength of the various steel sections used as beams is as follows:—

R.S.J I	Channel [	Unequal L	Equal L	Equal T	Unequal T
100%	90%	50%	40%	40%	20%

Therefore, it will be seen that I section is the most economical and should be used as far as possible.

Sections are generally rolled from 12 m to 18 m lengths.

Relative Strengths of Beams

Kind and position of load	Strength ratio
(*a*) *Beam supported at both ends* :	
Uniformly distributed over entire span	1.0
Concentrated at middle of span	0.5
(*b*) *Beams Fixed at both ends*:	
Uniformly distributed over entire span ..	1.5
Concentrated at middle of span ..	1.0
(*c*) *Cantilever beams*:	
Uniformly distributed over entire span	0.25
Concentrated at the free end	0.125
(*d*) *Cantilever beam supported at the free end* :	
Uniformly distributed over entire span	1.0
Concentrated near the middle of span ..	0.65

Design of Simple Beams

The common procedure in designing a beam is to calculate its maximum bending moment, section modulus and shear. Make use of the Tables given at page 4/18 and pages following. First determine its size in accordance with the strength in bending and then investigate the beam selected for shear and deflection. The allowable loads on beams may be limited by the shearing and buckling strength of the web, instead of by the maximum bending stress, although usually it is the bending moment or the deflection which settles the size of a joist and only occasionally the shear when the beam is of a short span and is heavily loaded or where the beam is purposely kept shallow to give head-room. It is quite possible for a beam to be perfectly capable of carrying its design load and at the same time to have undesirable deflection.

Limiting Depths of Beams for Deflection

The calculated deflection of any beam should not be greater than 1/325 of the span, which limits the depth to span ratio to 1 to 24. Where, however, the deflection of a beam is not of much importance, this limit may be exceeded. Depths of (simply supported at ends) girders and rolled beams in floors should not be less than 1/24 of the span. In floors subject to shocks and vibrations the depths should be limited to 1/16 of the span. In floors with ceilings, the depth to span ratio is 18. For heavy beam work, generally the ratio is 1/15. In order not to exceed the deflection of 1/480 of the span, the proportion of length to depth of girder or joist should not be greater than 20 to 1 for uniformly distributed load and 13 to 1 for the same load concentrated at the centre.

If the beam depth is not shallower than span/19, then the deflection need not be calculated, since it will not exceed span/325. Deflection should be measured at mid-span of beams and at the ends of cantilevers.

An increase in the modulus of elasticity has the effect of reducing the deflection by a corresponding proportion. If the breadth of the beam is doubled, the depth remaining the same, the value of I is doubled. If the depth is doubled, breadth remaining constant, the value of I rises to eight times its original value. The deflection is calculated on the gross moment of inertia.

For a greater depth than 30 to 40 cm, a built-up girder is usually preferable and more economical under heavy loads.

Camber is hardly worth considering unless the girder is over 25 or 30 metres span. The actual deflection can be used for the camber. Empirical rules are sometimes used, such as 1.5 mm for each 3 metres of span. Crane girders of 23 m or greater span should generally be cambered for approximately the dead and half live load deflection. Specified camber for rolled beams over 380 mm in depth, shall be only that offered as cold cambering at the mill.

Lateral Stability of Beams

In all beams without continous lateral support there is a tendency for the compression flange (upper flange) .to buckle at right angles to the plane of bending in the mid-span section of the beam as a result of which the mid-span section is displaced relative to the ends. This tendency of lateral instability of a beam without conti- nous lateral support is the controlling factor in design. The principal factor governing the elastic stability (against buckling) of a beam or a column is the "slenderness ratio" l/r, where l is the distance between lateral supports (or effective length) and r is the least radius of gyration of the beam. To safe-guard against this buckling the per- missible bending stress in compression is reduced where the ratio l/r exceeds 100.

The maximum allowable ratio of span to flange width is limited to 40. (Span is the distance between lateral supports.) The tabulated loads shown at page 4/19 to 22 and 4/24 should be reduced in value as per following table:—

Span/Flange width	20	22	24	26	28	30	32	34	36	38	40
Reduction factor	1.00	.82	.78	.75	.72	.69	.66	.63	.60	.57	.55

Web Stiffeners should be provided where the thick- ness of the web is less than $d/85$, where d is the depth of the web. Stiffeners should be provided at all concentra- ted load points and at ends as a precaution against buckl- ing of the web where there are high compressive stresses. (See under "Plate Girders" in Section 19.)

The average shear stress on the gross section of the web shall not exceed 945 kg/sq. cm for unstiffened webs. The gross section of the web is taken as:

(i) For rolled I-beams and channels—The depth of the beam multiplied by the web thickness.

(ii) For plate girders—The depth of the web plate multiplied by its thickness.

It will be seen that there is no advantage in using beam section where the permissible stress is considerably reduced. Where a beam is laterally supported conti- nuously throughout its length, a deep narrow section is most economical, but where the compression flange is laterally unsupported, a wider shallow section is prefer- able. A continous load over the top flange over an I- beam such as from a floor, or cross beams fixed at or near the top flange constitute a lateral support. Cross-members attached to the lower part of an I-beam do not constitute adequate lateral support. (Supports which give effective lateral restraint are adequate lateral supports.)

Stresses due to temperature changes. Total expansion of a structure due to temperature change = length of the structure × change of temperature × co- efficient of expansion of the structure metal. If the structure is rigidly fixed so that the expansion due to temperature change cannot take place, the compressive stress developed in the structure = total (stopped) expan- sion of the structure × modulus of elasticity of the struc- ture metal, in kg/sq. cm.

Co-efficient of linear expansion of steel is taken = 0.000012 per deg. C. (or 0.0000067 per deg. F.)

BENDING MOMENT & DEFLECTION IN BEAMS

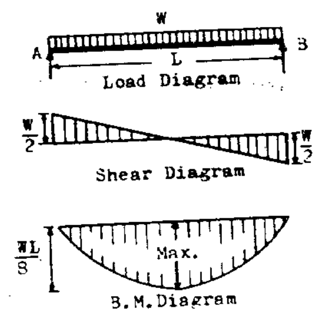
Under Various Systems of Loading

W = total load in kg; w = load per cm run in kg;
L = span in cm; BM = bending moment in cm kg;
E = modulus of elasticity of steel (21,00,000 kg/sq. cm)
R_A = reaction at A, R_B = reaction at B.
Deflection is in cm.

$$R_A = R_B = \frac{W}{2}$$

$$\text{Max: BM} = \frac{WL}{8}$$

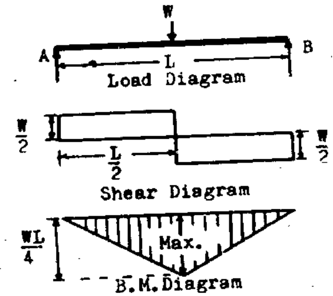
$$\text{Max: Def:} = \frac{5WL^3}{384\ EI}$$



$$R_A = R_B = \frac{W}{2}$$

$$\text{Max: B M} = \frac{WL}{4}$$

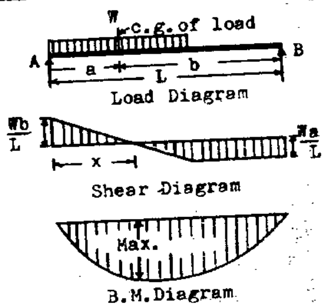
$$\text{Max: Def.} = \frac{WL^3}{48\ EI}$$



$$R_A = \frac{W.b}{L},\quad R_B = \frac{W.a}{L}$$

$$\text{Max: BM} = \frac{R_A.x}{2}$$

$$x = \frac{R_A}{w}$$

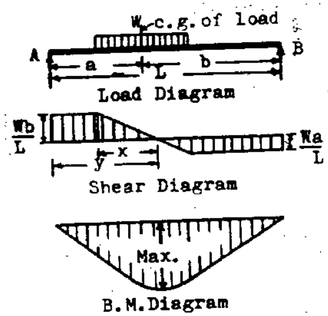


$$R_A = \frac{W.b}{L},\ R_B = \frac{W.a}{L}$$

$$x = \frac{R_A}{w}$$

$$\text{Max: BM} = R_A.y - \frac{w.x^2}{2}$$

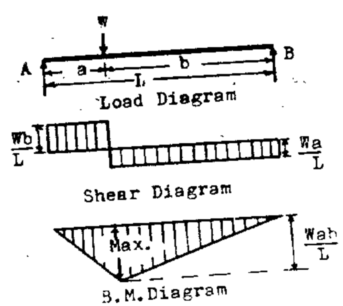
Def: varies with position of load



$$R_A = \frac{W.b}{L},\ R_B = \frac{W.a}{L}$$

$$\text{Max: BM} = \frac{W.a.b}{L}$$

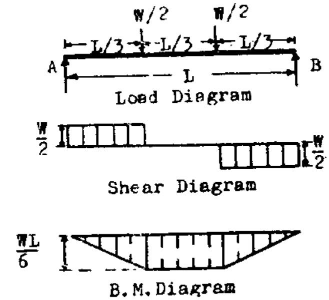
$$\text{Max: Def.} = \frac{W.a.b\ (2L-b)}{9EIL} \times \sqrt{\frac{b}{3}(2L-b)} \qquad b \geq a$$



$$R_A = R_B = \frac{W}{2}$$

$$\text{Max: BM} = \frac{WL}{6}$$

$$\text{Max. Def.} = \frac{6.82W}{384\ EI}$$



(*b*) If both the loads are placed L/4 instead of L/3 from each end:

$$\text{Max : BM} = \frac{WL}{8}$$

$$\text{Max : Def :} = \frac{5.5WL^3}{384\ EI}$$

(c) *Unsymmetrical Concentrated Loads:*

Suppose loads are W_1, W_2, W_3 placed at distances of a, b, c from end A on a span L:

R_B (taking moments about A)

$$= \frac{(W_1 \times a) + (W_2 \times b) + (W_3 \times c)}{L}$$

$$R_A = (W_1 + W_2 + W_3) - R_B$$

The maximum BM occurs at the point of application of one of the loads and may be found as follows:—

BM under $W_1 = R_A \times a$

" " $W_2 = R_A \times b - [W_1 \times (b-a)]$

" " $W_3 = R_A \times c - [W_1 \times (c-a) + W_2(c-b)]$

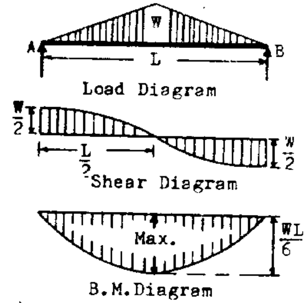
or $= R_B \times (L-c)$

Approximate BM for an unsymmetrically loaded beam can be found by working out the position of the centre of gravity of all the loads and considering the total of all the loads acting at the centre of gravity point.

$$R_A = R_B = \frac{W}{2}$$

$$\text{Max : BM} = \frac{W\,L}{6}$$

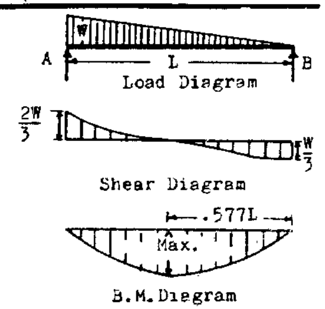
$$\text{Max : Def :} = \frac{W\,L^3}{60\,E\,I}$$



$$R_A = \frac{2}{3}W, \quad R_B = \frac{1}{3}W$$

$$\text{Max : BM} = \frac{W\,L}{7.8}$$

$$\text{Max : Def :} = \frac{5}{384}\frac{W\,L^3}{E\,I}$$

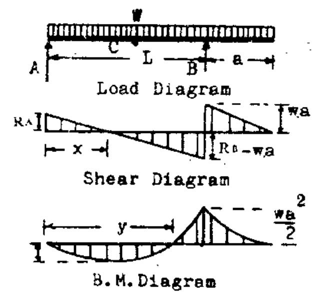


$$R_A = W\left[\left(\frac{L+a}{2}\right) - a\right] \times \frac{1}{L} \quad ; \quad R_B = W\left(\frac{L+a}{2}\right) \times \frac{1}{L}$$

$$x = \frac{R_A}{w}, \quad y = \frac{2R_A}{w}$$

$$\text{B M at C} = +\frac{R_A^2}{2w}$$

$$\text{B M at B} = -\frac{w.a^2}{2}$$

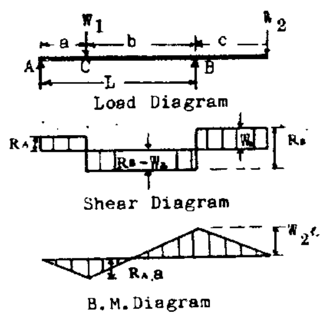


$$R_A = \frac{W_1.b - W_2.c}{L}$$

$$R_B = \frac{W_1.a + W_2(L+c)}{L}$$

$$\text{Max : BM at C} = +R_A.a$$

$$\text{Max : BM at B} = -W_2.c$$



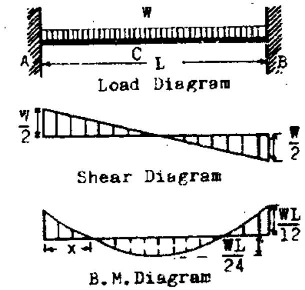
$$R_A = R_B = \frac{W}{2}$$

$$\text{BM at C} = +\frac{W\,L}{24}$$

$$\text{BM at A or B} = -\frac{W\,L}{12}$$

$x = 0.211L$

$$\text{Max : Def :} = \frac{WL^3}{384\,EI}$$

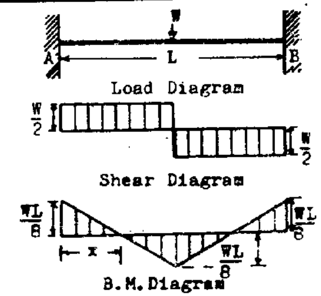


$$R_A = R_B = \frac{W}{2}$$

$$\text{Max : BM} = \pm\frac{WL}{8}$$

$x = 0.25L$

$$\text{Max : Def :} = \frac{WL^3}{192\,EI}$$



$$R_A = \frac{W.b^2(3a+b)}{L^3}$$

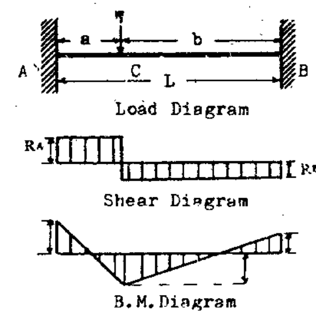
$$R_B = \frac{W.a^2(3b+a)}{L^3}$$

$$\text{BM at C} = R_A \times a - \frac{W.a.b^2}{L^2}$$

$$\text{or} = \frac{2W.a^2.b^2}{L^3}$$

$$\text{BM at A} = -\frac{W.a.b^2}{L^2}$$

$$\text{BM at B} = \frac{W.a^2.b}{L^2} \qquad \text{Max: Def :} = \frac{2W.a^3.b^2}{3\,EI\,(3a+b)^2}$$



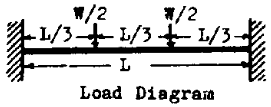
If $a = \frac{2}{3}L$, $b = \frac{1}{3}L$

Reactions		BM				Max. Def.	Point of contraflexure
R_A	R_B	Max.	at A	at B	at C		
$\frac{7W}{27}$	$\frac{20W}{27}$	$\frac{WL}{6.75}$	$\frac{WL}{13.5}$	$\frac{WL}{6.75}$	$\frac{WL}{10.1}$	$\frac{1.55WL}{384\,EI}$	0.29 L from A 0.67 L from B

If $a = \frac{3}{4}L,\ b = \frac{1}{4}L$

$\frac{5W}{32}$	$\frac{27W}{32}$	$\frac{WL}{7.1}$	$\frac{WL}{21.3}$	$\frac{WL}{7.1}$	$\frac{WL}{14.2}$	$\frac{1.08WL^3}{384\,EI}$	0.31 L from A 0.17 L from B

1.(*a*) Loads W/2 placed at L/3:—



Reaction at ends	Max : BM at ends	BM at centre	Max: Def.:	Points of contraflexure from ends
$\frac{W}{2}$	$-\frac{WL}{9}$	$+\frac{WL}{18}$	$\frac{1.48\,WL^3}{384\,EI}$	0.22 L

(*b*) Loads placed at a distance of '*a*' from each end instead of L/3 :—

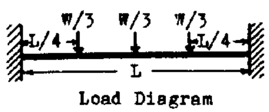
$\frac{W}{2}$	$-\frac{W.a(L-a)}{2L}$	$+\frac{W.a^2}{2L}$	$\frac{8W.a^2(3L-4a)}{384\,EI}$	$\frac{(L-a)a}{L}$

(*c*) Loads placed at L/4 from ends instead of L/3:—

$\frac{W}{2}$	$-\frac{3WL}{32}$	$+\frac{WL}{32}$	$\frac{WL^3}{384\,EI}$	0.19 L

2. Loads W/3 placed at L/4:—

Max : BM at ends $= -\frac{5WL}{48}$



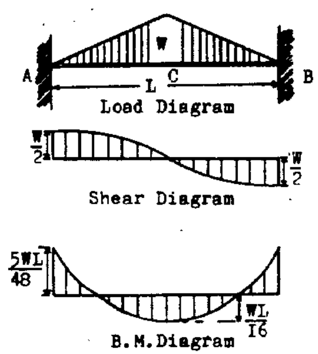
$R_A = R_B = \frac{W}{2}$

BM at A or B $= -\frac{5}{48}WL$

BM at C $= +\frac{1}{16}WL$

Max : Def : $= \frac{1 \cdot 4WL^3}{384EI}$

Point of contraflexure = 0.22L from A or B.



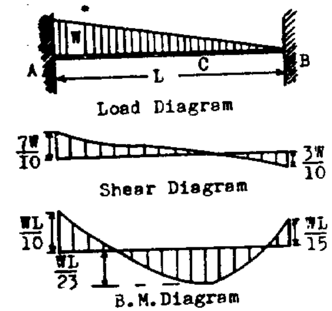
$R_A = \frac{7}{10}W, \quad R_B = \frac{3}{10}W$

BM at A $= -\frac{WL}{10}$

BM at B $= -\frac{WL}{15}$

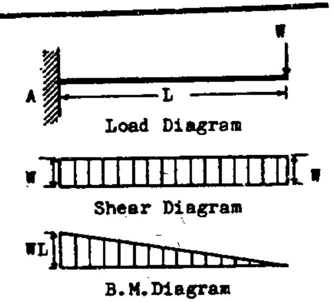
BM at C $= +\frac{WL}{23}$

Max : Def : $= \frac{WL^3}{384\,EI}$



BM at A = WL

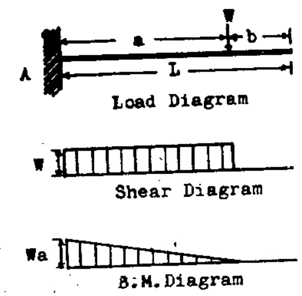
Max : Def $= \frac{WL^3}{3\,EI}$



BM at A = W.a

Max : Def:

$= \frac{W}{6EI} \times (3a^2L - a^3)$

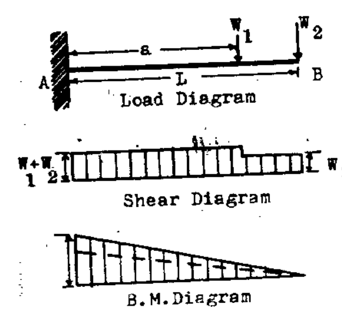


$R_A = W_1 + W_2$

Max: BM at A $= W_1.a + W_2.L$

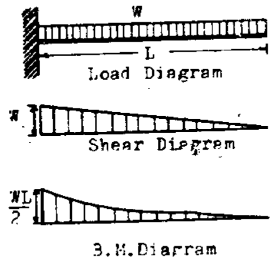
Max : Def :

$= \frac{W_1.a^2(3L-a) + 2W_2.L^3}{6\,EI}$



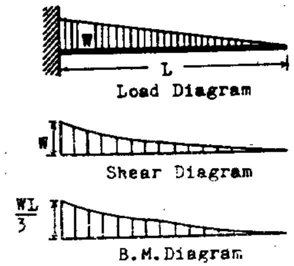
Max : $BM = \frac{WL}{2}$

Max : Def $:= \frac{WL^3}{8\ EI}$



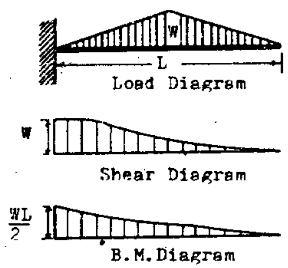
Max : $BM = \frac{WL}{3}$

Max : Def $:= \frac{WL^3}{15\ EI}$



Max: $BM = \frac{WL}{2}$

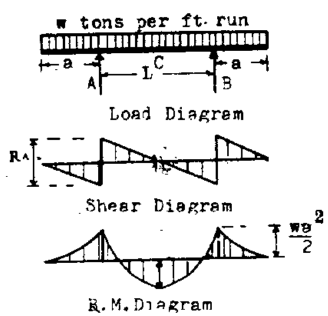
Max : Def $:= \frac{11\ WL^3}{96\ EI}$



$R_A = R_B = w.a + \frac{w.L}{2}$

BM at A or B $= -\frac{w.a^2}{2}$

BM at C $= +\left(\frac{w.L^2}{8} - \frac{w.a^2}{2}\right)$



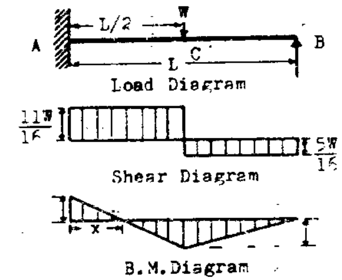
$R_A = \frac{11}{16}W, \quad R_B = \frac{5}{16}W$

$x = \frac{3}{11}L$

B M at A $= -\frac{3}{16}WL$

BM at C $= +\frac{5}{32}WL$

Max: Def: $= \frac{3.58\ WL^3}{384EI}$



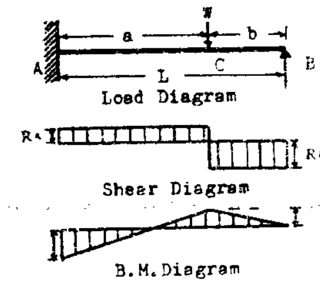
$R_A = \frac{W.b\ (3L^2 - b^2)}{2L^3}$

$R_B = \frac{W.a^2\ (3L - a)}{2L^3}$

B M at A $= -\frac{W.a.b\ (2L - a)}{2L^2}$

B M at C $= +\frac{W.a^2.b\ (3L - a)}{2L^3}$

Max: Def: $= \frac{W.a^2 b}{6\ EI}\sqrt{\frac{b}{3L - a}}$



If $a = \frac{2}{3}L$, $b = \frac{1}{3}L$ (Last Diagram)

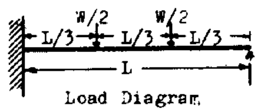
Reactions R_A	Reactions R_B	B M Max:	B M at A	B M at C	Max: Def:	Point of Contraflexure from A
$\frac{13W}{27}$	$\frac{14W}{27}$	$\frac{WL}{5.78}$	$\frac{WL}{6.75}$	$\frac{WL}{5.78}$	$\frac{3.58\ WL^3}{384\ EI}$	0.31 L
If $a = \frac{1}{3}L$, $b = \frac{2}{3}L$						
$\frac{23W}{27}$	$\frac{4W}{27}$	$\frac{WL}{5.405}$	$\frac{WL}{5.405}$	$\frac{WL}{10.12}$	$\frac{2.37\ WL^3}{384\ EI}$	0.21 L
If $a = \frac{3}{4}L$, $b = \frac{1}{4}L$						
$\frac{47W}{128}$	$\frac{81W}{128}$	$\frac{W\ L}{6.305}$	$\frac{W\ L}{8.5}$	$\frac{W\ L}{6.305}$	$\frac{3.06W\ L^3}{384\ E\ I}$	0.32 L

If $a = \frac{1}{4}L$, $b = \frac{3}{4}L$

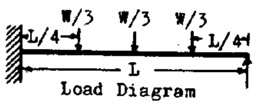
$\frac{117W}{128}$	$\frac{11W}{128}$	$\frac{WL}{6.10}$	$\frac{WL}{6.10}$	$\frac{WL}{15.5}$	$\frac{1.57\,WL^3}{384\,EI}$	0.186 L

$R_A = \frac{2}{3}W$, $R_B = \frac{1}{3}W$

Max: $BM = -\frac{WL}{6}$



Max : $BM = -\frac{5}{32}WL$

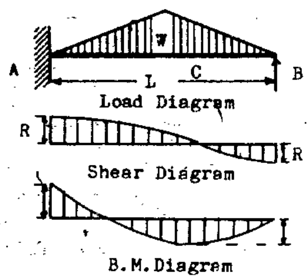


$R_A = \frac{21}{32}W$, $R_B = \frac{11}{32}W$

BM at $A = -\frac{5}{32}WL$

BM at $C = +\frac{17}{192}WL$

Max: Def: $= \frac{WL^3}{139.5\,EI}$

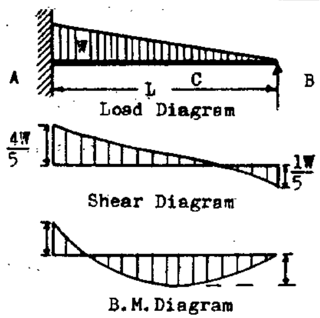


$R_A = \frac{4}{5}W$, $R_B = \frac{1}{5}W$

BM at $A = -\frac{2}{15}WL$

BM at $C = +\frac{1}{16.7}WL$

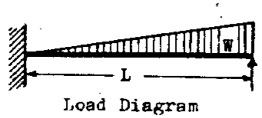
Max: Def : $= \frac{WL^3}{210\,EI}$



Max : BM at fixed end $= -\frac{7}{60}WL$

If there is no support at the end:

Max : $BM = -\frac{2}{3}WL$



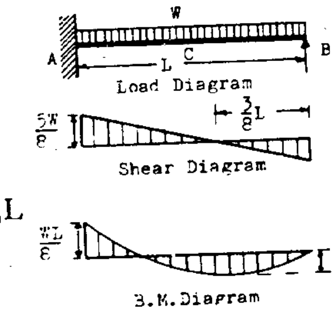
$R_A = \frac{5}{8}W$, $R_B = \frac{3}{8}W$

BM at $A = -\frac{WL}{8}$

BM at $C = +\frac{9}{128}WL$

Point of contraflexure is at 0.25L from A.

Max: Def: $= \frac{2.08\,WL^3}{384\,EI}$



(*a*) Uniformly distributed load over two equal spans:

$R_A = R_B = \frac{3}{8}w.L$

$R_C = \frac{5}{8}w.L + \frac{5}{8}w.L$

BM at $C = -\frac{w.L^2}{8}$

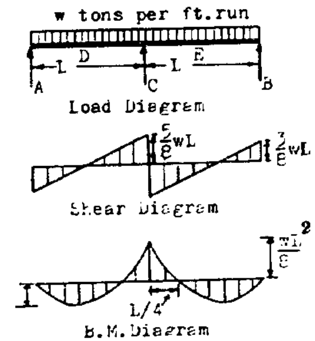
BM at D or $E = +\frac{9}{128} \times w.L^2$

(*ii*) Assuming only one span (AC) loaded:

$R_A = +\frac{7}{16}w.L$, $R_B = -\frac{1}{16}w.L$

Max: BM at 0.438 L from $A = +\frac{12}{125}w.L^2$

BM at $C = \frac{1}{16}w.L^2$



(*b*) Concentrated load W at centre of each span:

$R_A = R_B = \frac{5}{16}W$, $R_C = \frac{11}{8}W$

BM at D or $E = +\frac{5}{32}WL$

BM at $C = -\frac{3}{16}WL$

(*ii*) Assuming only one span (AC) loaded:

$R_A = +0.406\,W$, $R_B = -0.094\,W$

Max: BM under the load (at centre of span AC) $= +0.203\,WL$

BM at $C = -0.094\,WL$

Bending Moments for Inclined Loads:

The general rule is to resolve all forces, including the reactions, along and perpendicular to the beam. Thrust is along (parallel to) the beam. BM and shear are then worked out. *Thrust* at any point of a beam is the sum of the components in the direction of the beam of all the forces to the right of it. If the thrust is negative it becomes a *pull.*

For a sloping beam with vertical loads, (or reactions) the BM is the same as for an horizontal beam of the same span as the horizontally projected length of the sloping beam.

Moving Loads:

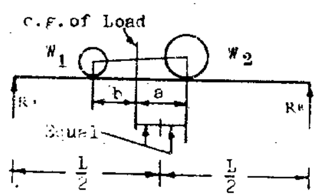
(1) *Beam supported at both ends and having a uniformly distributed moving load of length greater than the span:*

Maximum shear occurs at the ends when the load is over the whole span.

Maximum BM occurs at the centre when the load is over the whole span.

(2) *Beam supported at both ends and having two unequal concentrated moving loads:*

Maximum shear at the left hand support occurs when the load W_1 is over the left hand support. The shear is:



$$R_A = W_1 + \frac{W_2[L-(a+b)]}{L}$$

Maximum shear at the right hand support occurs when the load W_2 is over the right hand support. The shear is:

$$R_B = W_2 + \frac{W_1[L-(a+b)]}{L}$$

Should the distance $(a+b)$ between the loads be greater than 0.59 L, consider the shear due to one load only.

Maximum BM on the beam occurs under the heavier load when the position of the loads is such that the centre of the beam is half-way between the centre of gravity of the loads and the heavier load:

The maximum BMs are:

$$\text{under } W_2 = \frac{W_1+W_2}{4L}(L-a)^2$$

$$\text{under } W_1 = \frac{W_1+W_2}{4L}(L-b)^2$$

Curved Beams

If the axis of a beam is curved the neutral axis no longer passes through the centre of gravity of a cross-section but is shifted towards concave side of the beam. The maximum unit stress is increased by the curvature of the beam. The following formula may be used to find approximate stress in a curved beam: (Derived from Winkler Bach formula)

$$K = 1.0 + 0.5\frac{I}{bc^2}\left(\frac{1}{(R-c)} + \frac{1}{R}\right)$$

K is a correction factor by which the stress determined by the ordinary "straight beam" formula $\left(f=\frac{M}{Z}\right)$ is to be multiplied to give the extreme unit stress for the concave side of the curved beam,

I = moment of inertia of cross-section,

c = distance from the axis through the centre of gravity of cross-section to the extreme fibre on the concave side of the curved beam,

b = max. breadth of the cross-section.

R = radius of curvature of the centroidal line of the beam.

Effective Length of Compression Flange. For simply supported beams and girders where no lateral restraint of the compression flange is provided but where each end of the beam is restrained against torsion, the effective length of the compression flange shall be taken as follows:

(a) With ends of compression flanges unrestrained against lateral bending l = span

(b) With ends of compression flange partially restrained against lateral bending $l = 0.85 \times$ span

(c) With ends of compression flange fully restrained against lateral bending $l = 0.7 \times$ span

Restraint against torsion is provided by beams being built into walls.

Where the ends of the beam are not restrained against torsion, or where the load is applied to the compression flange and both the load and the flange are free to move laterally, the above values of the effective length shall be increased by 20 per cent.

For cantilever beams of projecting length L, the effective length to be used shall be taken as follows:

(a) Built-in at the supports, free at the end $l = 0.85$ L

(b) Built-in at the supports, restrained against torsion at the end by continuous construction $l = 0.75$ L

(c) Built-in at the support restrained against lateral deflection and torsion at the end $l = 0.50$ L

(d) Continuous at the support, unrestrained against torsion at the support and free at the end $l = 3$ L

(e) Continuous at the support, with partial restraint against torsion at the support and free at the end $l = 2$ L

(f) Continuous at the support, restrained against torsion at the support and free at the end $l =$ L

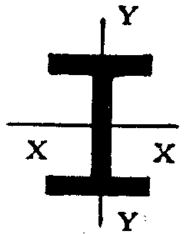
The ratio of the effective length of the compression flange to the appropriate radius of gyration shall not exceed 300.

Where the I-beam is encased in cement concrete its lateral stability is increased. (See under "Cased Beams")

If a concrete slab encases the top flange of the beam so that the bottom surface of the slab is flush with the bottom of the top flange of the beam, the beam may be considered as fully supported laterally.

PROPERTIES OF STRUCTURAL STEEL SECTIONS & DESIGN TABLES (Indian Standards)

(The Tables have been based generally on Indian Standards Institute "Handbook for Structural Engineers".)



INDIAN STANDARD STEEL BEAMS (R.S. Joists)

Dimensions and Properties

Size: Depth × Width	Weight per Metre Length	Sectional Area	Moment of Inertia About		Radii of Gyration About		Section Moduli About	
			X—X	Y—Y	X—X	Y—Y	X—X	Y—Y
mm	kg	cm^2	cm^4	cm^4	cm	cm	cm^3	cm^3
Junior Beams (ISJB Series)								
150× 50	7.1	9.01	322.1	9.2	5.98	1.01	42 .9	3.7
175× 50	8.1	10.28	479.3	9.7	6.83	0.97	54.8	3.9
200× 60	9.9	12.64	780.7	17.3	7.86	1.17	78.1	5.8
225× 80	12.8	16.28	1308.5	40.5	8.97	1.58	116.3	10.1
Light Weight Beams (ISLB Series)								
75× 50	6.1	7.71	72.7	10.0	3.07	1.14	19.4	4.0
100× 50	8.0	10.21	168.0	12 .7	4.06	1.12	33.6	5.1
125× 75	11.9	15.12	406.8	43.4	5.19	1.69	65.1	11.6
150× 80	14.2	18.08	688.2	55.2	6·17	1.75	91.8	13.8
175× 90	16.7	21.30	1096.2	79.6	7.17	1.93	125.3	17.7
200×100	19.8	25.27	1696.6	115.4	8.19	2.13	169.7	23.1
225×100	23.5	29.92	2501.9	112.7	9.15	1.94	222.4	22.5
250×125	27.9	35.53	3717.8	193.4	10.23	2.33	297.4	30.9
275×140	33.0	42.02	5375.3	287.0	11.31	2.61	392.4	41.0
300×150	37.7	48.08	7332.9	376.2	12.35	2.80	488.9	50.2
325×165	43.1	54.90	9874.6	510.8	13.41	3.05	607.7	61·9
350×165	49.5	63.01	13158.3	631.9	14.45	3.17	751.9	76.6
400×165	56.9	72.43	19306.3	716.4	16·33	3.15	965.3	86.8
450×170	65.3	83.14	27536.1	853.0	18.20	3.20	1223.8	100.4
500×180	75.0	95.50	38579.0	1063.9	20.10	3.34	1543.2	118.2
550×190	86.3	109.97	53161.6	1335.1	21.99	3.48	1933.2	140.5
600×210	99.5	126·69	72867.6	1821.9	23.98	3.79	2428.9	173.5
Medium Weight Beams (ISMB Series)								
100× 75	11.5	14.60	257.5	40.8	4.20	1.67	51.5	10.9
125× 75	13.0	16.60	449.0	43.7	5.20	1.62	71.8	11.7
150× 80	14.9	19.00	726.4	52.6	6.18	1.66	96.9	13.1
175× 90	19.3	24.62	1272.0	85·0	7.19	1.86	145.4	18.9
200×100	25.4	32.33	2235.4	150.0	8.32	2.15	223.5	30.0
225×110	31.2	39.72	3441.8	218.3	9.31	2.34	305.9	39.7
250×125	37.3	47.55	5131.6	334.5	10.39	2.65	410.5	53.5

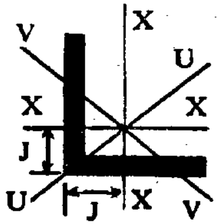
INDIAN STANDARD STEEL BEAMS (Contd.)

Dimensions and Properties

Size: Depth × Width	Weight per Metre Length	Sectional Area	Moment of Inertia About		Radii of Gyration About		Section Moduli About	
			X—X	Y—Y	X—X	Y—Y	X—X	Y—Y
mm	kg	cm^2	cm^4	cm^4	cm	cm	cm^3	cm^3
300×140	44.2	56.26	8603.6	453·9	12.37	2.84	573.6	64.8
350×140	52.4	66.71	13630.3	537.7	14.29	2.84	778.9	76.8
400×140	61.6	78.46	20458.4	622.1	16.15	2.82	1022.9	88.9
450×150	72.4	92.27	30390.8	834.0	18.15	3.01	1350.7	111.2
500×180	86.9	110.74	45218.3	1369.8	20.21	3.52	1808.7	152.2
550×190	103.7	132.11	64893.6	1833.8	22.16	3.73	2359.8	193.0
600×210	122 .6	156.21	91813.0	2651.0	24.24	4.12	3060.4	252.5
Wide Flange Beams (ISWB Series)								
150×100	17.0	21.67	839.1	94.8	6.22	2.09	111.9	19.0
175×125	22.1	28.11	1509.4	188.6	7.33	2.59	172.5	30.2
200×140	28.8	36·71	2624.5	328.8	8.46	2.99	262.5	47.0
225×150	33.9	43.24	3920.5	448.6	9.52	3.22	348.5	59.8
250×200	40.9	52.05	5943.1	857.5	10.69	4.06	475.4	85.7
300×200	48.1	61.33	9821.6	990.1	12·66	4.02	654.8	99.0
350×200	56.9	72.50	15521.7	1175.9	14.63	4.03	887.0	117·6
400×200	66·7	85.01	23426·7	1388.0	16.60	4.04	1171.3	138.8
450×200	79.4	101.15	35057.6	1706.7	18.63	4.11	1558.1	170.7
500×250	95.2	121.22	52290.9	2987.8	20.77	4.96	2091.6	239.0
550×250	112.5	143.34	74906.1	3740.6	22 .86	5.11	2723.9	299.2
600×250	133.7	170.38	106198.5	4702.5	24·97	5.25	3540.0	376.2
600×250	145.1	184.86	115626.6	5298.3	25.01	5.35	3854.2	423.9
Column Sections— H-Beams (ISHB Series)								
150×150	27.1	34.48	1455.6	431.7	6.50	3.54	194.1	57.6
150×150	30.6	38.98	1540.0	460.3	6.29	3.44	205.3	60.2
150×150	34.6	44.08	1635.6	494.9	6.09	3.35	218.1	63·2
200×200	37.3	47.54	3608.4	967.1	8.71	4.51	360·8	96.7
200×200	40.0	50.94	3721.8	994.6	8.55	4.42	372 .2	98.6
225×225	43.1	54.94	5279.5	1353.8	9.80	4.96	469.3	120.3
225×225	46.8	59.66	5478.8	1396.6	9.58	4.84	487.0	123.0
250×250	51.0	64.96	7736.5	1961.3	10.91	5.49	618.9	156.9
250×250	54.7	69.71	7983.9	2011.7	10.70	5·37	638.7	159.7
300×250	58.8	74.85	12545.2	2193.6	12.95	5.41	836.3	175.5
300×250	63.0	80.25	12950.2	2246.7	12.70	5:29	863.3	178.4
350×250	67.4	85·91	19159.7	2451.4	14.93	5.34	1094.8	196.1
350×250	72.4	92.21	19802.8	2510.5	14.65	5.22	1131.6	199.4
400×250	77.4	98.66	28083.5	2728.3	16.87	5.26	1404.2	218.3
400×250	82.2	104.66	28823.5	2783·0	16.61	5.16	1444·2	221.3
450×250	87.2	111.14	39210.8	2985.2	18.78	5.18	1742 .7	238.8
450×250	92.5	117·89	40349.9	3045.0	18.50	5.08	1793.3	242.1

INDIAN STANDARD EQUAL ANGLES

Dimensions and Properties



Size	Thickness (t)	Weight per Metre Length	Sectional Area	Centre of Gravity Dimension J	Moment of Inertia About X—X	Radii of Gyration About X—X	Radii of Gyration About U—U (max.)	Radii of Gyration About V—V (min.)	Section Modulus About X—X (min.)
mm	mm	kg	cm²	cm	cm⁴	cm	cm	cm	cm³
20×20	3.0	0.9	1.12	0.59	0.4	0.58	0.73	0.37	0.3
	4.0	1.1	1.45	0.63	0.5	0.58	0.72	0.37	0.4
25×25	3.0	1.1	1.41	0.71	0.8	0.73	0.93	0.47	0.4
	4.0	1.4	1.84	0.75	1.0	0.73	0.91	0.47	0.6
	5.0	1.8	2.25	0.79	1.2	0.72	0.91	0.47	0.7
30×30	3.0	1.4	1.73	0.83	1.4	0.89	1.13	0.57	0.6
	4.0	1.8	2.26	0.87	1.8	0.89	1.12	0.57	0.8
	5.0	2.2	2.77	0.91	2.1	0.88	1.11	0.57	1.0
35×35	3.0	1.6	2.03	0.95	2.3	1.05	1.33	0.67	0.9
	4.0	2.1	2.66	1.00	2.9	1.05	1.32	0.67	1.2
	5.0	2.6	3.27	1.04	3.5	1.04	1.31	0.67	1.4
	6.0	3.0	3.86	1.08	4.1	1.03	1.29	0.67	1.7
40×40	3.0	1.8	2.34	1.08	3.4	1.21	1.54	0.77	1.2
	4.0	2.4	3.07	1.12	4.5	1.21	1.53	0.77	1.6
	5.0	3.0	3.78	1.16	5.4	1.20	1.51	0.77	1.9
	6.0	3.5	4.47	1.20	6.3	1.19	1.50	0.77	2.3
45×45	3.0	2.1	2.64	1.20	5.0	1.38	1.74	0.87	1.5
	4.0	2.7	3.47	1.25	6.5	1.37	1.73	0.87	2.0
	5.0	3.4	4.28	1.29	7.9	1.36	1.72	0.87	2.5
	6.0	4.0	5.07	1.33	9.2	1.35	1.70	0.87	2.9
50×50	3.0	2.3	2.95	1.32	6.9	1.53	1.94	0.97	1.9
	4.0	3.0	3.88	1.37	9.1	1.53	1.93	0.97	2.5
	5.0	3.8	4.79	1.41	11.0	1.52	1.92	0.97	3.1
	6.0	4.5	5.68	1.45	12.9	1.51	1.90	0.96	3.6
55×55	5.0	4.1	5.27	1.53	14.7	1.67	2.11	1.06	3.7
	6.0	4.9	6.26	1.57	17.3	1.66	2.10	1.06	4.4
	8.0	6.4	8.18	1.65	22.0	1.64	2.07	1.06	5.7
	10.0	7.9	0.02	1.72	26.3	1.62	2.03	1.06	7.0

INDIAN STANDARD EQUAL ANGLES (Contd.)

Dimensions and Properties

Size	Thickness (t)	Weight per Meter Length	Sectional Area	Centre of Gravity Dimension J	Moment of Inertia About X—X	Radii of Gyration About X—X	Radii of Gyration About U—U (max.)	Radii of Gyration About V—V (min.)	Section Modulus About X—X (min.)
mm	mm	kg	cm²	cm	cm⁴	cm	cm	cm	cm³
60×60	5.0	4.5	5.75	1.65	19.2	1.82	2.31	1.16	4.4
	6.0	5.4	6.84	1.69	22.6	1.82	2.29	1.15	5.2
	8.0	7.0	8.96	1.77	29.0	1.80	2.27	1.15	6.8
	10.0	8.6	11.00	1.85	34.8	1.78	2.23	1.15	8.4
65×65	5.0	4.9	6.25	1.77	24.7	1.99	2.51	1.26	5.2
	6.0	5.8	7.44	1.81	29.1	1.98	2.50	1.26	6.2
	8.0	7.7	9.76	1.89	37.4	1.96	2.47	1.25	8.1
	10.0	9.4	12.00	1.97	45.0	1.94	2.44	1.25	9.9
70×70	5.0	5.3	6.77	1.89	31.1	2.15	2.71	1.36	6.3
	6.0	6.3	8.06	1.94	36.8	2.14	2.70	1.36	7.3
	8.0	8.3	10.58	2.02	47.4	2.12	2.67	1.35	9.5
	10.0	10.2	13.02	2.10	57.2	2.10	2.64	1.35	11.7
75×75	5.0	5.7	7.27	2.02	38.7	2.31	2.92	1.46	7.1
	6.0	6.8	8.66	2.06	45.7	2.30	2.91	1.46	8.4
	8.0	8.9	11.38	2.14	69.0	2.28	2.88	1.45	11.0
	10.0	11.0	14.02	2.22	71.4	2.26	2.84	1.45	13.5
80×80	6.0	7.3	9.29	2.18	66.0	2.46	3.11	1.56	9.6
	8.0	9.6	12.21	2.27	72.5	2.44	3.08	1.55	12.6
	10.0	11.8	15.05	2.34	87.7	2.41	3.04	1.55	15.5
	12.0	14.0	17.81	2.42	101.9	2.39	3.01	1.54	18.3
90×90	6.0	8.2	10.47	2.42	80.1	2.77	3.50	1.75	12.2
	8.0	10.8	13.79	2.51	104.2	2.75	3.47	1.75	16.0
	10.0	13.4	17.03	2.59	126.7	2.73	3.44	1.74	19.8
	12.0	15.8	20.19	2.66	147.9	2.71	3.41	1.74	23.3
100×100	6.0	9.2	11.67	2.67	111.3	3.09	3.91	1.95	15.2
	8.0	12.1	15.39	2.76	145.1	3.07	3.88	1.95	20.0
	10.0	14.9	19.03	2.84	177.0	3.05	3.85	1.94	24.7
	12.0	17.7	22.59	2.92	207.0	3.03	3.82	1.94	29.2
110×110	8.0	13.4	17.02	3.00	195.0	3.38	4.28	2.14	24.4
	10.0	16.5	21.06	3.08	238.4	3.36	4.25	2.14	30.1
	12.0	19.6	25.02	3.16	279.6	3.34	4.22	2.13	35.7
	15.0	24.2	30.81	3.27	337.4	3.31	4.17	2.13	43.7

INDIAN STANDARD EQUAL ANGLES (Contd.)

Dimentions and Properties

Size	Thickness (t)	Weight per Meter Length	Sectional Area	Centre of Gravity Dimension J	Moment of Inertia About X—X	Radii of Gyration About X—X	U—U (max.)	V—V (min.)	Section Modulus About X—Z)min.
mm	mm	kg	cm²	cm	cm	cm	cm	cm	cm³
130 × 130	8.0	15.9	20.22	3.50	328.3	4.03	5.10	2.55	34.5
	10.0	19.7	25.06	3.58	402.7	4.01	5.07	2.54	42.7
	12.0	23.4	29.82	3.66	473.8	3.99	5.03	2.54	50.7
	15.0	28.9	36.81	3.78	574.6	3.95	4.98	2.53	62.3
150 × 150	10.0	22.8	29.03	4.06	622.4	4.63	5.86	2.93	56.9
	12.0	27.2	34.59	4.14	735.4	4.61	5.83	2.93	67.7
	15.0	33.6	42.78	4.26	896.8	4.58	5.78	2.92	83.5
	18.0	39.9	50.79	4.38	1048.9	4.54	5.73	2.91	98.7
200 × 200	12.0	36.6	46.61	5.36	1788.9	6.20	7.84	3.92	122.2
	15.0	45.4	57.80	5.49	2197.7	6.17	7.79	3.91	151.4
	18.0	54.0	58.81	5.61	2588.7	6.13	7.75	3.90	179.9
	25.0	73.6	93.80	5.88	3436.3	6.05	7.63	3.88	243.3

Standard Wire & Sheet Gauges (Also see page 4/17)

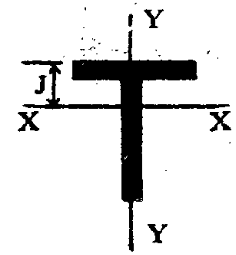
There are many gauges in use differing from each other, and to avoid confusion "gauge" or thickness should be clearly mentioned.

1. Gauge No. or Size 1/0, 2/0, 3/0 and so on, is also written as 0,00, 000, etc. 2. SWG is Imperial (English) Standard Wire Gauge and is used for wires and sheets 3 BWG is Birmingham Wire Gauge (also called Birminghan Gauge), is used for strips, sheets, bands, hoops and wires. Stubb's Steel Wire Gauge (SSWG) is nearly equal to BWG. 4. B&S or AWG is Browne and Sharpe (American) 5. BG is Birmingham Gauge and is used for sheets, hoops and plate thickness.

WSWG is Whitworth Standard Wire Gauge and is used mainly for threads of bolts, nuts and screws.

Zinc gauge is different and is used for zinc sheets.

"Gauge measures" are available made of thin iron plates of flat, circular or oblong shapes in which gauge thickness are cut at the edges.



INDIAN STANDARD STEEL TEE BARS

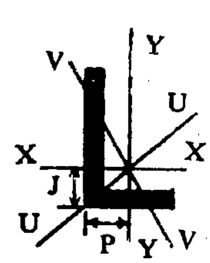
Dimensions and Properties

Size Depth × Width	Weight per Metre Length	Sectional Area	Centre of Gravity Dimension J	Moment of Inertia About X—X	Radii of Gyration About X—X	Y—Y	Section Modulus About X—X
mm	kg	cm²	J	cm⁴	cm	cm	cm³
20×20	0.9	1.13	0.60	0.4	0.59	0.39	0.3
30×30	1.4	1.75	0.83	1.4	0.89	0.57	0.6
40×40	3.5	4.48	1.20	6.3	1.18	0.82	2.2
75×50	3.5	4.50	2.00	24.8	2.35	1.01	4.5
50×50	4.0	5.11	1.19	9.9	1.39	1.12	2.6
87.5×50	4.0	5.14	2.50	39.0	2.75	0.97	6.2
50×50	4.5	5.70	1.44	12.7	1.50	1.02	3.6
100×60	5.0	6.32	2.81	63.5	3.17	1.17	8.8
60×60	5.4	6.90	1.67	22.5	1.81	1.21	5.2
112.5×80	6.4	8.14	3.01	101.6	3.53	1.58	12.3
75×80	7.1	9.04	1.72	41.9	2.15	1.75	7.2
100×50	8.1	10.37	3.03	99.0	3.09	0.96	14.2
80×80	9.6	12.25	2.23	71.2	2.41	1.62	12.3
100×100	12.7	16.16	2.13	116.6	2.69	2.15	14.8
100×100	15.0	19.10	2.79	173.8	3.02	2.05	24.1
75×150	15.3	19.49	1.62	96.2	2.22	3.44	16.4
150×75	15.7	19.96	4.75	450.2	4.75	1.36	43.9
100×200	20.0	25.47	1.91	193.8	2.76	4.42	24.0
150×150	22.8	29.08	3.95	603.8	4.56	3.03	54.6
125×250	27.4	34.85	2.37	415.4	3.45	5.37	41.0
200×165	28.4	36.22	4.78	1267.8	5.92	3.15	83.3
150×250	29.4	37.42	2.66	573.7	3.92	4.41	46.5
250×180	37.5	47.75	6.40	2774.4	7.62	3.34	149.2

Depth is total depth: Width is width of flange at top.

INDIAN STANDARD UNEQUAL ANGLES

Dimensions and Properties

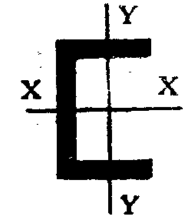


Size mm	Thickness (t) mm	Weight per Meter Length kg	Sectional Area cm²	Centre of Gravity Dimension J cm	Centre of Gravity Dimension P cm	Moment of Inertia About X—X cm⁴	Moment of Inertia About Y—Y cm⁴	Radii of Gyration About X—X cm	Radii of Gyration About Y—Y cm	Radii of Gyration (min.) V—V cm	Section Moduli About X—X cm³	Section Moduli About Y—Y cm³
30×20	3.0	1.1	1.41	0.98	0.49	1.2	0.4	0.92	0.54	0.41	0.6	0.3
	4.0	1.4	1.84	1.02	0.53	1.5	0.5	0.92	0.54	0.41	0.8	0.4
	5.0	1.8	2.25	1.06	0.57	1·9	0.6	0.91	0.53	0.41	1.0	0.4
40×25	3.0	1.5	1.88	1.30	0.57	3.0	0.9	1.25	0.68	0.52	1.1	0.5
	4.0	1.9	2.46	1.35	0.62	3.8	1.1	1.25	0.68	0.52	1.4	0.6
	5.0	2.4	3.02	1.39	0.66	4.6	1.4	1.24	0.67	0.52	1.8	0.7
	6.0	2.8	3.56	1.43	0.69	5.4	1.6	1.23	0.66	0.52	2.1	0.9
45×30	3.0	1.7	2.18	1.42	0.69	4.4	1.5	1.42	0.84	0.63	1.4	0.7
	4.0	2.2	2.86	1.47	0.73	5.7	2.0	1.41	0.84	0.63	1.9	0.9
	5.0	2.8	3.52	1.51	0.77	6.9	2.4	1.40	0.83	0.63	2.3	1.1
	6.0	3.3	4.16	1.55	0.81	8.0	2.8	1.39	0.82	0.63	2.7	1.3
50×30	3.0	1.8	2.34	1.63	0.65	5.9	1.6	1.59	0.82	0.65	1.7	0.7
	4.0	2.4	3.07	1.68	0.70	7.7	2.1	1.58	0.82	0.63	2.3	0.9
	5.0	3.0	3.78	1.72	0.74	9.3	2.5	1.57	0.81	0.63	2.8	1.1
	6.0	3.5	4.47	1.76	0.78	10.9	2.9	1.56	1.80	0.63	3.4	1.3
60×40	5.0	3.7	4.76	1.95	0.96	16.9	6.0	1.89	1.12	0.85	4.2	2.0
	6.0	4.4	5.65	1.99	1.00	19.9	7.0	1.88	1.11	0.85	5.0	2.3
	8.0	5.8	7.37	2.07	1.08	25.4	8.8	1.86	1.10	0.84	6.5	3.0
65×45	5.0	4.1	5.26	2.07	1.08	22.1	8.6	2.05	1.28	0.96	5.0	2.5
	6.0	4.9	6.25	2.11	1.12	26.0	10.1	2.04	1.27	0.95	5.9	3.0
	8.0	6.4	8.17	2.19	1.20	33.2	12.8	2.02	1.25	0.95	7.7	3.9
70×45	5.0	4.3	5.52	2.27	1.04	27.2	8.8	2.22	1.26	0.96	5.7	2.5
	6.0	5.2	6.56	2.32	1.09	32.0	10.3	2.21	1.25	0.96	6.8	3.0
	8.0	6.7	8.58	2.40	1.16	41.0	13.1	2.19	1.24	0.95	8.9	3.9
	10.0	8.3	10.52	2.48	1.24	49.3	15.6	2.16	1.22	0.95	10.9	4.8
75×50	5.0	4.7	6.02	2.39	1.16	34.1	12.2	2.38	1.42	1.07	6.7	3.2
	6.0	5.6	7.16	2.44	1.20	40.3	14.3	2.37	1.41	1.07	8.0	3.8
	8.0	7.4	9.38	2.52	1.28	51.8	18.3	2.35	1.40	1.06	10.4	4.9
	10.0	9.0	11.52	2.60	1.36	62.3	21.8	2.33	1.38	1.06	12.7	6.0
80×50	5.0	4.9	6.27	2.60	1.12	40.6	12.3	2.55	1.40	1.07	7.5	3.2
	6.0	5.9	7.46	2.64	1.16	48.0	14.4	2.54	1.39	1.07	9.0	3.8
	8.0	7.7	9.78	2.73	1.24	61.9	18.5	2.52	1.37	1.06	11.7	4.9
	10.0	9.4	12.02	2.81	1.32	74.7	22.1	2.49	1.36	1.06	14.4	6.0
90×60	6.0	6.8	8.65	2.87	1.39	70.6	25.2	2.86	1.71	1.28	11.5	5.5
	8·0	8.9	11.37	2.96	1.48	91.5	32.4	2.84	1.69	1.28	15.1	7.2
	10.0	11.0	14.01	3.04	1.55	110.9	39.1	2.81	1.67	1.27	18.6	8.8
	12.0	13.0	16.57	3.12	1.63	129.1	45.2	2.79	1.65	1·27	22.0	10.3
100×65	6.0	7.5	9.55	3.19	1.47	96.7	32.4	3.18	1.84	1.39	14.2	6.4
	8.0	9.9	12.57	3.28	1.55	125.9	41.9	3.16	1.83	1.39	18.7	8.5
	10.0	12.2	15.51	3.37	163	153.2	50.7	3.14	1.81	1.38	23.1	10.4
100×75	6.0	8.0	10.14	3.01	1.78	100.9	48.7	3.15	2.19	1.59	14.4	8.5
	8.0	10.5	13.36	3.10	1.87	131.6	63.3	3.14	2.18	1.59	19.1	11.2
	10.0	13.0	16.50	3.19	1.95	160.4	76.9	3.12	2.16	1.58	23.6	13.8
	12.0	15.4	19.56	3.27	2.03	187.5	89.5	3.10	2.14	1.58	27.9	16.3

INDIAN STANDARD UNEQUAL ANGLES (Contd.)

Dimensions and Properties

Size	Thick-ness (t)	Weight per Metre Length	Sectional Area	Centre of Gravity Dimension J	Centre of Gravity Dimension P	Moment of Inertia About X—X	Moment of Inertia About Y—Y	Radius of Gyration About X—X	Radius of Gyration About Y—Y	Radius of Gyration (min:) V—V	Section Moduli About X—X	Section Moduli About Y—Y
mm	mm	kg	cm²	cm	cm	cm⁴	cm⁴	cm	cm	cm	cm³	cm³
125 × 75	6.0	9.2	11.66	4.05	1.59	187.8	51.6	4.10	1.10	1.62	22.2	8.7
	8.0	12.1	15.38	4.15	1.68	245.5	67.2	4.00	2.09	1.61	29.4	11.5
	10.0	14.9	19.02	4.24	1.76	300.3	81.6	3.97	2.07	1.61	36.3	14.2
125 × 95	6.0	10.1	12.86	3.10	2.22	203.2	102.1	3.97	2.82	2.03	23.1	14.0
	8.0	13.3	16.98	3.80	2.31	266.0	133.3	3.96	2.80	2.02	30.6	18.5
	10.0	16.5	21.02	3.88	2.39	325.8	162.7	3.94	2.78	2.02	37.8	22.9
	12.0	19.6	24.98	3.96	2.47	382.6	190.4	3.91	2.76	2.01	44.8	27.1
150 × 75	8.0	13.7	17.42	5.23	1.53	407.2	70.2	4.83	2.01	1.60	41.7	11.8
	10.0	16.9	21.56	5.32	1.61	499.1	85.3	4.81	1.99	1.59	51.6	14.5
	12.0	20.1	25.62	5.41	1.69	587.0	99.5	4.79	1.97	1.58	61.2	17.1
150 × 115	8.0	16.2	20.58	4.46	2.73	465.7	238.9	4.76	3.41	2.45	44.2	23.2
	10.0	20.0	25.52	4.55	2.82	573.3	293.4	4.74	3.39	2.44	54.9	33.8
	12.0	23.8	30.38	4.64	2.90	676.5	245.3	4.72	3.37	2.44	65.3	40.2
	15.0	29.5	37.52	4.76	3.02	823.5	418.6	4.69	3.34	2.43	80.4	49.4
200 × 100	10.0	22.8	29.03	6.96	2.01	1210.0	209.2	6.46	2.68	2.14	92.8	26.2
	12.0	27.2	34.59	7.05	2.10	1431.7	246.2	6.43	2.67	2.13	110.6	31.8
	15.0	33.6	42.78	7.18	2.22	1750.5	298.1	6.40	2.64	2.12	136.5	38.3
200 × 150	10.0	26.7	34.00	5.99	3.51	1377.9	669.6	6.37	4.44	3.21	98.3	58.3
	12.0	31.8	40.56	6.08	3.60	1634.9	793.2	6.35	4.42	3.21	117.4	69.6
	15.0	39.4	50.25	6.20	3.72	2005.6	969.2	6.32	4.39	3.20	145.4	86.0
	18.0	46.9	59.76	6.33	3.84	2359.4	1136.9	6.28	4.36	3.19	172.5	101.9

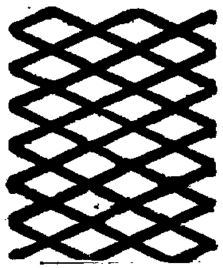


INDIAN STANDARD STEEL CHANNELS

Dimensions and Properties

Size Depth × Width	Weight per Metre Length	Sectional Area	Moment of Inertia About X—X	Moment of Inertia About Y—Y	Radii of Gyration About X—X	Radii of Gyration About Y—Y	Section Moduli About X—X	Section Moduli About Y—Y
mm	kg	cm²	cm⁴	cm⁴	cm	cm	cm³	cm³
Junior Channels (ISJC Series)								
100×45	5.8	7.41	123.8	14.9	4.09	1.42	24.8	4.8
125×50	7.9	10.07	270.0	25.7	5.18	1.60	43.2	7.6
150×55	9.9	12.65	471.1	37.9	6.10	1.73	62.8	9.9
175×60	11.2	14.24	719.9	50.5	7.11	1.88	82.3	11.9
200×70	13.9	17.77	1161.2	84.2	8.08	2.18	116.1	16.7
Light Channels (ISLC Series)								
75×40	5.7	7.26	66.1	11.5	3.02	1.26	17.6	4.3
100×50	7.9	10.02	164.7	24.8	4.06	1.57	32.9	7.3
125×65	10.7	13.67	356.8	57.2	5.11	2.05	57.1	12.8
150×75	14.4	18.36	697.2	103.2	6.16	2.37	93.0	20.
175×75	17.6	22.40	1148.4	126.5	7.16	2.38	131.3	24.8
200×75	20.6	26.22	1725.5	146.9	8.11	2.37	172.6	28.
225×90	24.0	30.53	2547.9	209.5	9.14	2.62	226.5	32.0
250×100	28.0	35.65	3687.5	298.4	10.17	2.89	295.0	40.
300×100	33.1	42.11	6047.9	346.0	11.98	2.87	403.2	46.
350×100	38.8	49.47	9312.6	394.6	13.72	2.82	532.1	52.
400×100	45.7	58.25	13989.5	460.4	15.50	2.81	699.5	60.
Medium Weight Channels (ISMC Series)								
75×40	6.8	8.67	76.0	12.6	2.96	1.21	20.3	4.
100×50	9.2	11.70	186.7	25.9	4.00	1.49	37.3	7.
125×65	12.7	16.19	416.4	59.9	5.07	1.92	66.6	13.
150×75	16.4	20.88	779.4	102.3	6.11	2.21	103.9	19.
175×75	19.1	24.38	1223.3	121.0	7.08	2.23	139.8	22.
200×75	22.1	28.21	1819.3	140.4	8.03	2.23	181.9	26.
225×80	25.9	33.01	2694.6	187.2	9.03	2.38	339.5	32.
250×80	30.4	38.67	3816.8	219.1	9.94	2.38	305.3	38.
300×90	35.8	45.64	6362.6	310.8	11.81	2.61	424.2	46.
350×100	42.1	53.66	10008.0	430.6	13.66	2.83	571.9	57.
400×100	49.4	62.93	15082.8	504.8	15.48	2.83	754.1	66.

SIZE & WEIGHT OF EXPANDED METAL (XPM) SHEETS

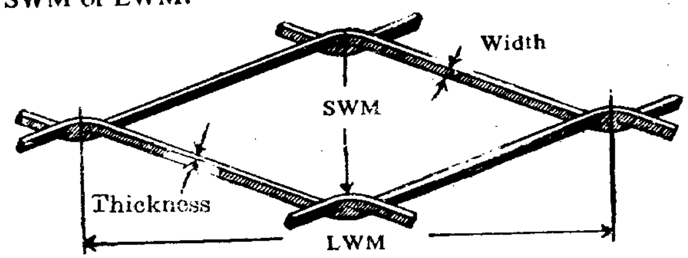
Size of Mesh		Dimensions of Strands		Weight per sq. m	Size of Sheet Normally Stocked
SWM mm	LWM mm	Width mm	Thickness mm	kg	metres
100	250	6.25	3.15	3.082	
100	250	5.00	3.15	2.470	
100	250	3.25	3.15	1.599	
75	200	6.50	3.15	4.282	
75	200	5.00	3.15	3.294	
75	200	3.25	3.15	2.141	
40	115	6.50	3.15	8.023	
40	115	5.00	3.15	6.172	
40	75	5.00	3.15	6.172	
40	75	3.25	2.24	2.854	2.50×3.75
40	115	3.25	3.15	4.007	
40	75	3.25	3.15	4.007	2.50×3.75
40	115	3.25	1.60	2.039	1.25×3.75
40	75	3.25	1.60	2.039	
25	75	3.25	3.15	6.423	2.50×3.75
25	75	3.25	2.24	4.564	
25	75	3.25	1.60	3.262	2.50×3.75
25	75	3.25	1.25	2.548	1.25×3.75
20	60	3.25	3.15	8.023	
20	50	3.25	3.15	8.023	2.50×3.75
20	60	3.25	2.24	5.709	
20	50	3.25	2.24	5.709	2.50×3.75
20	60	3.25	1.60	4.078	1.25×3.75
20	50	3.25	1.60	4.078	
20	60	2.50	1.25	2.447	
20	50	2.50	1.25	2.447	
12.5	50	3.25	1.60	6.525	
12.5	40	3.25	1.60	6.525	
12.5	50	2.50	1.60	5.019	
12.5	50	2.50	1.25	3.921	2.50×2.75
12.5	40	2.50	1.25	3.921	1.25×2.75
12.5	50	2.50	1.00	3.137	
12.5	40	2.50	1.00	3.137	

(Contd.)

SIZE & WEIGHT OF EXPANDED METAL (XPM) SHEETS

Size of Mesh		Dimensions of Strands		Weight per sq. m	Size of Sheet Normally Stocked
SWM mm	LWM mm	Width mm	Thickness mm	kg	metres
10	40	3.25	1.60	8.156	
10	40	2.50	1.25	4.901	2.50×1.75
10	40	2.50	1.00	3.921	1.25×1.75
6	25	3.25	1.60	13.591	
6	25	2.50	1.25	8.164	
6	25	2.50	1.00	6.533	
5	20	2.50	1.00	7.843	2.50×1.25
3	15	1.50	1.00	7.843	

SWM is shortway of mesh, LWM is longway of mesh. When ordering XPM sheets, it is necessery to specify which dimension refers to SWM or LWM.



RIVET HOLES IN TENSION MEMBERS

Area in sq. cm to be deducted for each rivet hole per mm thickness of metal

Dia. of Rivet—mm	12	14	16	18	20	22
Rivet Hole Dia—mm	13.5	15.5	17.5	19.5	21·5	23.5
Area of Rivet Hole—sq. cm.	0.135	0.155	0.175	0.195	0.215	0.235

24	27	30	33	36	39	42	48
25.5	29.0	32.0	35.0	38.0	41.0	44.0	50.0
0.255	0.290	0.320	0.350	0.380	0.410	0.440	0.500

WEIGHTS & STANDARD SIZES OF STEEL (BLACK) SHEETS

Thickness BG	28	26	24	22	21	20	19	18
mm	0.40	0.50	0.63	0.80	0.90	1.00	1.12	1.25
Weight—kg/sq. m	3.15	3.90	4.95	6.30	7.05	7.85	8.80	9.80

17	16	15	14	13	12	11	10	9	8	7
1.40	1.60	1.80	2.00	2.24	2.50	2.80	3.15	3.55	4.00	4.50
11.00	12.55	14.15	15.70	17.60	19.60	22.00	24.75	27.85	31.40	35.30

BG is Birmingham Gauge No. which is nearest to the standard thickness in mm. (See page 4/17)

Steel sheets in odd BG gauges, viz., 9, 11, 13, 15, 17, 19, 21 are not generally manufactured except by special arrangement (for bulk supply).

Standard Lengths are : 1800, 2000, 2200, 2500, 2800, 3000, 3200, 3600, 4000, 4500, mm.

Standard Widths are :—600, 750, 900, 1000, 1100, 1200, 1250 mm.

Theoretical weights in kg/sq. m have been calculated on the basis that steel weights 7.85 kg/sq. m per mm thickness, or 7.85 grams/cu. cm rounded off to the nearest 0.05 kg.

Sheets of thickness above 4 mm. are classified as plates.

COMPARISON OF WEIGHTS OF SHEETS OF DIFFERENT METALS

in kg/sq. metre for 16 BG gauge (1.588 mm) thickness—approximate

Cast iron	Wrought iron	Steel	Brass	Copper	Lead	Zinc	Aluminium
11.47	12.20	12.55	13.37	13.08	18.10	11.42	4.26

WEIGHTS & STANDARD SIZES OF STEEL STRIPS

in kilograms per metre length per 10 mm width

Thickness BG	16	15	14	13	12
mm	1.6	1.80	2.00	2.24	2.50
Weight—Kg/m/10mm	0.126	0.141	0.157	0.176	0.196

11	10	9	8	7	6	4	2	0
2.80	3.15	3.55	4.00	4.50	5.00	6.00	8.00	10.0
0.220	0.248	0.279	0.314	0.353	0.392	0.471	0.628	0.785

BG is the No. nearest to the standard thickness in mm.

Standard Widths are : 100, 125, 160, 200, 250, 320, 400, 500, 650, 800, 950, 1050, 1150, 1250, 1300, 1450, 1550 mm.

Weight of Mild Steel Flats and Plates

$$\frac{\text{Width in mm} \times \text{Thickness in mm}}{127} \text{ kg per metre}$$

WEIGHTS & STANDARD SIZES OF STEEL PLATES

Thickness—mm	5	6	8	10	12	14	16
Weight—kg/sq. m	39.25	47.10	62.80	78.50	94.20	109.90	125.60

18	20	22	25	28	32	36	40
141.30	157.00	172.70	196.25	219.80	251.20	282.60	314.00

Sheets of thickness above 4 mm are classified as plates.

Standard Widths are :—900, 1000, 1100, 1200, 1250, 1400, 1500, 1600, 1800, 2000, 2200, 2500, mm.

Standard Lengths are :—2000, 2200, 2500, 2800, 3200, 3600, 4000, 4500, 5000, 5600, 6300, 7100, 8000, 9000, 10000, 11000, 12500 mm.

WEIGHTS OF PLAIN & CORRUGATED GALVANIZED STEEL SHEETS

Thickness							
mm	1.60	1.25	1.00	0.80	0.63	0.50	0.40
BG	16	18	20	22	24	26	28
Class I (750 g of zinc coating per sq. m)							
Weight-kg/sq.m	13.31	10.56	8.60	7.03	5.70	4.65	3.90
Class II (600 g of zinc coating per sq. m)							
Weight-kg/sq.m	13.16	10.41	8.45	6.88	5.55	4.50	3.75
Class III (450 g of zinc coating per sq. m)							
Weight-kg/sq.m	13.01	10.26	8.30	6.73	5.40	4.35	3.60
Class IV (375 g of zinc coating per sq. m)							
Weight-kg/sq.m	12.94	10.19	8.22	6.66	5.32	4.27	3.52

Zinc (spelter) coating is on both sides.

Corrugated Sheets

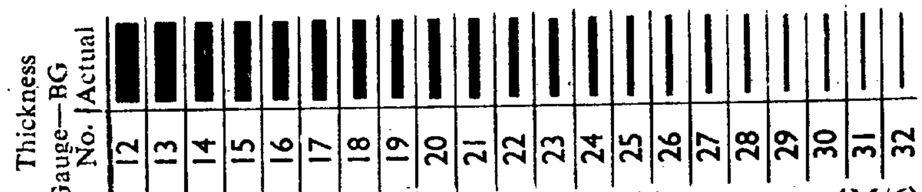
In the corrugated galvanized iron sheets (CGI), the depth of corrugation in 18 mm. and the pitch 75 mm. Number of corrugations are 8 or 10 per sheet. 10 corrugations is most common.

75 cm wide flat sheets are given 8 corrugations and 90 cm wide sheets 10 corrugations, which reduce the width of the sheets to 66 cm and 80 cm. (Sometimes 11 corrugations are given to 99 cm wide sheets reducing the width to 88.5 cm)

Weights of corrugated sheets are worked out based on weights of plain galvanized sheets taking into account width of sheets before corrugation.

WIRE & SHEET METAL GAUGES

No.	Dia. or Thickness in mm SWG	BG	B & S or AWG	No.	Dia. or Thickness in mm SWG	BG	B & S or AWG
7/0	12.700	16.93	—	23	0.610	0.707	0.573
6/0	11.786	15.88	—	24	0.559	0.629	0.511
5/0	10.973	14.94	—	25	0.508	0.560	0.455
4/0	10.160	13.76	11.68	26	0.457	0.498	0.405
3/0	9.449	12.70	10.40	27	0.417	0.443	0.361
2/0	8.839	11.31	9.27	28	0.376	0.397	0.321
0	8.230	10.07	8.25	29	0.345	0.353	0.286
1	7.620	8.971	7.35	30	0.315	0.312	0.255
2	7.010	7.993	6.54	31	0.295	0.279	0.227
3	6.401	7.122	5.83	32	0.274	0.249	0.202
4	5.893	6.350	5.19	33	0.254	0.221	0.180
5	5.385	5.652	4.62	34	0.234	0·196	0.160
6	4.877	5.032	4.11	35	0.213	0.175	0.143
7	4.470	4.481	3.67	36	0.193	0.155	0.127
8	4.064	3.988	3.26	37	0.173	0.137	0.113
9	3.658	3.551	2.91	38	0.152	0.122	0.101
10	3.251	3.175	2.59	39	0.132	0.109	0.090
11	2.946	2.827	2.30	40	0.122	0.098	0.080
12	2.642	2.517	2.05	41	0.112	0.087	—
13	2.337	2.240	1.83	42	0.102	0.078	—
14	2.032	1.994	1.63	43	0.091	0.069	—
15	1.829	1.775	1.45	44	0.081	0.061	—
16	1.626	1.588	1.29	45	0.071	0.055	—
17	1.422	1.412	1.15	46	0.061	0.049	—
18	1.219	1.257	1.02	47	0.051	0.043	—
19	1.016	1.118	0.91	48	0.041	0.039	—
20	0.914	0.996	0.81	49	0.030	0.034	—
21	0.813	0.886	0.72	50	0.025	0.030	—
22	0.711	0.794	0.64	51	—	0.027	—

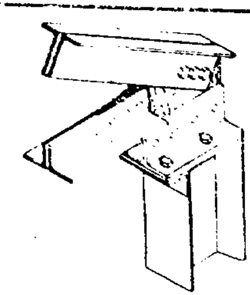


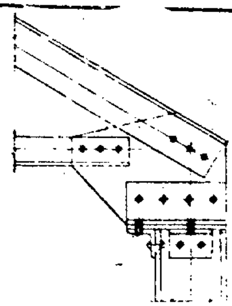
(Also see page 4M/6)

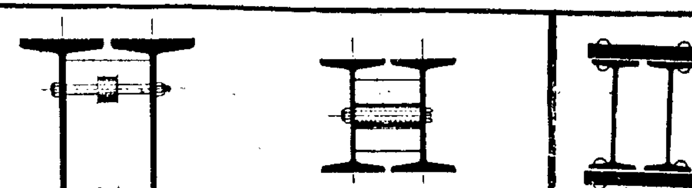
SAFE DISTRIBUTED LOADS FOR INDIAN STANDARD STEEL BEAMS (R.S. Joists)

With Adequate Lateral Support for Compression Flange

No.	Size of Beam: Depth × Width × Web Thickness (mm)	Weight per Metre Length (kg)	Section Modulus (cm³)	2× Shear Carrying Capacity (tonnes)	"L" (see note d) (m)	Span in Metres: 3.0	3.5	4.0	4.5	5.0	5.5	6.0	7.0	8.0	9.0	10.0	No.
						Safe Loads in Metric Tonnes											
1	600×250×11.8	145.1	3854.2	133.8	6.5	162	139	121	108	97	88	81	69	61	54	49	1
2	600×250×11.2	133.7	3540.0	127.0	6.0	149	127	111	99	89	81	74	64	56	50	45	2
3	600×210×12.0	122.6	3060.4	136.0	5.0	129	110	96	86	77	70	64	55	48	43	39	3
4	550×250×10.5	112.5	2723.9	109.2	6.0	114	98	85	76	69	62	57	49	43	38	34	4
5	550×190×11.2*	103.7	2359.8	116.4	4.5	99	85	74	66	59	54	50	42	37	33	30	5
6	600×210×10.5	99.5	2428.9	119.0	4.5	102	87	76	68	61	56	51	44	38	34	31	6
7	500×250× 9.9	95.2	2091.6	93.6	5.5	88	75	66	59	53	48	44	38	33	29	26	7
8	500×180×10.2*	86.9	1808.7	96.4	4.0	76	65	57	51	46	41	38	33	28	25	22	8
9	550×190× 9.9	86.3	1933.2	103.0	4.0	81	70	61	54	49	44	41	35	30	27	24	9
10	450×200× 9.2	79.4	1558.1	78.2	4.5	65	56	49	44	39	36	33	28	25	21	17	10
11	500×180× 9.2	75.0	1543.2	87.0	4.0	65	56	49	43	39	35	32	28	24	22	19	11
12	450×150× 9.4	72.4	1350.7	80.0	3.5	57	49	43	38	34	31	28	24	21	19	15	12
13	400×200× 8.6*	66.7	1171.3	65.0	4.5	49	42	37	33	21	27	25	21	18	14	12	13
14	450×170× 8.6	65.3	1223.8	73.2	4.0	51	44	39	34	39	28	26	22	19	17	14	14
15	400×140× 8.9	61.6	1022.9	67.2	3.5	43	37	32	29	26	23	21	18	16	12	10	15
16	350×200× 8.0*	56.9	887.0	53.0	4.5	37	32	28	25	22	20	19	16	12	9.5	—	16
17	400×165× 8.0	56.9	965.3	60.4	3.5	41	35	30	27	24	22	20	17	15	12	9.5	17
18	350×140× 8.1	52.4	778.9	53.6	3.5	33	28	24	22	20	18	16	14	10	8.3	—	18
19	350×165× 7.4*	49.5	751.9	49.0	3.5	32	27	24	21	19	17	16	13	10	8.1	—	19
20	300×200× 7.4*	48.1	654.8	42.0	4.5	27	24	21	18	16	15	13	10	—			20
21	300×140× 7.5*	44.2	573.6	42.6	3.5	24	21	18	16	14	13	12	8.7				21
22	325×165× 7.0	43.1	607.7	43.0	3.5	25	22	19	17	15	14	13	10				22
23	250×200× 6.7*	40.9	475.4	31.6	5.0	20	17	15	13	12	9.8	8.2	—				23
24	200×150× 6.7	37.7	488.9	38.0	3.5	21	18	15	14	12	11	10	7.4				24







I BEAMS WITH CAST IRON SEPARATORS

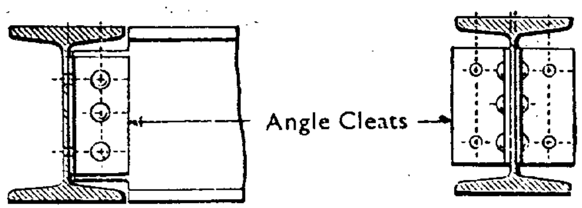
SAFE DISTRIBUTED LOADS FOR INDIAN STANDARD STEEL BEAMS (Contd.)

No.	Size of Beam: Depth × Width × Web Thickness mm	Weight per Metre Length kg	Section Modulus cm³	2× Shear Carrying Capacity tonnes	"L" m
25	250×125× 6.9*	37.3	410.5	32.6	3.5
26	225×150× 6.4*	33.9	348.5	27.2	4.0
27	275×140× 6.4	33.0	392.4	33.2	3.0
28	225×110× 6.5	31.2	305.9	27.6	3.0
29	200×140× 6.1*	28.8	262.5	23.0	3.5
30	250×125× 6.1	27.9	297.4	28.8	3.0
31	200×100× 5.7*	25.4	223.5	21.6	3.0
32	225×100× 5.8	23.5	222.4	24.6	2.5
33	175×125× 5.8	22.1	181.9	19.2	3.0
34	200×100× 5.4	19.8	169.7	20.4	2.5
35	175× 90× 5.5	19.3	145.4	18.2	2.5
36	150×100× 5.4*	17.0	111.9	15.4	2.5
37	175×190× 5.1	16.7	125.3	16.8	2.0
38	150× 80× 4.8*	14.9	96.9	13.6	2.0
39	150× 80× 4.8*	14.2	91.8	13.6	2.0
40	125× 75× 4.4*	13.0	71.8	10.4	2.5
41	225× 80× 3.7	12.8	116.3	15.8	1.5
42	125× 75× 4.4*	11.9	65.1	10.4	2.0
43	100× 75× 4.0*	11.5	51.5	7.6	2.5
44	200× 60× 3.4*	9.9	78.1	12.8	1.0
45	175× 50× 3.2	8.1	54.8	10.6	1.0
46	100× 50× 4.0*	8.0	33.6	7.6	1.5
47	150× 50× 3.0	7.1	42.9	8.6	1.0
48	75× 50× 3.7*	6.1	19.4	5.2	1.5

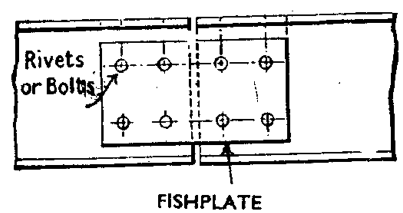
Span in Metres / Safe Loads in Metric Tonnes

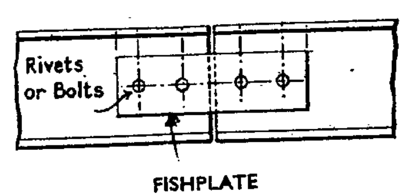
1.0	1.5	2.0	2.5	3.0	3.5	4.0	4.5	5.0	5.5	6.0	No.
52	34	26	21	17	15	13	11	10	8.4	7.0	25
44	29	22	18	15	12	11	9.5	7.8	—	—	26
49	33	25	20	16	14	12	11	9.9	8.8	7.4	27
38	26	19	15	13	11	9.6	8.3	6.8	—	—	28
33	22	16	13	11	9.4	8.2	6.5	5.2	—	—	29
37	25	19	15	12	11	9.4	8.3	7.4	6.0	5.1	30
28	19	14	11	9.4	8.0	6.9	5.5	4.4	—	—	31
28	19	14	11	9.4	8.0	7.0	6.0	5.0	—		32
22	14	11	8.7	7.2	6.1	4.6	—				33
21	14	11	8.6	7.2	6.1	5.3	4.2	3.4	—		34
18	12	9.2	7.3	6.1	5.1	4.0	—				35
14	9.4	7.0	5.6	4.6	3.4	2.6	—				36
16	10	7.9	6.3	5.2	4.4	3.4	—				37
12	8.1	6.1	4.9	3.9	2.9	2.2	—				38
12	7.7	5.8	4.6	3.7	2.8	2.1	—				39
9.0	6.0	4.5	3.5	2.5	—						40
15	9.8	7.4	5.9	4.9	4.2	3.7	3.2	1.6			41
8.2	5.5	4.1	3.2	2.3	—						42
6.5	4.3	3.3	2.6	1.4	—						43
9.8	6.6	4.9	3.9	3.3	2.8	2.4	1.9	1.6	—		44
6.9	4.6	3.4	2.8	2.3	1.8	1.5	—				45
4.2	2.8	2.1	1.6	0.9	—						46
5.4	3.6	2.7	2.2	1.8	1.3	1.0	—				47
2.4	1.6	1.1	—								48

CONNECTIONS FOR BEAMS



CONNECTIONS FOR BEAMS





SAFE DISTRIBUTED LOADS FOR INDIAN STANDARD STEEL BEAMS (Contd.)

No.	Size of Beam — Depth × Width × Web Thickness (mm)	10.5	11.0	11.5	12.0	12.5	13.0	13.5	14.0	14.5	15.0
		Safe Loads in Metric Tonnes									
1	600×250×11.8	46	44	42	40	37	34	31	29	27	25
2	600×250×11.2	42	41	39	37	34	31	29	27	25	23
3	600×210×12.0	37	35	33	32	29	27	25	23	22	20
4	550×250×10.5	33	31	28	26	24	22	—	—	—	—
5	550×190×11.2*	28	26	24	22	21	19	—	—	—	—
6	600×210×10.5	22	28	27	25	23	21	20	18	17	16
7	500×250× 9.9	25	21	23	18	—	—	—	—	—	—
8	500×180×10.2*	22	18	20	16	—	—				
9	550×190× 9.9	23	22	20	18	17	16				
10	450×200× 9.2	16	14	—	—	—	—				
11	500×180× 9.2	17	16	14	13	—					
12	450×150× 9.4	13	12	—	—						

(Span in Metres across columns 10.5–15.0)

Notes for R.S. Joists and Channels Used as Beams:

(a) The tabulated loads are calculated as uniformly distributed loads on a simply supported beam and include the weight of the joist.

(b) Allowable loads are in direct proportion to the length of the span except where the deflection exceeds the limit of 1/325 of the span, the loads have been reduced in value (in the right hand end columns) so as to keep the deflection within limits.

(c) The allowable loads have been calculated on the assumption that the beams have adequate lateral supports to the compression flange at distances apart not exceeding 20 times the width of the flange. Where a beam is not provided with adequate lateral support with floors or cross-beams, etc., the tabulated loads have to be reduced in value. (See Section 3)

(d) Column "L" gives the lengths of spans up to which tabulated loads are safe even without lateral supports.

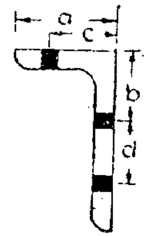
(e) Lengths of Bearing to take full shear at the ends should not be less than half of the depth of the beam.

(f) Beams marked with an asterisk (*) are not economical except when depth is limited. If Channels can be used their economy should also be investigated along with the beams.

(g) Check the selected section for its shear carrying capacity. Where the total load is in excess of the tabulated "Shear Carrying Capacity" of the beam, its web should be stregthened with web stiffeners.

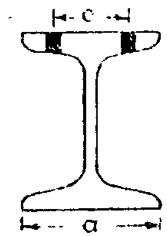
(h) Make proper provision in cases of eccentric loadings or any other special conditions of loading.

(i) Safe load figures have been 'rounded off' to the nearest tonne beyond 10 tonnes for ease of practical application.



SPACINGS OF HOLES & SIZE OF RIVETS IN INDIAN STANDARD STEEL SECTIONS

(Dimensions are in Millimetres)



Leg a	c	Rivet Size	Double Row of Rivets b	d
20	12			
25	15			
30	17			
35	19			
40	21	12 (Single Row of Rivets)		
45	25	12		
50	28	16		
55	30	20		
60	35	22		
65	35	22		
70	40	24		
75	40	24		
80	45	27		
90	50	27		
95	55	27		
100	60	12 (Double Row)	40	40
110	65	12	45	45
115	70	12	45	50
125	75	20	45	55
130	80	20	50	55
150	90	22	55	65
200	115	27	75	85

Width of Flange a	c	Rivet size
50	30	6
60	30	6
75	35	12
80	40	12
90	50	12
100	55	16
110	60	20
125	65	22
140	80	22
150	90	22
165	100	24
170	100	24
180	100	27
190	100	32
200	100	32
200	140	22
210	100	32
210	140	24
225	140	32

Sizes given for Angles apply to Channel Sections and sizes for Beams apply to Tees.

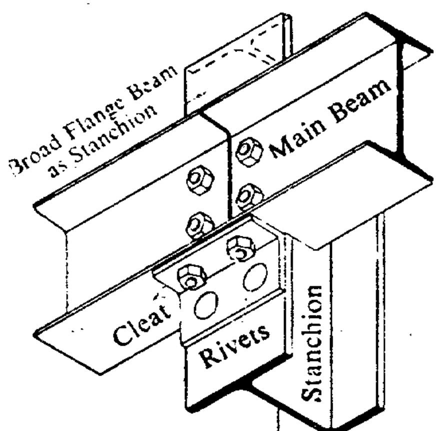
SAFE DISTRIBUTED LOADS FOR INDIAN STANDARD STEEL CHANNELS USED AS BEAMS

With Adequate Lateral Support for Compression Flange

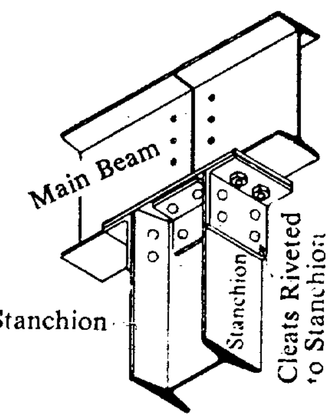
No.	Size of Channel: Depth × Width × Web Thickness (mm)	Weight per Metre Length (kg)	Section Modulus (cm^3)	2× Shear Carrying Capacity (Tonnes)	"L" (m)	Span in Metres 1.0	1.5	2.0	2.5	3.0	3.5	4.0	5.0	6.0	7.0	8.0	9.0	No.
						Safe Loads in Metric Tonnes												
1	75 × 40 × 3.7	5.7	17.6	5.2	1.5	2.1	1.4	1.0	—									1
2	100 × 45 × 3.0	5.8	24.8	5.6	1.0	3.0	2.0	1.5	1.1	0.8	—							2
3	75 × 40 × 4.4*	6.8	20.3	6.2	2.0	2.4	1.6	1.2	—									3
4	125 × 50 × 3.0	7.9	43.2	7.0	1.5	5.2	3.5	2.6	2.1	1.5								4
5	100 50 × 4.0*	7.9	32.9	7.6	1.5	3.9	2.6	2.0	1.4	1.1								5
6	100 × 50 × 4.7*	9.2	37.3	8.8	2.0	4.5	3.0	2.2	1.6	1.3								6
7	150 × 55 × 3.6*	9.9	62.8	10.2	1.5	7.5	5.0	3.8	3.0	2.5	2.0	1.6						7
8	125 × 65 × 4.4*	10.7	57.1	10.4	2.0	6.9	4.6	3.4	2.7	2.1								8
9	175 × 60 × 3.6	11.2	82.3	12.0	1.5	9.9	6.6	4.9	4.0	3.3	2.8	2.3						9
10	125 65 × 5.0*	12.7	66.6	11.8	2.0	8.0	5.3	4.0	3.2	2.5	—							10
11	200 × 70 × 4.1*	13.9	116.1	15.4	1.5	14	9.3	7.0	5.6	4.6	4.0	3.5	2.6	—				11
12	150 75 × 4.8*	14.4	93.0	13.6	2.0	11	7.4	5.6	4.5	3.7	3.0	2.5						12
13	150 × 75 × 5.4*	16.4	103.9	15.4	2.5	12	8.3	6.2	5.0	4.2	3.4	2.8						13
14	175 × 75 × 5.1	17.6	131.3	16.8	2.0	16	10	7.9	6.3	5.3	4.5	3.7						14
15	175 × 75 × 5.7	19.1	139.8	18.8	2.5	17	11	8.4	6.7	5.6	4.8	4.0						15
16	200 × 75 × 5.5	20.6	172.6	20.8	2.0	21	14	13	8.3	6.9	5.9	5.2	3.7					16
17	200 × 75 × 6.1	22.1	181.9	23.0	2.5	22	15	11	8.7	7.3	6.2	5.5	4.1					17
18	225 × 90 × 5.8	24.0	226.5	24.6	2.5	27	18	14	11	9.1	7.8	6.8	5.2					18
19	225 × 80 × 6.4	25.9	239.5	27.2	2.5	29	19	14	11	9.6	8.2	7.2	5.5					19
20	250 × 100 × 6.1	28.0	295.0	28.8	2.5	—	24	18	14	12	10	8.8	7.1	5.4				20
21	250 × 80 × 7.1	30.4	305.3	33.6	2.5	—	24	18	15	12	10	9.2	7.3	5.6				21
22	300 × 100 × 6.7	33.1	403.2	38.0	2.5	—	32	24	19	16	14	12	9.7	8.1	6.2			22
23	300 × 90 × 7.6	35.8	424.2	43.0	2.5	—	34	25	20	17	14	13	10	8.5	6.6	—	—	23
24	350 × 100 × 7.4	38.8	532.1	49.0	2.5	—	43	32	25	21	18	16	13	11	9.1	7.4	6.0	24
25	350 × 100 × 8.1	42.1	571.9	53.6	2.5	—	46	34	27	23	20	17	14	11	9.8	8.0	6.5	25
26	400 × 100 × 8.0	45.7	699.5	60.4	2.5	—	56	42	34	28	24	21	17	14	12	10	8.5	26
27	400 × 100 × 8.6	49.4	754.1	65.0	2.5	—	60	45	36	30	26	23	18	15	13	11	9.4	27

SAFE CONCENTRIC LOADS FOR INDIAN STANDARD STEEL H BEAMS USED AS COLUMNS (NO. 1)

No.	Size of H Beam: Depth × Width × Web Thickness (mm)	Weight per Metre Length (kg)	Radius of Gyration on Y—Y Axis (cm)	"Effective Height" in Metres: 2.0	2.5	3.0	3.5	4.0	4.5	5.0	5.5	6.0	6.5	7.0	7.5	8.0	8.5	9.0	9.5	10.0	11.0	12.0	"EH"	No.
				Safe Loads in Metric Tonnes Based on Y—Y Axis (Least Radius of Gyration)																				
1	150 × 150 × 5.4	27.1	3.54	40	38	35	31	26	22	19	16	13	11	10	8	7	6	—	—	—	—	—	6.0	1
2	150 × 150 × 8.4	30.6	3.44	45	42	38	34	29	24	20	17	14	12	10	9	8	7	—	—	—	—	—	6.0	2
3	150 × 150 × 11.8	34.6	3.35	50	47	43	37	31	26	22	18	15	13	11	9	9	—	—	—	—	—	—	6.0	3
4	200 × 200 × 6.1	37.3	4.51	56	55	53	50	46	42	37	33	29	25	23	19	17	15	—	—	—	—	—	8.0	4
5	200 × 200 × 7.8	40.0	4.42	60	59	56	53	49	44	39	34	30	26	23	20	17	15	13	12	11	8.5	—	7.5	5
6	225 × 225 × 6.5	43.1	4.96	66	64	62	60	57	53	48	43	38	34	30	27	24	21	18	16	15	12	10	8.5	6
7	225 × 225 × 8.6	46.8	4.84	71	70	67	65	61	56	51	46	40	36	32	28	24	21	19	17	15	12	10	8.5	7
8	250 × 250 × 6.9	51.0	5.49	78	77	75	73	70	67	62	57	52	47	42	38	34	30	27	24	22	18	14	9.5	8
9	250 × 250 × 8.8	54.7	5.37	84	82	80	78	75	70	66	60	54	49	44	39	35	31	28	25	22	18	15	9.5	9
10	300 × 250 × 7.6	58.8	5.41	90	89	87	84	80	76	71	65	59	53	48	43	38	34	30	27	24	20	16	9.5	10
11	300 × 250 × 9.4	63.0	5.29	96	95	92	89	85	80	75	68	61	55	49	44	39	35	31	27	25	20	17	9.5	11
12	350 × 250 × 8.3	67.4	5.34	103	102	99	96	92	87	80	74	67	60	53	47	43	38	34	30	27	22	18	9.5	12
13	350 × 250 × 10.1	72.4	5.22	111	109	106	102	98	92	85	77	69	62	55	49	44	39	35	31	28	22	18	9.0	13
14	400 × 250 × 9.1	77.4	5.26	118	117	113	110	105	99	91	83	75	67	60	54	48	42	38	33	30	24	20	9.0	14
15	400 × 250 × 10.6	82.2	5.16	126	123	120	116	110	103	95	86	78	69	62	55	49	43	38	34	31	25	20	9.0	15
16	450 × 250 × 9.8	87.2	5.18	133	131	127	123	117	110	101	92	83	74	68	59	52	46	41	36	33	26	22	9.0	16
17	450 × 250 × 11.3	92.5	5.08	141	139	135	130	123	115	106	95	85	76	68	60	53	47	42	37	33	27	22	9.0	17

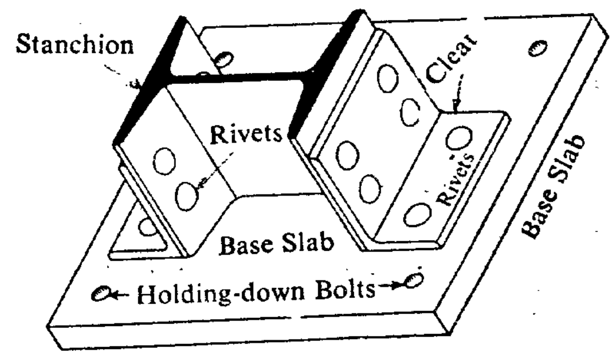


Beam Eccentrically Connected to Stanchion



TYPES OF JOINTS FOR STANCHIONS

STANCHION BASE



SAFE CONCENTRIC LOADS FOR INDIAN STANDARD H BEAMS USED AS COLUMNS (NO. 2)

Size of H Beam: Depth × Width × Web Thickness (mm)	Weight per Metre Length (kg)	Radius of Gyration on x—x Axis (cm)	"Effective Height" in Metres — Safe Loads in Metric Tonnes Based on x—x Axis: 4	5	6	7	8	9	10	11	12
150×150× 5.4	27.1	6.50	39	36	33	28	24	20	16	13	11
150×150× 8.4	30.6	6.29	44	41	36	31	25	21	17	14	12
150×150×11.8	34.6	6.09	49	45	39	33	27	23	18	15	12
200×200× 6.1	37.3	8.71	56	55	52	49	45	41	36	31	27
200×200× 7.8	40.0	8.55	60	58	56	52	48	43	37	33	28
225×225× 6.5	43.1	9.80	66	64	62	60	56	52	48	43	38
225×225× 8.6	46.8	9.58	71	70	67	64	60	56	50	45	40
250×250× 6.9	51.0	10.91	78	77	75	73	70	66	62	57	52
250×250× 8.8	54.7	10.70	84	82	80	78	75	70	65	60	54
300×250× 7.6	58.8	12.95	91	90	89	87	86	82	79	75	71
300×250× 9.4	63.0	12.70	97	96	95	93	90	88	84	80	75
350×250× 8.3	67.4	14.93	105	104	103	102	100	98	95	92	89
350×250×10.1	72.4	14.65	112	111	110	109	107	105	102	98	94
400×250× 9.1	77.4	16.87	120	120	119	118	117	115	114	110	108
400×250×10.6	82.2	16.61	128	127	126	125	123	121	119	117	113
450×250× 9.8	87.2	18.78	136	135	135	134	132	131	129	127	125
450×250×11.3	92.5	18.50	144	144	143	142	140	139	137	135	132

Notes for Columns Tables No. 1 and No. 2.

(i) For "Effective Height" see under "Design of Columns".

(ii) The safe loads tabulated are for ratio of slenderness up to but not exceeding 250.

(iii) "EH" is the "Effective Height" in Table No. 1 beyond which the safe loads indicated are for ratio of slenderness exceeding 180.

(iv) When two beams are placed side by side (which are laced or battened), the radius of gyration on the Y—Y axis will increase depending upon the spacings between the beams and which would be greater than the radius of gyration on the x—x axis even if the beams are abutting together. In that case the radius of gyration given for single beams on x—x axis may be taken for calculating concentrated loads on simple columns.

(v) Safe load figures have been 'rounded off' to the nearest tonne beyond 10 tonnes for practical application.

DATA FOR DOUBLE CHANNELS USED AS COLUMNS

For Single Channels: Size of Channel — Depth × Width × Web Thickness (mm)	Weight per Metre Length (kg)	Sectional Area (cm^2)	Radius of Gyration on x—x Axis (cm)	Radius of Gyration on Y—Y Axis: When abutting back to back (S=o)][(cm)	Radius of Gyration on Y—Y Axis: With flanges butting [] (cm)	Distance S between two channels when $R_{XX}=R_{YY}$ (approx.) (mm)
75× 40×3.7	5.7	7.26	3.02	1.85	2.93	28
100× 45×3.0	5.8	7.41	4.09	1.99	3.41	48
75× 40×4.4	6.8	8.67	2.96	1.78	2.95	27
125× 50×3.0	7.9	10.07	5.18	2.29	3.72	65
100× 50×4.0	7.9	10.02	4.06	2.26	3.73	42
100× 50×4.7	9.2	11.70	4.00	2.13	3.78	45
150× 55×3.6	9.9	12.65	6.10	2.40	4.21	85
125× 65×4.4	10.7	13.67	5.11	2.89	4.91	58
175× 60×3.6	11.2	14.24	7.11	2.57	4.65	105
125× 65×5.0	12.7	16.19	5.07	2.73	4.95	55
200× 70×4.1	13.9	17.77	8.08	2.94	5.48	115
150× 75×4.8	14.4	18.36	6.16	3.36	5.64	68
150× 75×5.4	16.4	20.88	6.11	3.13	5.73	70
175× 75×5.1	17.6	22.40	7.16	3.38	5.63	88
175× 75×5.7	19.1	24.38	7.08	3.13	5.75	90
200× 75×5.5	20.6	26.22	8.11	3.34	5.67	118
200× 75×6.1	22.1	28.21	8.03	3.11	5.78	110
225× 90×5.8	24.0	30.53	9.14	3.59	7.05	155
225× 80×6.4	25.9	33.01	9.03	3.31	6.18	130
250×100×6.1	28.0	35.65	10.17	3.96	7.85	142
250× 80×7.1	30.4	38.67	9.94	3.31	6.18	150
300×100×6.7	33.1	42.11	11.98	3.84	7.98	182
300× 90×7.6	35.8	45.64	11.81	3.52	7.13	185
350×100×7.4	38.8	49.47	13.72	3.71	8.10	222
350×100×8.1	42.1	53.66	13.66	3.74	8.07	215
400×100×8.0	45.7	58.25	15.50	3.67	8.14	258
400×100×8.6	49.4	62.93	15.48	3.73	8.09	255

No. 1. SAFE LOADS FOR SINGLE INDIAN STANDARD CHANNELS USED AS COLUMNS

Size of Channel: Depth × Width × Web Thickness mm	Weight per Metre Length kg	"Effective Height" in Metres: 4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	"EH" metres
		Safe Loads in Metric Tonnes Based on X—X Axis									
75 × 40 × 3.7	5.7	4.4	3.0	2.0	1.4	—					5.0
75 × 40 × 4.4	6.8	5.1	3.4	2.3	1.6	—					5.0
100 × 50 × 4.0	7.9	8.9	6.7	5.1	3.8	2.8	2.2	1.7	—		7.0
100 × 50 × 4.7	9.2	10	7.8	5.8	4.3	3.2	2.5	1.9	—		7.0
125 × 65 × 4.4	10.7	14	12	10	7.9	6.3	4.9	3.9	3.2	2.6	9.0
125 × 65 × 5.0	12.7	17	14	12	9.3	7.3	5.7	4.6	3.7	3.0	9.0
150 × 75 × 4.8	14.4	20	19	17	14	12	9.6	7.8	6.4	5.3	11.0
150 × 75 × 5.4	16.4	23	21	19	16	13	11	8.8	7.1	6.0	10.0
175 × 75 × 5.1	17.6	26	25	23	20	17	15	13	11	8.9	—
175 × 75 × 5.7	19.1	28	27	24	22	19	16	13	11	9.5	—
200 × 75 × 5.5	20.6	31	30	28	26	23	21	18	15	13	—
200 × 75 × 6.1	22.1	33	32	30	28	25	22	19	16	14	—
225 × 90 × 5.8	24.0	36	35	34	32	30	27	24	22	19	—
225 × 80 × 6.4	25.9	39	38	37	35	32	29	26	30	27	—
250 × 100 × 6.1	28.0	43	42	41	39	37	35	32	29	26	—
250 × 80 × 7.1	30.4	46	45	44	42	40	37	34	30	27	—
300 × 100 × 6.7	33.1	51	50	50	48	47	45	43	40	37	—
300 × 90 × 7.6	35.8	55	54	53	52	50	48	46	43	40	—
350 × 100 × 7.4	38.8	60	60	59	58	57	55	53	51	49	—
350 × 100 × 8.1	42.1	65	65	64	63	61	60	58	55	53	—
400 × 100 × 8.0	45.7	71	71	70	69	68	67	65	64	61	—
400 × 100 × 8.6	49.4	77	76	76	75	74	72	70	69	66	—

(i) Safe load figures have been 'rounded off' to the nearest tonne beyond 10 tonnes for practical application.
(ii) The safe loads tabulated are for ratio of slenderness up to but not exceeding 250.
(iii) "EH" is the 'effective height' beyond which the safe loads indicated are for ratio of slenderness exceeding 180.

No. 2. SAFE LOADS FOR SINGLE INDIAN STANDARD CHANNELS USED AS COLUMNS

Size of Channel: Depth × Width × Web Thickness mm	Weight per Metre Length kg	"Effective Height" in Metres: 2.0	2.5	3.0	3.5	4.0	4.5	5.0	5.5	6.0	"EH" metres
		Safe Loads in Metric Tonnes Based on Y—Y Axis									
75 × 40 × 3.7	5.7	3.2	2.0	1.3	—						2.0
75 × 40 × 4.4	6.8	3.6	2.2	1.5	—						2.0
100 × 50 × 4.0	7.9	6.5	4.4	3.0	2.1	—					2.5
100 × 50 × 4.7	9.2	7.0	4.7	3.2	2.2	—	—				2.5
125 × 65 × 4.4	10.7	12	9.5	7.1	5.2	4.0	3.0	2.4	—		3.5
125 × 65 × 5.0	12.7	14	10	7.5	5.4	4.0	3.1	—	—		3.0
150 × 75 × 4.8	14.4	18	15	12	9.4	7.2	5.6	4.4	3.6	—	4.0
150 × 75 × 5.4	16.4	20	16	12	9.3	7.0	5.5	4.3	3.5	—	3.5
175 × 75 × 5.1	17.6	23	19	15	12	8.9	6.9	5.5	4.4	—	4.0
175 × 75 × 5.7	19.1	24	19	15	11	8.4	6.5	5.2	4.1	—	4.0
200 × 75 × 5.5	20.6	26	22	17	13	10	8.0	6.3	5.1	—	4.0
200 × 75 × 6.1	22.1	27	22	17	13	9.7	7.6	6.0	4.8	—	4.0
225 × 90 × 5.8	24.0	32	28	23	18	15	12	9.3	7.4	6.2	4.5
225 × 80 × 6.4	25.9	33	28	22	17	13	10	8.0	6.5	—	4.0
250 × 100 × 6.1	28.0	39	35	30	25	20	16	13	11	9.0	5.0
250 × 80 × 7.1	30.4	39	32	26	20	15	12	9.4	7.6	—	4.0
300 × 100 × 6.7	33.1	46	42	35	29	24	19	15	13	10	5.0
300 × 90 × 7.6	35.8	48	42	34	27	22	17	14	11	9.1	4.5
350 × 100 × 7.4	38.8	54	48	41	33	27	22	17	14	12	5.0
350 × 100 × 8.1	42.1	59	52	45	36	30	24	19	16	13	5.0
400 × 100 × 8.0	45.7	64	57	48	39	32	25	20	17	14	5.0
400 × 100 × 8.6	49.4	69	62	52	43	35	28	22	18	15	5.0

(i) Safe load figures have been 'rounded off' to the nearest tonne beyond 10 tonnes for practical application
(ii) The safe loads tabulated are for ratio of slenderness up to but not exceeding 250.
(iii) "EH" is the "effective height" beyond which the safe loads indicated are for ratio of slenderness exceeding 180.

SAFE LOADS FOR INDIAN STANDARD SINGLE ANGLES USED AS STRUTS

Size of Angle Leg × Leg × Thickness mm	Weight per Metre Length Single Angle kg	Single Angle L Struts — Effective Length in Metres — Safe Loads in Metric Tonnes						"EL" m
		1.0	1.5	2.0	2.5	3.0	3.5	
50 × 50 × 4	3.0	3.3	1.8	1.0	—	—		1.5
60 × 40 × 5	3.7	3.5	1.7	0.9	—	—		1.5
50 × 50 × 5	3.8	4.1	2.2	1.2	—	—		1.5
55 × 55 × 5	4.1	4.9	2.9	1.6	1.0	—		
65 × 45 × 5	4.1	4.4	2.4	1.3	—	—		1.5
70 × 45 × 5	4.3	4.7	2.5	1.4	—	—		1.5
60 × 40 × 6	4.4	4.1	2.0	1.1	—	—		1.5
50 × 50 × 6	4.5	4.8	2.6	1.4	—	—		1.5
60 × 60 × 5	4.5	5.7	3.6	2.2	1.3	—		2.0
55 × 55 × 6	4.9	5.8	3.4	1.9	1.2	—		1.5
65 × 45 × 6	4.9	5.2	2.8	1.5	—	—		1.5
65 × 65 × 5	4.9	6.5	4.5	2.8	1.7	1.1		2.0
70 × 45 × 6	5.2	5.6	3.0	1.6	—	—		1.5
70 × 70 × 5	5.3	7.3	5.4	3.5	2.2	1.5		2.0
60 × 60 × 6	5.4	6.8	4.3	2.5	1.5	—		2.0
75 × 50 × 6	5.6	6.7	4.0	2.3	1.4	—		1.5
60 × 40 × 8	5.8	5.3	2.6	1.4	—	—		1.5
65 × 65 × 6	5.8	7.8	5.3	3.3	2.1	1.4		2.0
80 × 50 × 6	5.9	7.0	4.2	2.4	1.4	—		1.5
70 × 70 × 6	6.3	8.7	6.4	4.2	2.6	1.8		2.0
55 × 55 × 8	6.4	7.6	4.5	2.5	1.5	—		1.5
65 × 45 × 8	6.4	6.8	3.7	2.0	—	—	—	1.5
70 × 45 × 8	6.7	7.2	3.9	2.1	—	—	—	1.5
75 × 75 × 6	6.8	9.6	7.4	5.0	3.3	2.2	1.6	2.5
60 × 60 × 8	7.0	8.9	5.6	3.3	2.0	—	—	2.0
80 × 80 × 6	7.3	10	8.5	6.0	4.1	2.8	2.0	2.5
75 × 50 × 8	7.4	8.7	5.2	2.9	1.8	—	—	1.5
65 × 65 × 8	7.7	10	6.9	4.3	2.7	1.8	—	2.0
80 × 50 × 8	7.7	9.1	5.4	3.0	1.8	—	—	1.5
55 × 55 × 10	7.9	9.3	5.5	3.1	1.9	—	—	1.5
70 × 70 × 8	8.3	11	8.3	5.4	3.4	2.3	—	2.0
70 × 45 × 10	8.3	8.8	4.7	2.5	—	—	—	1.5
60 × 60 × 10	8.6	11	6.9	4.0	2.5	[illegible]	—	2.0
75 × 75 × 8	8.9	12	9.7	6.5	4.3	2.9	2.0	2.5
90 × 60 × 8	8.9	12	8.3	5.2	3.3	2.2	—	2.0
75 × 50 × 10	9.0	11	6.3	3.6	2.2	—	—	1.5
65 × 65 × 10	9.4	12	8.5	5.3	3.3	2.2	—	2.0

SAFE LOADS FOR INDIAN STANDARD SINGLE ANGLES USED AS STRUTS

Size of Angle Leg × Leg × Thickness mm	Weight per Metre Length Single Angle kg	Single Angle Struts — Effective Length in Metres — Safe Loads in Metric Tonnes								"EL" m
		1.0	1.5	2.0	2.5	3.0	3.5	4.0	4.5	
80 × 50 × 10	9.4	11	6.6	3.7	2.3	—	—	—	—	1.5
80 × 80 × 8	9.6	14	11	7.8	5.3	3.6	2.5	—	—	2.5
100 × 65 × 8	9.9	14	10	6.7	4.3	2.9	—	—	—	2.5
70 × 70 × 10	10.2	14	10	6.6	4.2	2.8	—	—	—	2.0
100 × 75 × 8	10.5	15	12	8.9	6.1	4.1	2.9	—	—	2.5
90 × 90 × 8	10.8	—	14	10	7.5	5.2	3.8	—	—	3.0
75 × 75 × 10	11.0	15	12	8.0	5.3	3.5	2.5	—	—	2.5
90 × 60 × 10	11.0	15	10	6.3	4.0	2.6	—	—	—	2.0
80 × 80 × 10	11.8	17	14	9.6	6.5	4.4	3.1	—	—	2.5
100 × 100 × 8	12.1	—	16	13	9.9	7.3	5.3	4.0	3.0	3.5
100 × 65 × 10	12.2	17	12	8.2	5.2	3.5	—	—	—	2.5
90 × 60 × 12	13.0	17	12	7.5	4.7	3.1	—	—	—	2.0
100 × 75 × 10	13.0	19	15	11	7.4	5.1	3.6	—	—	2.5
90 × 90 × 10	13.4	—	17	13	9.1	6.4	4.6	3.4	—	3.0
80 × 80 × 12	14.0	20	16	11	7.6	5.2	3.7	—	—	2.5
100 × 100 × 10	14.9	—	20	16	12	9.0	6.4	4.9	3.7	3.0
100 × 75 × 12	15.4	22	18	13	8.8	6.0	4.2	—	—	2.5
90 × 90 × 12	15.8	—	20	15	11	7.6	5.5	4.0	—	3.0
100 × 100 × 12	17.7	—	24	19	14	11	7.6	5.8	4.4	3.0

CORRUGATED GALVANIZED IRON SHEETS

The following table gives the spans the sheets will carry for a load of 110 kg/sq. metre

Gauge No.	16	18	20	22	24	26	
Length in metres (without laps)	2.2	2.1	1.8	1.6	1.4	1.2	Cantilevers will be only 1/4 of these lengths.
With end lap of 23 cm and riveted	2.7	2.4	2.0	1.8	1.6	1.4	

SAFE LOADS FOR INDIAN STANDARD DOUBLE ANGLES USED AS STRUTS

Size of Angle Leg × Leg × Thickness	Weight per Metre Length Single Angle	Double Angle ⌉⌈ Struts (Back to Back)											"EL"	Axis
		Effective Lengths in Metres												
		1.0	1.5	2.0	2.5	3.0	3.5	4.0	4.5	5.0	5.5	6.0		
mm	kg	Safe Loads in Metric Tonnes											m	
50×50×4	3.0	0.7	7.0	4.8	3.3	2.2	1.6						2.5	x—x
50×50×5	3.8	11	8.6	5.9	4.0	2.7	1.9						2.5	x—x
55×55×5	4.1	12	10	7.5	5.3	3.6	2.6	1.9					3.0	x—x
65×45×5	4.1	12	11	8.7	6.4	4.6	3.3	2.5	1.9				3.0	y—y
70×45×5	4.3	12	11	8.9	6.5	4.7	3.4	2.5	1.9	—			3.0	y—y
50×50×6	4.5	13	10	7.0	4.6	3.2	2.2	—	—	—			2.5	x—x
60×60×5	4.5	13	12	9.1	6.6	4.7	3.4	2.5					3.0	x—x
75×50×5	4.7	14	13	11	8.3	6.2	4.6	3.4	2.6	—			3.5	y—y
55×55×6	4.9	14	12	8.8	6.2	4.2	3.0	2.2					2.5	x—x
65×45×6	4.9	15	13	10	7.8	5.7	4.1	3.1	2.4	—			3.0	y—y
80×50×5	4.9	15	13	11	8.3	6.2	4.5	3.4	2.6				3.5	y—y
65×65×5	4.9	15	13	11	8.3	6.2	4.5	3.4	—				3.5	x—x
70×45×6	5.2	15	14	11	7.9	5.7	4.1	3.1	2.4	—			3.0	y—y
70×70×5	5.3	—	—	13	10	7.6	5.7	4.3	3.3	2.6	--		3.5	x—x
60×60×6	5.4	16	14	11	7.9	5.6	4.1	3.0					3.0	x—x
75×50×6	5.6	17	15	13	10	7.5	5.5	4.2	3.2				3.5	y—y
75×75×5	5.7	—	—	14	12	9.2	7.1	5.4	4.2	3.3	2.7	—	4.0	x—x
65×65×6	5.8	17	16	13	9.4	7.3	5.3	4.0	—	—	—	—	3.5	x—x
80×50×6	5.9	18	16	13	10	7.6	5.5	4.2	3.2				3.5	y—y
70×70×6	6.3	—	—	15	12	9.0	6.8	5.1	3.9	3.1			3.5	x—x
55×55×8	6.4	19	16	11	7.9	5.4	3.9	2.9					2.5	x—x
65×45×8	6.4	19	17	14	10	7.8	5.6	4.2	3.2				3.5	y—y
70×45×8	6.7	20	18	14	11	7.8	5.6	4.2	3.2	—			3.0	y—y

Size	Weight	1.0	1.5	2.0	2.5	3.0	3.5	4.0	4.5	5.0	5.5	6.0	"EL"	Axis
75×75×6	6.8	—	—	17	14	11	8.4	6.4	5.0	3.9	3.1	—	4.0	x—x
90×60×6	6.8	—	—	18	15	12	9.1	7	5.4	4.3	3.5	2.8	4.0	y—y
60×60×8	7.0	21	18	14	10	7.2	5.2	3.9	—	—	—	—	3.0	x—x
80×80×6	7.3	—	—	19	16	13	10	7.9	6.1	4.9	4.0	3.2	4.0	x—x
75×50×8	7.4	22	20	17	13	10	7.6	5.8	4.4	—	—	—	3.5	y—y
100×65×6	7.5	—	—	20	17	14	11	8.7	6.8	5.5	4.4	3.6	4.5	y—y
65×65×8	7.7	23	21	17	13	9.4	6.8	5.1	—	—	—	—	3.5	x—x
80×50×8	7.7	23	21	18	14	10	7.6	5.7	4.4	—	—	—	3.5	y—y
55×55×10	7.9	23	19	14	9.5	6.5	4.6	3.4	—	—	—	—	2.5	x—x
100×75×6	8.0	—	—	23	21	18	15	12	10	8.3	6.7	5.6	5.0	y—y
90×90×6	8.2	—	—	23	20	17	14	11	8.9	7.1	5.8	4.8	4.5	x—x
70×70×8	8.3	—	—	20	15	12	8.7	6.6	5.0	4.0	—	—	3.5	x—x
70×45×10	8.3	25	22	18	14	10	7.3	5.5	4.2	—	—	—	3.5	y—y
60×60×10	8.6	25	22	17	12	8.7	6.3	4.6	—	—	—	—	3.0	x—x
75×75×8	8.9	—	—	22	18	14	11	8.2	6.4	5.0	4.1	—	4.0	x—x
90×60×8	8.9	—	—	23	20	16	12	9.6	7.4	6.0	4.8	3.9	4.0	y—y
75×50×10	9.0	27	25	22	17	13	9.8	7.4	5.7	—	—	—	3.5	y—y
65×65×10	9.4	28	25	21	15	11	8.1	6.1	—	—	—	—	3.5	x—x
80×50×10	9.4	28	26	22	17	13	9.8	7.4	5.7	—	—	—	3.5	y—y
80×80×8	9.6	—	—	25	21	17	13	10	7.9	6.3	5.1	4.2	4.0	x—x
100×65×8	9.9	—	—	27	23	19	15	12	9.4	7.5	6.0	5.0	4.5	y—y
70×70×10	10.2	—	—	24	19	14	10	7.9	6.1	4.8	—		3.5	x—x
100×75×8	10.5	—	—	30	27	24	20	17	14	11	9.2	7.7	5.5	y—y
90×90×8	10.8	—	—	30	26	22	18	14	12	9.2	7.6	6.2	4.5	x—x
75×75×10	11.0	—	—	27	22	17	13	9.9	7.8	6.1	4.9	—	4.0	x—x
90×60×10	11.0	—	—	29	25	20	16	12	9.5	7.6	6.1	5.0	4.0	y—y

Size	Weight	2.0	2.5	3.0	3.5	4.0	4.5	5.0	5.5	6.0	"EL"	Axis
80×80×10	11.8	31	26	20	16	12	9.5	7.6	6.1	5.1	4.0	x—x
100×100×8	12.1	34	32	28	23	19	16	13	11	8.9	5.5	x—x
100×65×10	12.2	33	29	24	19	15	12	9.6	7.7	6.4	4.5	y—y
90×60×12	13.0	35	30	24	19	15	12	9.4	7.6	6.2	4.5	y—y
100×75×10	13.0	37	34	30	26	21	18	14	12	9.8	5.5	y—y
90×90×10	13.4	37	32	27	22	18	14	11	9.2	7.5	4.5	x—x
90×80×12	14.0	36	30	24	18	14	11	8.8	7.1	—	4.0	x—x
100×100×10	14.9	42	39	34	29	24	20	16	13	11	5.0	x—x
100×75×12	15.4	44	41	36	31	26	21	18	14	12	5.5	y—y
90×90×12	15.8	43	38	32	26	21	16	13	11	8.8	4.5	x—x
100×100×12	17.7	50	46	40	34	28	23	19	15	13	5.0	x—x

Notes for Struts:

(i) Safe load figures have been rounded off to the nearest tonne beyond 10 tonnes for ease of practical application.

(ii) Tabulated loads are for ratio of slenderness up to but not exceeding 250.

(iii) "EL" is the effective span beyond which ratio of slenderness exceeds 180.

(iv) Safe loads are on the basis of full unit stress since the sections are more often used as bracings and as secondary members than as main members.

(v) Single angle struts are double riveted or double bolted or with welded end connections. Where struts are single bolted or are with single riveted end connections (in case of small sections), only 80% of the tabulated loads shall be allowed.

(vi) For double angle struts back to back connected to both sides of a gusset or a member by a single rivet or bolt, only 80% of the tabulated loads shall be allowed.

(vii) In double unequal angles longer legs are connected back to back. (Shorter legs connected take lesser loads).

(viii) Space between back to back of double angle struts has been taken as 6 mm.

(ix) Effective length of a strut is taken as centre-to-centre of fastening at each end.

(x) Radius of gyration on x – x axis of two angles back to back is the same as for single angle on x—x axis.

(xi) When two angles are placed heel to heel (starred) the least radius of gyration for both angles is the same as for single angle on u – u axis.

(xii) The least allowable safe loads have been taken whether on the x – x or on Y—Y axis.

ROUND & SQUARE STEEL BARS

Areas & Weights

Dia. or Side		Round Bars			Square Bars
		Weight per Metre Length	Cross Sectional Area	Circumference	Weight per Metre Length
mm	in	kg	cm^2	cm	kg
5	0.20	0.154	0.200	1.57	0.20
6	0.24	0.222	0.283	1.88	0.28
8	0.31	0.395	0.503	2.51	0.50
10	0.39	0.620	0.785	3.14	0.78
12	0.47	0.888	1.131	3.77	1.13
14	0.55	1.208	1.539	4.40	1.54
16	0.63	1.578	2.010	5.03	2.01
18	0.71	2.000	2.545	5.65	2.54
20	0.79	2.465	3.142	6.28	3.14
22	0.87	2.983	3.801	6.91	3.80
25	0.98	3.852	4.909	7.85	4.91
28	1.10	4.832	6.158	8.80	6.15
32	1.26	6.311	8.042	10.05	8.04
36	1.42	7.990	10.180	11.31	10.17
40	1.57	9.860	12.566	12.57	12.56
45	1.77	12.49	15.904	14.14	15.90
50	1.97	15.41	19.635	15.71	19.62
56	2.20	19.34	24.630	17.59	24.62
63	2.48	24.47	31.172	19.79	31.16

The weight of steel is taken at 7.85 grams/cu. cm or 0.785 kg/ sq. cm per metre length.

Deformed Bars

Twin Twisted

Ribbed Torsteel

Square Twisted

SAFE LOADS FOR INDIAN STANDARD ANGLES USED AS TIES

Size of Angle: Leg × Leg × Thickness	Weight per Metre Length Single Angle	Single Angle Ties: Equal Angles: Connected by one leg	Single Angle Ties: Unequal Angles: Shorter leg Connected	Single Angle Ties: Unequal Angles: Longer Leg Connected	Double Angle Ties: Equal Angles: Connected by one leg	Double Angle Ties: Unequal Angles: Shorter leg Connected	Double Angle Ties: Unequal Angles: Longer leg Connected
mm	kg	Safe Loads (Tensile) in Metric Tonnes					
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
50× 30× 3	1.8	—	2.5	2.3	—	6.6	5.2
50× 50× 3	2.3	2.7	—	—	6.9	—	—
50× 30× 4	2.4	—	3.3	3.0	—	8.7	6.7
50× 30× 5	3.0	—	4.1	3.6	—	11	8.3
50× 50× 4	3.0	3.5	—	—	9.0	—	—
50× 30× 6	3.5	—	4.8	4.3	—	13	9.7
60× 40× 5	3.7	—	5.3	4.5	—	13	10
50× 50× 5	3.8	4.4	—	—	11	—	—
55× 55× 5	4.1	4.6	—	—	12	—	—
65× 45× 5	4.1	—	6.0	5.1	—	15	12
70× 45× 5	4.3	—	6.1	5.6	—	16	13
60× 40× 6	4.4	—	6.3	5.3	—	16	12
50× 50× 6	4.5	5.1	—	—	13	—	—
60× 60× 5	4.5	5.2	—	—	13	—	—
75× 50× 5	4.7	—	6.8	6.2	—	17	14
55× 55× 6	4.9	5.4	—	—	14	—	—
65× 45× 6	4.9	—	7.1	6.1	—	18	14
80× 50× 5	4.9	—	6.9	6.6	—	18	15
65× 65× 5	4.9	5.8	—	—	15	—	—
70× 45× 6	5.2	—	7.3	6.6	—	19	15
70× 70× 5	5.3	6.5	—	—	16	—	—
60× 60× 6	5.4	6.1	—	—	16	—	—
75× 50× 6	5.6	—	5.8	7.3	—	20	17
75× 75× 5	5.7	7.1	—	—	18	—	—
60× 40× 8	5.8	—	8.2	6.9	—	21	16
65× 65× 6	5.8	6.9	—	—	17	—	—
80× 50× 6	5.9	—	6.0	7.8	—	18	17
70× 70× 6	6.3	7.7	—	—	19	—	—
55× 55× 8	6.4	7.0	—	—	18	—	—
65× 45× 8	6.4	—	9.2	7.9	—	23	18
70× 45× 8	6.7	—	8.7	6.6	—	24	19
75× 75× 6	6.8	8.4	—	—	21	—	—
90× 60× 6	6.8	—	7.0	9.3	—	25	21
60× 60× 8	7.0	8.4	—	—	21	—	—

SAFE LOADS FOR INDIAN STANDARD ANGLES USED AS TIES

mm	kg	Safe Loads (Tensile) in Metric Tonnes					
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
80× 80× 6	7.3	9.2	—	—	23	—	—
75× 50× 8	7.4	—	7.6	9.6	—	23	22
100× 65× 6	7.5	—	8.0	10	—	23	22
65× 65× 8	7.7	9.0	—	—	23	—	—
80× 50× 8	7.7	—	7.7	10	—	24	23
55× 55×10	7.9	8.5	—		22	—	—
100× 75× 6	8.0	—	9.2	10	—	29	24
90× 90× 6	8.2	11	—	—	26	—	—
70× 70× 8	8.3	10	—	—	25	—	—
70× 45×10	8.3	—	12	10	—	30	24
60× 60×10	8.0	10	—	—	25	—	—
75× 75× 8	8.9	11	—	—	27	—	—
90× 60× 8	8.9	—	9.2	12	—	27	27
75× 50×10	9.0	10	9.2	12	—	28	27
65× 65×10	9.4	11	—	—	28	—	—
80× 50×10	9.4	—	9.4	13	—	29	28
80× 80× 8	9.6	12	—	—	30	—	—
100× 65× 8	9.9	—	10	13	—	31	30
70× 70×10	10.2	12	—	—	31	—	—
100× 75× 8	10.5	—	12	14	—	33	32
90× 90× 8	10.8	14	—	—	34		—
75× 75×10	11.0	13	—	—	34	—	—
90× 60×10	11.0	—	11	15	—	34	34
80× 80×10	11.8	15	—	—	37	—	—
100×100× 8	12.1	15	—	—	38	—	—
100× 65×10	12.2	—	13	16	—	38	36
90× 60×12	13.0	—	13	18	—	40	40
100× 75×10	13.0	—	15	17	—	41	39
90× 90×10	13.4	17	—	—	42	—	—
80× 80×12	14.0	17	—	—	43	—	—
100×100×10	14.9	9	—	—	46	—	—
100× 75×12	15.4	—	—	20	—	48	46
90× 90×12	15.8	20	—	—	50	—	—
100×100×12	17.7	22	—	—	55	—	—

(i) Safe allowable loads have been rounded off to the nearest tonne for practical application

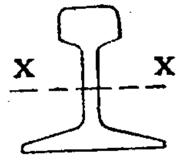
(ii) All angles are connected by one leg; longer leg connected of unequal angles.

(iii) Double angles are fixed on both sides of the gusset. Where fixed on one side, the allowable loads shall be only 90% of the tabulated loads.

(iv) Rivet hole/holes have been deducted from the gross areas to arrive at the allowable loads.

29. RAILS

Properties of Flat Bottom Rails used on Indian Railways



Bottom width inches	Weight of Rail in lbs. per yard	Distance from top of Rail to N.A. inches	Section Moduli about X X	Moment of Inertia about X X	Radii of Gyration about X X	Total depth inches
5.000	100 FF	3.03	15.40	46.70	2.18	6.000
5.375	90 R	2.95	13.05	38.45	2.09	5.625
5.375	90 BSS FF	2.80	12.33	34.51	1.98	5.375
5.000	87 F F	2.59	11.50	29.72	1.86	5.150
2.375	84 BH	2.87	9.50	27.27	1.85	5.469
2.500	85 BH	2.94	10.46	30.70	1.91	5.550
2.406	78 DH	2.72	10.09	27.47	1.92	5.437
4.812	75 R	2.61	9.72	25.36	1.86	5.062
4.812	75 BSS FF	2.49	9.23	22.97	1.77	4.812
2.375	75 D H	2.60	8.63	22.45	1.73	5.187
4.000	74 F F	2.52	8.16	20.56	1.67	4.750
4.500	70 F F	2.44	7.46	18.21	1.65	4.562
2.437	68 D H	2.50	7.86	19.66	1.71	5.000
2.125	68 B H	2.91	7.63	22.19	1.81	5.343
2.250	64 D H	2.50	7.32	18.30	1.73	5.000
4.000	62 F F	2.32	6.91	16.03	1.62	4.500
4.312	60 R	2.31	7.04	16.26	1.66	4.500
4.312	60 BSS	2.25	5.77	12.98	1.48	4.312
4.000	60 F F	2.13	6.57	13.99	1.54	4.250
3.875	$58\frac{1}{2}$ F F	2.20	5.56	12.22	1.46	4.312
3.937	50 BSS	1.97	5.13	10.10	1.44	3.937
3.500	50 F F	2.00	5.03	10.06	1.43	4.000
3.500	40 BSS	1.75	3.74	6.55	1.29	3.500
3.250	35 BSS	1.62	3.04	4.94	1.20	3.250
3.105	35 F F	1.67	3.06	5.09	1.21	3.307
3.000	30 BSS	1.50	2.39	2.59	1.11	3.000

For old Rails the Moment of Inertia and Section Modulus should be reduced by $12\frac{1}{2}$ per cent. where proposed to be used as beams. Min. Radius of Gyration at right angle to XX is about $\frac{1}{3}$ to $\frac{1}{2}$.

SECTION 5

PROPERTIES & USES OF METALS

1. DEFINITIONS OF TERMS

Alloy : Two or more metals mixed in certain determined proportions while in molten state form an alloy.

Alloy Steels : Steels to which certain elements (metals) not existing in carbon steel have been added.

Annealing : A process of softening steel by heating it to a suitable temperature and then cooling slowly.

Blow-holes : Gas cavaties in a casting.

Brittleness : Tendency to break or give way under an impact of load, and is the opposite of toughness.

Carbon : Either as charcoal or graphite, is a soft brittle snbstance, easily powdered.

Carbon Steels : Ordinary steels.

Coke : Purer form of coal and contains a majority of carbon, is produced from coal artificially.

Coke Oven : A sort of a distiller which heats the coal out of contact with air and produces coke.

Cold-Drawing . Method of producing a bar or wire by drawing or stretching mechanically the metal through a die without heating the material, thus reducing its cross-section. This improves physical properties of the metal, increasing tensile strength, yield point, hardness, and resistance to fatigue ; gives superior qualities to hot rolled process.

Cold-Rolling : Has about the same effects on the structure and physical properties of steel and most of the non-ferrous metals as cold-drawing, the extent of changes depending upon the amount of reduction in cross-section. Before steel is cold-rolled, it must be annealed. Cold-rolling gives clean and smooth finished surface.

Creep : The slow stretching or continuous plastic extension under steady stress (particularly at high temperature).

Critical Temperature : The temperature at which the elements of steel become fluid.

Drawing (of wires) : The pullying of round sections or wires through holes (dies) of successively smaller diametres, which elongates lengthwise and proportionately reduces the diameter.

Ductility : Is the property of (a metal of) being permanently extended or stretched (*i.e.*, drawn into wires) by a tensile force to a smaller section, before it fractures. Ductility is opposite to brittleness. *Ductile* thus means able to stretch before it gives way.

Elasticity : Is the property of a solid material to return to its original size and shape on removal of the applied load provided the load has not exceeded a certain limit called the *elastic limit.*

Elastic Limit : The point at which a metal ceases to be elastic under

increasing tensile stress and does not spring back to its original length but remains stretched, *i.e.*, the deformation for elongation produced as a result of the load stays on when it has passed its elastic limit. Steel is to a certain degree elastic.

When tension is applied to steel it elongates slightly, the elongation being proportional to the load applied up to the elastic limit after which elongation increases at a much greater rate than the rate of increase of the load. When steel is loaded beyond its elastic limit, it continues to stretch, but if the load is taken off, it will be found to have acquired a certain amount of (only slightly) permanent elongation.

Forging : Moulding a hot piece of metal to a different shape by hammering.

Graphite : Free or uncombined carbon (present in cast iron).

Hardness : The resistance of a metal to penetration, abrasion or wear. Hardness of a metal can be measured by several instruments.

Hardening : Process of increasing the hardness of a metal which is done by first heating the metal to a red heat and then suddenly cooling it by quenching in cold water or oil.

Hard-drawn : Is the temper produced in a wire, rod or tube by cold-drawing.

High Tensile Steels : Steels in which enhanced mechanical properties and increased resistance to corrosion has been obtained by the incorporation of some alloying elements.

Ingots : Castings of uniform sizes and shapes made of molten steel for subsequent rolling, forging or processing.

Iron : Pure iron is a soft, malleable and ductile metal to which carbon is added to make it useful for engineering purposes.

Malleability : Property of being permanently expanded or flattened into sheets without fracture when rolled or hammered (usually cold). The more malleable the metal, the thinner the sheets into which it can be formed.

Milling : Different operations such as rolling, drawing, forging, pressing, are called milling. The milling operations improve the quality of steel and make it more ductile.

Mild Steel : Steel of low carbon content.

Metallurgy : Art of extracting metals from their ores and their subsequent adaptation for use for engineering purposes after refining.

Ores : Surface or underground deposits of metals which are in combination with earthy impurities and have to be obtained by quarrying or mining, and from which metals are extracted.

Pig Iron : Is the raw iron obtained after heating iron ores in blast furnances, and which is further refined to obtain useful irons.

Pickling : Removing scale from steel by immersion in a dilluted acid bath.

Plasticity : Is the property of a metal to undergo large deformations when stressed without rupture. In a plastic deformation, the material does not return to its original shape.

Processing : Is another name for annealing.

Proof Stress : Stress which is just sufficient to cause a specified permanent deformation or elongation of a material (yield point).

Quenching : Rapid cooling of heated metals by immersion either in liquids (water, oil, salt solutions), gases, solids (molten) or in air.

Resilience : Is the property of a solid whereby it will sustain shock load without permanent deformation. Resilience test is a test for strength and ductility.

Rolling : Castings of iron (ingots) are passed through rolling machines to shape them into different steel sections such as I, L, T, rounds, flats and sheets.

Smelting : The process of heating iron ores in blast furnaces for refining and manufacture of pig iron.

Special Steels : Alloy steels are called special steels.

Slag : Waste product obtained from blast and cupola furnaces. It is crushed after cooling and used as coarse aggregate for concrete, road metal or ballast for rail roads.

Stiffness : Is the ability of a material to resist deformation. It is a measure of the modulus of elasticity, the higher the modulus, the stiffer the material.

Tenacity : Is the property of a material to resist a tensile force without giving way.

Tensile Strength : Is the resistance of material to a pullying stress.

Toughness : Is the resistance of a material to fracture by bending, twisting, fatigue or impact of load ; opposite to brittleness. Greater the hardness the less will be the toughness. Toughening a material by tempering or annealing decreases tensile strength and hardness. Toughness enables materials to undergo relatively large deformations at high stress. Deflection of a test-piece under load gives some indication of the toughness of a material. This factor depends upon the chemical composition and the heat treatment given to a material.

Work-hardening : Process of increasing the hardness and strength of materials by plastic deformation.

Yield Point : Is the point where the elongation of a bar under tension increases without increase of load. *Yield Stress* is the lowest stress in tension at which yield point is reached. When a load causing stress less than yield stress is removed, the steel contracts almost to its original length, but when yield stress is exceeded, the deformation remains after removal of the stress load. Yield point is well defined in mild steel but ill-defined or absent in other metals. A mild steel specimen when tested in tension suffers considerable plastic flow prior to fracture.

Metals used by engineers are divided into two classes, ferrous and non-ferrous or non-rusting metals. The ferrous metal iron is by far the most

important metal used in engineering construction. Other metals used are copper, lead and zinc which may be considered of primary importance while aluminium, nickel, tin, chromium, tungsten, antimony, manganese, and vanadium may be considered of secondary importance.

The important properties of metals for construction purposes are elasticity, tenacity, malleability, ductility, density, hardness, fusibility, corrodibility, electrical and thermal conductivity, expansibility and magnetic qualities etc.

2. PRODUCTION AND GENERAL PROPERTIES OF IRON

Metals do not occur in nature in a pure metallic state but in combination with other earthy impurities, called *ores*, generally in the forms of oxides, sulphides and carbonates, etc. iron occurs in various forms mixed with stone and other impurities either as surface or underground deposits which have to be obtained by quarrying or mining. There are various types of iron ores, the most important being : Red Haematite (or Hematite) which is deep red in colour and an oxide of iron containing about 70 per cent of iron (with impurities) and is considered to be the most valuable iron-producing ore. Brown Haematite or Limonite is brown in colour and yields about 60 per cent of metallic iron. Magnetite is a black oxide of iron and when pure yields about 55 to 72 per cent of metallic iron. Spathic iron ore or Siderite is grey or brown iron carbonate which yields about 48 per cent of metallic iron. Iron ores are found in Assam, Bengal, Bihar, Orissa, Karnatka, and some other parts of India, and are of different qualities.

Manufacture of Iron : Iron ores are heated in a special type of furnace called Blast Furnace,* under intense heat (1600-1800 deg. C.), which is called *smelting*. The metallic iron melts out to a liquid form from the crude iron ore which when solidified is called *Pig Iron*. The impurities left after the formation of pig iron is called "slag". Pig iron is the raw material consisting of a combination of pure iron with carbon and some other substances from which cast iron, wrought iron and steel are manufactured by various rofining processes so that only the requisite quantity of carbon and other elements remain. The pig iron from blast furnace contains about 90 to 92 per cent of iron. Chemically pure iron is not an article of commerce and carbon has to be added to it to make it useful for engineering purposes.

Pig Irons which are cast irons are classified into various grades and qualities according to the carbon contents and other elements. The usual classifications are : grey, white and mottled pig irons. Grey pig iron is a soft variety of pig iron and is more suitable for foundry casting works. White pig iron is closely grained, hard and strong and can be easily melted and is used for making wrought iron. Mottled pig iron has a grandular and mottled appearance on fracture and is used for heavy foundry works.

Carbon in Iron : Carbon, which is considered as an "alloy", plays a very important part on iron. Carbon affects the properties of iron consider-

* In the modern iron manufacturing works, the main blast furnace is a huge shaft over 36 m in height and about 7.5 m in diametre near the hearth at the bottom.

ably modifying its melting point, strength, hardness and properties of becoming hard by heat treatment. The higher the percentage of carbon the lower is the melting point which is between 1535 deg. C. for pure iron free from carbon to 1130 deg. C. for 1.8 per cent of carbon steel.

Cast iron contains a high percentage of carbon, about 1.5 to 6.5 per cent ; steel contains carbon up to a maximum of 1.5 per cent. Steel containing up to 0.5 per cent of carbon is termed "mild steel". Wrought iron when pure is nearly free from carbon and may contain only 0.05 to 0.15 per cent.

Classes of Irons. There are three general classes : Cast iron, Wrought iron, and Steel. The great difference in mechanical properties and characteristics that exist between the different irons depend chiefly upon the amount and nature of the carbon they contain. As soon as carbon is introduced into iron, its tensile strength begins to rise up to about one per cent of carbon and the hardness also increases. There is no hard and fast line of demarcation of the percentage of carbon between the different types of irons as the properties also depend upon the conditions in which this carbon exists and some other factors as well.

Simple Field Tests to Distinguish Different Irons : Cast iron can be readily distinguished from wrought iron by its crystalline structure, by its want of ductility and its brittleness. If a piece of cast iron is struck with a hammer it breaks clean without bending and the fractured surface presents large grains of whitish gray and dark colour with black spots interspersed, while wrought iron when struck bends and then breaks and the broken surface presents an uneven thick fibrous oppearance.

Heat and Bending Tests : Cast iron cannot be bent when cold while wrought iron can be bent very easily and steel can be bent with some difficulty. Cast iron breaks very quickly when heated while steel can be bent easily when hot.

Sound : Steel when struck gives a treble musical or sharp metallic ring like that of bell metal, wrought iron a note of a low pitch, while cast iron when struck gives a hollow sound.

Fracture : A fracture in steel is grannular, in wrought iron fibrous and in cast iron crystalline in appearnnce.

Acid test : Apply a drop of nitric acid on the iron to be tested and let it remain for a moment, and then rinse with water. There will be no perceptible stain on wrought iron ; a dark grey stain on steel ; and a black or brownish stain on cast iron, which is due to the presence of carbon.

3. PROPERTIES AND USES OF CAST IRON

Cast iron is obtained by remelting pig iron with certain refining processes which are carried out in a special furnace called "cupola furnace" which is more or less like a small blast furnace. Old castings or scrap iron are sometimes added with the pig iron before melting, these improve the quality of the cast iron produced.

Cast iron is an alloy of carbon and iron with or without other elements. Carbon in cast irons is usually between 1.7 to 4.5 per cent. A

line of demarcation is generally drawn between cast iron and steel at 2.0 per cent of carbon.

The heavy percentage of carbon in cast iron makes it brittle, non-malleable and non-ductile metal. It cannot be forged at any temperature or tempered, rolled, drawn under a hammer or easily welded, but it can be hardened by heating and dipping in cold water. Cast iron cannot be punched or riveted like steel but it can be easily melted and cast into various shapes and machined. It melts at a temperature of about 1100 deg. C. Normally offers excellent resistance to corrosion compared with any other ferrous metal and rusts slowly, but corrodes in sea water.

Cast irons are generally classified into three main varieties : Grey cast irons, White cast irons and Malleable cast irons.

Grey cast iron has a dark appearance and contains a large proportion of free carbon and is much better than white cast iron in which carbon is in chemical combination. It is softer and tougher than white cast iron and runs freely into moulds and can be easily machined. Is used for pipes, cylinders, machine beds, frames, etc.

Grey cast iron is the strongest variety of cast iron and is made from best varieties of pig iron. It is best suited for foundry work and is generally used for ordinary casting and for engineering works. Grey cast iron can be distinguished from white cast iron by dropping nitric acid on a clean fractured surface : on grey iron black stain is formed and on a white cast iron a brown stain.

White cast iron has a light appearance. It is very hard and brittle and is unsuitable for general castings. Forms a pasty mass when heated and does not freely fill a mould and is therefore used for inferior castings. Is mostly used for conversion to wrought iron.

Malleable cast iron is annealed white cast iron from which some of its carbon has been extracted by heating to red heat, which makes it stronger, less brittle, more malleable and ductile than ordinary cast iron (but is inferior to cast steel). It is somewhat of the composition of wrought iron. It is softer and tougher than grey cast iron and can be bent cold, forged and welded and compares favourably with cast steel. It is generally freer from blow-holes than cast steel, and is also more resistant to corrosion, and withstands shocks and blows. Malleable cast iron has an ultimate compressive strength twice its tensile strength. It is used for small castings such as levers, door fastenings, hinges, pipe-fittings, hardware and agricultural implements.

Small articles of cast iron are sometimes only partially treated so as to make the outer crust malleable. Castings which have to withstand some blows are also treated in this manner.

Chilled iron is obtained by melting grey iron and pouring it into cold metallic moulds which cause sudden cooling of the fluid metal making castings with hard outer surface.

Alloy Cast Irons : Cast iron can be made hard and malleable by alloying with nickel and chromium and non-corrosive with brass or such other metals. Nickel is very often used and its effect is to harden an strengthen the cast iron. Alloyed cast iron can be heat treated like steel to further modify it and to obtain increased strength. Very intricate castings can be produced from alloyed cast irons. Cast iron alloys are not very commonly used.

Foundry Work. Patterns of the articles to be moulded are made either of well seasoned wood, metal or plaster of Paris. If a large number of castings are to be made of the same shape, moulds are made of cast metal so that they can be used repeatedly. Sand moulds can be used only once. Patterns are made about one to three per cent larger in size than the size of the cast required to allow for the contraction of the molten metal on cooling and solidifying. The mould is usually made in two halves to facilitate its removal. Sharp corners and angles in the castings are avoided as far as possible. Where moulds are made of sand, special foundry sand is used possessing the necessary properties of adhesion. Pure silica sands with small quantities of alumina and lime give best results. Damp sand or dry sand mouldings are used according to the castings to be made.

The pattern is placed in the moulding box and filled around with the foundry sand ; the pattern is then taken out leaving behind a hollow core in which the molten metal is poured. Holes are left for the iron to be poured in and the sand round the mould is also pierced with holes to allow escape of the air. Holes made for pouring in the molten metal should be so arranged that all the metal flows at the same time from different parts and all portions of the casting cool together.

After the casting has solidified and cooled it is taken out from the mould, cleaned with a wire brush and all irregularities on the surface are removed by either filing or chipping. Cast iron should be painted soon after it leaves the mould to preserve in tact the hard skin before it has time to rust ; oxide of iron paints should be preferred to lead paints.

Drop Forgings : This process is adopted where small parts have to be manufactured in large numbers. Dies are attached to the anvil and also to the head of the hammer (usually steam hammers are used), the required article is forged to shape when the hammer falls on the anvil, the metal being fed at regular intervals. Drop forgings are commonly used for manufacturing levers, wrenches, small connecting rods and other intricate works.

Average Shrinkage Allowance in Patterns per metre in mm

Aluminium	13	Cast iron	10	Steel	15
Brass	15	Copper	15	Tin	7
Bronze	13	Lead	10	Zinc	15

Defects in Castings. A cast iron casting should show a close-grained texture with smooth clean surface and all edges true and sharp. A casting when tapped with a small hammer should give a clear ring. A defective casting gives a dull and deadening sound which indicates the presence of sand-holes or blow-holes and air bubbles in the body of the casting, a crack or other imperfections. A good casting should not have any cracks or honeycomb surfaces. Blow-holes are gas cavities and are due to the formation of steam from damp moulds ; sand-holes are due to misplaced sand particles ; rough surface is due to chilling of the iron and failure to fill the parts of the moulds ; shrinkage cracks are formed due to uneven cooling of the castings in parts of different thicknesses ; a coarse-grain is caused by too slow cooling. On a good iron casting a blow from a light hammer on the edges makes a slight impression ; if fragments fly off without making visible indentation, the iron is hard and brittle.

Uses of Cast Irons. Cast iron is not recommended to be used horizontally for either heavy or variable loads, nor where the slightest liability for shock exists. These irons are very strong in compression but weak in tension and crack and snap suddenly when subjected to shocks, overloading or fire, without giving any warning of approaching failure under such stresses. It is five to six times as strong in compression as in tension. Ultimate tensile strength of cast irons vary from 14 to 29 kg./sq. mm and crushing (or compression) strength 47 to 110 kg./sq. mm and which appears to vary inversely as the tensile strength. Cast irons can take transverse rupture stress up to 61 kg./sq. mm.

Cast iron is used for building columns, caps and bases of columns, brackets, water and sewage pipes, wheels, spiral staircases, gratings, agricultural implements, etc.

All cast irons used on works should be tough, close-grained grey metal, free from air holes, sand holes, flaws, and with an even surface. It should be sufficiently soft to admit of being easily cut by either a chised or a drill.

Brazing or riveting is not done in cast iron ; holes for bolts etc. are either drilled out or cast in the casting.

Field Tests for Castings. A bar 2.5 cm square in section laid upon supports 30 cm apart should resist a load of one tonne placed in the centre of its length.

4. PROPERTIES AND USES OF WROUGHT IRON

Wrought iron is the purest form of iron with low carbon content, less than 0.15 per cent. It is made from white pig iron by remelting and purifying it in the puddling furnace. It possesses the important qualities of toughness, ductility and malleability. It can be easily welded at a temperature of 900 deg. C. by a hammer but not by fusion as its melting point is at about 1500 deg. C. Wrought iron becomes soft and plastic at red heat when it can be easily forged to take any shape under the hammer. It can be bent or twisted when either hot or cold, but it cannot be cast into moulds, and can neither be hardened nor tempered like steel but can be case-hardened. Is not appreciably hardened by quenching (suddenly cooling) or tempering. When a bar of wrought iron is heated to redness and quenched in water it becomes permanently shorter than before. Bars and plates of wrought iron rolled cold under pressure acquire a polished surface, and have their tensile strength increased and ductility reduced. It can be worked more easily than steel in threading machines. Mild steel which is stronger than wrought iron is now largely replacing it.

Some grades of wrought iron are more rust resisting than steel and are at times used on hydraulic and marine works. Wrought iron rusts more quickly than cast iron but stands salt water better.

Uses of Wrought Iron. Wrought iron is used for making spikes, nails, bolts and nuts, wires, chains, horseshoe bars, sheets and plates, stay bolts, fish plates, ties, handrails, ornamental gates, straps for timber roof trusses, pipes and tubing, armatures, electromagnets, etc. Steel is now superseding wrought iron for rivets. Wrought iron is sold as "merchant bar" for subsequent working into various wrought shapes. It is very suitable to resist tensile stresses.

Field Tests for Strength of Wrought Iron. Strength varies according to the grade of the iron. It is usually specified that a wrought iron bar should elongate 20 per cent of its length at the time of rupture under a slowly applied tensile breaking stress and fail under a gradually increasing stress of not less than 3.5 tonnes/sq. cm. Rivet and bolt bars should stand bending double when cold without cracking. Bars over 5 cm in diameter should be capable of bending double when cold without cracking, to a curve of which the inner radius is twice the thickness of the piece tested. The better the iron the more it can be bent.

Manufacture of Corrugated Sheets. Plane iron sheets are passed between grooved rollers which bend them into a series of parallel corrugations. These corrugations increase the stiffness and strength of the sheets considerably.

5. PROPERTIES AND USES OF STEEL

The term "steel" is employed in general sense to those alloys of iron and carbon whose total carbon content does not exceed 2 per cent (as a rule less than 1.5 per cent—difference from cast iron) and steels are graded according to the percentage of carbon present. The smaller the amount of carbon steel contains, the nearer will its properties resemble those of wrought iron, and greater the quantity of carbon it possesses, tends to make its characteristics similar to cast iron. In ordinary steel, the mechanical properties are chiefly affected by modification of the carbon percentage. Steels always contain in minute quantities some or all of the elements (as they usually have beneficial effects), manganese, silicon, sulphur, phosphorus; sulphur and phosphorus are next in importance to carbon.

Steel consisting of iron and carbon only is called "carbon steel", which is ordinary steel. Where small quantities of other elements such as nickel, chromium, vanadium are also present in addition to carbon, it is known as "alloy steel". Carbon steels are roughly divided into two categories, *viz.*, soft steels with less than 0.45 per cent of carbon and hard steels with over 0.5 per cent carbon. The lesser the carbon, the softer the steel. The proportion of carbon is varied in the steels depending on the purpose for which they are required.

Carbon which is the most important constituent of steel, gives it strength and hardness, but increase of carbon percentage correspondingly decreases ductility, malleability and toughness. The tensile strength, hardness, yield point and elastic limit of carbon steels increase with the carbon content up to about 1 per cent of carbon. The higher the percentage of carbon the lower is the melting point.

There are many grades and classes of steels in the market under different trade names: (*i*) Ordinary structural steel has 0.24 per cent carbon and no alloys (except in minute quantities as described earlier). (*ii*) High tensile structural steel has carbon content not exceeding 0.3 per cent (0.25 per cent for rivet bars) with silicon and manganese up to 1.5 per cent. High tensile structural steels with corrosive resisting properties have copper and chromium in small quantities. A general classification of steel according to carbon percentage is as follows:

Carbon up to 0.25%	— Mild or soft steel; low carbon steel.
Carbon 0.25 to 0.70	— Medium carbon steel.
Carbon 0.70 to 1.25%	— High carbon steel; tool steel; hard steel.
Carbon over 1.25%	— Extra-hard steel; very high carbon steel.

A soft and malleable steel is required for rolling into thin sheets; a very soft and ductile steel for drawing into wires; and a very hard and brittle steel is required for making tools.

Most steels become hard and more or less brittle by hardening but very low carbon steels and wrought irons containing less than 0.2 per cent of carbon cannot be hardened or tempered by heating, but they can be "case hardened." Steels respond to heat treatment to an increasing extent as the carbon content is increased, degree of hardness obtained depending upon the carbon content. Steels containing less carbon can be more easily welded. Steels are hardened by heating to redness and quenching in water or oil. They can also be annealed or softened by heating to redness and cooling slowly.

Steels are highly elastic, ductile, malleable, forgeable and weldable. They can be hardened and tempered and are fusible at a lower temperature than wrought iron and retain magnetic properties. Smithing of steel is more difficult than wrought iron and it is more liable to injury from over-heating. Steels have much higher tensile and compressive strength than wrought iron and stand wear and tear much better. Steel plates sustain greater injury when punched than wrought iron, therefore holes should be drilled in steel plates as far as possible.

Mild Steel. Is a soft carbon steel and may contain 0.2 to 0.5 per cent of carbon. Ordinary mild steels possess excellent ductility and they are superior to wrought iron being stronger, more uniform and more ductile. It is an elastic material and can be easily cut, machined, punched or drilled, welded, forged and rolled (some of the qualities are not always superior to wrought iron but they are more workable). This steel cannot be hardened or tempered but can be case-hardened or surface hardened, and cannot be used for making any cutting tools. Its high ductility enables the material to be bent when cold. Mild steel will recover from deflection when relieved of stresses if they have not exceeded the yield point, and where the yield point has been exceeded, it will elongate up to about 25 to 30 per cent of its length before it breaks, which often permits a steel structure to relieve itself of load overstress without damage. Mild steel is used for all kinds of structural steel works such as Joists, Channels, Angles, Bolts, Rivets, Sheets.

Steel is manufactured by several processes the most important being *Bessemer Process* and *Open-hearth Process*. The steel thus made is converted into steel objects and sections by the process of casting, rolling, forging or pressing.

Mechanical Properties of High Tensile Structural Steel. Specifications prescribe ultimate tensile stress of 60 to 70 kg./sq. mm ; yield stress of 30 to 36 kg./sq. mm, according to thickness. For rivet bars ultimate tensile stress prescribed is 48 kg./sq. mm. This steel is rather difficult to weld and fusion welding method has to be adopted.

Cast Steel. Is a high carbon steel ; it is a term to denote any article of steel formed by casting. It is the strongest and most uniform steel that is made. It is unweldable and cannot be forged, and should never be heated beyond red heat. It is much denser and harder than ordinary steel and also heavier than cast iron. These steels are less ductile and less tough than ordinary steel. Cast steels are used for the manufacture of high grade surgical instruments, cutlery and intricate parts. *Hard cast steel* is used for making cutting tools. A smaller allowance for shrinkage is required for steel castings than for cast iron.

Wrought Steel. Any article of steel formed by forging or hot rolling or hot working in any way.

Hard Steel. Is fusible and gives a much higher resistance to compression than cast steel. It cannot be welded or forged easily. Hard steel is used for special purposes such as bullet-proof sheeting.

Spring Steel. May be either medium or high carbon plain steel or alloyed with other elements in small proportions. Suitable for the manufacture of springs. Steel is heated to 760-780 deg. C. quenched in oil, water or brine and tempered to required hardness.

Weight of Steel. Weight of plates may be taken at 78.43 kg./sq. metre per cm of thickness, and the weight of sections and bars at 0.7843 kg./sq. cm of sectional area per metre run.

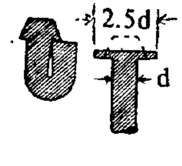
SIMPLE FIELD TESTS FOR STRUCTURAL STEEL

(Based on IS : 226, 277, 432, 1148 and BS : 15, 18, 785.)

Cold Bend Field Tests for Bars

The test pieces should be cut lengthwise and crosswise from plates and lengthwise from sections and bars (including flat bars). For cold bend tests, the test pieces should not be subjected to any heat treatment. The test should be made for each thickness or diameter of bar in a lot. The test piece shall withstand, without fracture, being doubled over either by pressure or by slow and steady blows from a hammer until the two sides of the test piece are parallel and, in the case of bars above 25 mm in diameter or thickness, the internal radius is not greater than 1.5 times the diameter or thickness of the bar, and, in the case of bars of 25 mm and under in diameter or thickness, the internal radius of the bend is not greater than the diameter or thickness of the bar.

The bend test determines whether the metal is adequately ductile.



Tests for Rivet Bars. The rivet shank shall be capable of being bent cold back on itself and hammered until the two parts of the shank touch without fracture on the outside of the bend. The rivet head shall be capable of being flattened at red heat to a uniform thickness and without cracking at the edges until its diameter is 2.5 times the diameter of the shank.

The same tests as given above for ordinary bars have now been prescribed for rivet bars also. In addition, short lengths of rivet bars equal to twice their diameter, when cold, shall withstand, without fracture, being compressed to half their length.

Tensile test is the most important and commonest mechanical test applied to metals. A specially prepared test-piece is stretched by a gradually increasing load applied in opposite directions at its ends until the test-piece breaks. The tougher the steel the more force will be needed to break it. There is an elastic limit and yield point for steel under tension or compression.

Compression test is made in a machine which exerts pressure of a compressive nature until the steel "gives" and will not recover.

Shearing test indicates the ability to resist a shearing action as with rivets.

Creep test indicates how long a metal will withstand a maintained load or stress at a specified temperature without deformation within certain limits.

Fatigue test is repeated reversed stresses test. Pullying and pushing stresses are repeated alternatively in quick successions for a time till the test-piece gives way. (Quickly repeated stresses are more likely to set up fatigue and eventual failure much lower than the yield point than a continuous stress applied in one direction.)

Impact or Shock test indicates the toughness of the material and is a measure of the energy required to break a test-piece, under the shock of the impact, when subjected to a sudden blow.

Torsion test is made to find out the ability of the steel to withstand twisting stresses as found in crankshaft of engines, etc. Torsion test gives more reliable data than direct shear test. Tests made within elastic limit determine the modulus of rigidity and test continued up to destruction determines the maximum shear stress. Modulus of rigidity provides data for arriving at the elastic properties of steel.

Hardness test : A small ball of very hard steel or other material is pressed into the test-piece under dead load and hardness is measured by the indentation made by the ball.

Cold Bend Test for Hard-Drawn Steel Wires :

The test-piece shall withstand, without showing signs of fracture, the following treatment : One end of the test-piece shall be firmly gripped in a vice and the free end shall be bent round a radius equal to the diameter of the wire through an angle of 90 deg., and then bent in the opposite direction round the same radius through an angle of 180 deg., thereafter being bent back again through an angle of 90 deg., to come to the original position.

The ultimate tensile stress of hard drawn steel wires is specified to be 58-66 kg/sq. mm with minimum elongation of 7.5 per cent.

Cold Bend Tests for Mild Steel Sheets :

(*a*) *Black Sheets :* (Gauge Nos. are given at page 4M/14)

A strip from sheet 18 BG and under cut lengthwise or crosswise, shall withstand bending through 180 deg., flat on itself, without fracture ; while a strip of sheet over 16 BG, shall withstand bending through 180 deg. with an included radius equal to 1.5 times the thickness of the sheet without fracture.

(*b*) *Galvanized Sheets :*

Shall withstand the doubling test as given for black sheets above.

Strips 25 mm in width shall withstand bending round a rod of diameter 15 times the thickness of the metal without flaking or peeling the zinc coating.

The following tests are prescribed for galvanized sheets :—

Test pieces, preferably 'L' shaped, 230 mm long and 75 to 100 mm wide, shall be cut both along and across the direction of rolling. Samples of sheets shall withstand bending through 180 deg. around a rod having a diameter equal to the thickness of the number of pieces of the same gauge as specified in the table below, without peeling or flaking of zinc coating.

Number of Pieces of Same Gauge for Inside Spacing of Bend

Zinc Coating in	Birmingham Gauge for Sheets							
grams per sq. m	16	18	20	22	24	26	28	30
750	10	10	11	12	12	14	14	14
600	8	8	9	10	10	11	11	11
450	6	6	7	8	8	8	8	8

Methods for testing weight and uniformity of coating on galvanized steel sheets are given in IS: 429.

Temper Bend Test. The test piece shall be heated to a blood red colour and then quenched in water at a temperature not exceeding 27 deg. C. The colour shall be judged indoors in the shade. The test piece shall then withstand without fracture, being doubled over either by pressure or by blows from a hammer until the internal radius is not greater than 1.5 times the thickness of the test piece, and the sides are parallel.

Tests for Tensile Strength, Yield Point and Elongation, etc., (which are done with machines) are described in IS: 223, 226, 277, 432, and in BS: 15, 18, 785.

Mechanical (Tensile) Properties of Structural Steel : Mild steel shall have an ultimate tensile stress of 42 to 54 kg/sq. mm ; Yield stress 23 to 26 kg/sq. mm (according to thickness) ; elongation 16 to 20 per cent.

For bars from which rivets are cut a more ductile steel is specified having an ultimate tensile stress of 39.0 to 47.0 kg/sq. mm and a minimum elongation of 26 per cent on a gauge length of 8 diameters.

Working Stresses are given in Section 10.

6. HEAT TREATMENT OF STEELS

Heat treatment is another factor, in addition to carbon, which affects the mechanical characteristics of steel considerably. The object of heat treatment is to develop in the steel a particular structure or condition which is required for the metal to be used for any particular purpose. Hardening, tempering, annealing and normalizing are the main heat treatments of steel. All or any of them are used to give the steel the required degree of elasticity, hardness and ductility so that it could develop various properties for different purposes. The properties developed depend upon the percentage of carbon and other elements in the steel and the treatment given. Heat treatment characteristics of steel differing it from cast iron or wrought iron are : The steel can be hardened by heating it to red heat and then suddenly cooling it which process has no effect upon wrought iron. A hardened steel can be softened again, *i.e.*, tempered, while cast iron can be hardened but cannot be tempered or softened again.

Hardening. Hardening means heating the metal to a particular temperature and then cooling it more or less rapidly in a suitable quenching media such as water, brine, oil, special liquids or air. Steels are usually heated to temperature between 950 to 1000 deg. C., according to the thickness of the section, and quenched. The degree of temperature required for heating depends upon the percentage of carbon present in the steel. It is not possible to harden steel with very low carbon percentage. The quenching medium and the rate of cooling is determined by the degree of hardness required and could be anything from ice-cold water to boiling water and oil, the quicker the cooling the harder the steel becomes. Quenching in water or a brine solution gives a much more rapid cooling than oil quenching ; oil takes up the heat slowly. The larger the mass the longer is the time required to make the effect of the quenching reach the core, and the thichness of the material has also a great effect on the uniformity of the hardness developed in it from the surface to the interior. Thin sections such as knife-blades are effectively hardened by cooling in air. On account of the danger from cracking it is usual to air cool smaller sections and oil quench larger sections.

The surface only of steel articles could be hardened by heating with a blow pipe and quenching. The hardness obtained depends upon the temperature to which the steel is heated and then quenched, the higher the temperature of steel before quenching the harder it becomes. But if the temperature exceeds 50 deg. above the upper critical temperature, the steel becomes too brittle to be of any practical use. (The optimum range of temperature for hardening is 1300 deg. to 1350 deg. C.). Therefore steel should not be heated for hardening to a temperature much above its upper critical point. If steel is heated to red heat and then suddenly plunged in cold water it becomes very hard and brittle. Hardening needs an accurate knowledge of the operation of heating and quenching temperature. The steel should be lightly tempered immediately after hardening to make it tough enough for most purposes, otherwise it may crack. Steel can also be hardened by cold mechanical work.

Suitability and purity of the *quenching liquid* is very important. When toughness and elasticity are required rather than an extreme hardness, oil is used instead of water both for hardening and tempering. Grease or acids in water are objectionable as grease is liable to cause uneven hardness and acids produce brittleness. Hard water is very satisfactory.

Carburising is a process of adding extra carbon to the surface of steel by which the outer skin or case is made hard and wear resistant while retaining toughness in the core or inside so that the part may not be readily fractured by shock. Certain parts of machines which are subjected to vibrations, repeated shocks, or wear and tear have to be made of hard and wear resisting surface with tough inside. Such parts are usually made of low carbon steel or wrought iron. The method usually adopted is, the part is packed in a box containing a highly carbonaceous substance such as bone dust, wood charcoal dust, and heated to red heat (about 900 deg. C.) for about 6 to 8 hours when the steel absorbs carbon on the outer skin. The part is then cooled suddenly by immersion in water. Case depths of up to 1.5 mm can be obtained in this manner. The process is also called *Pack Carburising*. After the steel has been cooled it is reheated to a lesser temperature and quenched in air, oil or water, thus hardening all or part of the surface portion of the piece of iron. Wrought iron can have its outer crust partially converted into steel by this method.

The Salt Bath. Small articles can be given a hard case by immersion in a salt bath and afterwards quenched. The salt mixtures are proprietary brands. The use of salt bath for the treatment of metals has increased considerably during recent years and quite a number of salt bath processes are available for case-hardening steel.

Tempering. A "hardened" steel unless tempered (as explained above) is too hard and brittle for most of the practical uses. In the hardening process the contraction and the dimensional change in the article is not also uniform, therefore, another treatment is given with much lower temperature. Tempering or softening is re-heating of hardened (quenched) steel to a temperature below its critical temperature range (transformation of lower change-point, usually 600 deg. to 650 deg. C.) and subsequent cooling in air or quenching in oil or water. Tempering imparts ductility, improves toughness, and reduces brittleness, and hardness can be modified. Slow and uniform heating in tempering is essential. Time and temperature of tempering have to be decided in order to give the steel the desired properties. Usually re-heating is done up to a temperature of 400 deg. C. In the re-heating process, as the heat increases, the hardness diminishes. Steel which has not been thoroughly hardened cannot be tempered.

Annealing is the opposite of hardening, or in other words, is a softening or "hardness removing process so that the article may be machined readily. It is a heat treatment in which the iron is usually reheated to slightly above the critical temperature (light red heat) and then allowed to cool slowly. Annealing is thus different from tempering. The usual

annealing temperatures for the high carbon tool steels are between 760 deg. and 780 deg. C., and largely depend upon the percentage of carbon in the steel. The required annealing temperature is maintained for a specific period. This process rids the material of internal stresses set up by uneven cooling or by rolling, forging or by usage which result in the loss of ductility and make the metal brittle. Annealing reduces tensile strength but increases ductility and brings back the steel to the best physical state to resist fracture under sudden stresses. High carbon and high speed steels must necessarily be annealed before hardening to improve the toughness.

Case Hardening or Surface Hardening also called **Case Carburizing** is designed to give steel parts called upon to withstand wear, shock and fatigue, an exceptionally hard surface or case combined with a tough core or interior. Case hardening is obtained by any of the methods detailed below and usually involves the use of low carbon steels commonly referred to as mild steels with less than 0.25 per cent steel. The steel is heated to a specific temperature in either a liquid (usually a salt bath), or a solid, or a gas (carburizing gas), rich in carbon. The process raises the carbon content of the steel surface. The advantages of liquid case hardening are : uniform depth of case, absence of distortion, rapidity of operation.

Flame Hardening. The surface of the article is heated rapidly with an oxy-acetylene blowpipe followed immediately by a rapid cooling obtained from a jet of water on spraying. This process makes the surface hard and as the hardened surface is purely local no volumetric change occurs. This method is particularly useful for the hardening of gear teeth.

Rolling—hot working operation. Is a method of reducing the cross-section of a piece of steel by passing it through two revolving cylindrical rolls driven in opposite directions, heated to a temperature of between 900 deg. and 1250 deg. C. Its effect on the structure of the steel corresponds largely to those of forging, and the process increases the yield point and ultimate tensile strength of steel.

Chromising. This is a process for converting the surface of articles made from mild steel into stainless steel by impregnating them with chromium. It is not a plating process but the chromium is caused to diffuse into steel altering the composition of the surface metal. Articles which are chromised include tubes, sheets, nuts, bolts, woodscrews, pump components and other similar castings.

7. PROTECTION OF METALS AGAINST CORROSION

Rusting of Iron and Steel. Rust is hydrated oxide of iron and a product of corrosion. Oxygen and moisture are necessary for rusting of iron. Rusting has some direct relation with carbon content on an iron. Pure iron resists corrosion and rusting much better than a medium carbon steel. Usually steel rusts more easily than wrought iron. There appears to be little difference between good wrought iron and steel when exposed to atmosphere, sea water or as pipes in water supply systems. Cast iron offers excellent resistance to corrosion than any other ferrous metal ; cast iron water mains is an example. Pores or cracks in the surface induce corrosion and a material whose structure is uniform resists corrosion better than one having a non-uniform structure. A hardened carbon steel does not rust so readily as when the same steel has been annealed because its structure is more uniform.

In locations where structural steel is exposed to excessive atmospheric corrosion, copper bearing steel may be used, the percentage of copper being 0.20 to 0.35 per cent. Copper bearing steels are as suitable as mild steel and only slightly more expensive, but have resistance to corrosion at least twice that of mild steel. Copper bearing steels are now employed to give long life to steel sheet piling.

Copper and lead placed in contact with iron will set up galvanic action and induce corrosion of iron.

Galvanizing (Zinc Plating). The surface of iron or steel is coated with a thin layer of zinc to protect it against corrosion. There are two processes : Hot Dip galvanizing and Electro-galvanizing (cold process). In the former process the article is cleaned in a bath or acid and then dipped in molten zinc, the surface of which is covered with a layer of ammonium chloride (sal-ammoniac). This produces a skin of zinc alloyed to the steel. A small quantity of aluminium is generally added to the molten zinc. The amount of zinc coating on galvanized sheets varies from 300 to 750 grams/sq. m, both sides inclusive. Electro-galvanizing is a process of zinc plating similar to other forms of electro-plating. The coating produced by the electrolytic process is an improvement upon that produced by the hot process and the film of zinc although ample to protect from oxidation is porous and inferior to that produced by the hot dip process. With the hot galvanizing process, ductility and tensile strength of fine gauge wires or sheets are reduced. Zinc will not adhere on a dirty surface or scaly places and will blister or leave out uncoated patches.

Sheradising (Dry Galvanizing). Is a process for coating articles with zinc by packing them in zinc dust and heating to 300 deg. C. The articles to be coated are first dipped in an acid bath. The zinc combines with the surface of the metal at a temperature below the melting point of zinc which slightly hardens the metal superficially. The coating is very durable and can be polished. This is also known as *cementation (coating) process.*

Metal Spraying. A coating of molten zinc, tin, lead or aluminium, etc. is given by means of a spray-gun under high pressure at ordinary temperature. The surface of the material to be treated must be scrupulously clean. It is a useful method for the protection of works already in service.

Nickel Plating. Iron, steel, copper, brass, bronze, lead, aluminium are often nickel plated by immersing the articles in a solution of nickel cyanide and sulphuric acid. Nickel is readily deposited over the surface by electro-plating. Before nickel-plating the article, if not made of copper or brass, usually require preliminary copper plating before nickel can adhere to them satisfactorily. In copper plating, a copper sulphate solution is generally used.

Chromium Plating. Chromium plating is more difficult than nickel plating as special precautions are necessary in the process. Chromium if plated sufficiently thick gives a very hard surface and is therefore employed for coating wearing parts of soft steel articles, which is done at a lesser

cost, instead of using heat-treated high alloy steels. Due to its low co-efficient of friction chromium is used for coating the wearing parts of pins, bushes, etc. Chromium is superior to nickel and has brighter lustre than nickel and does not tarnish and will stay indefinitely while nickel may turn black in some atmospheres. For chromium plating iron or other ferrous metals the articles are nickel plated first which has been applied over a base coat of cadmium, otherwise they do not give a good finish and there is a tendency for peeling of the chromium. When chromium plate is applied to brass or other non-ferrous metals, it is applied over a base coat of nickel.

Chromium plating can usually be distinguished as it has a bluish cast as compared with nickel which has a white colour and a little more of a yellow tinge. Chromium is also softer and more easily scratched than nickel. Cadmium plating has a dull gray colour. To test to ascertain if plating is chromium or nickel, hang a plated sample in a ten per cent solution of hydrochloric acid and if the plating is chromium it will be dissolved in about five minutes time but if the plating is nickel the acid will have little effect.

Tin Plating is a process similar to hot galvanizing, tin being used for coating instead of zinc. Sheets of black iron or other base metal are first dipped in a four to eight per cent solution of sulphuric acid to remove any rust, then washed and dipped in a flux of zinc chloride or ammonium chloride and then passed through a bath of molten tin. The protection afforded to iron or steel (against corrosion) by tin is less effective than is that of zinc.

Terne Plating is similar to tin plating the differenee being, lead-tin alloy is used instead of pure tin for coating. Pure lead cannot be coated on iron. Gasoline tanks, metal furniture, non-food containers are terne plated.

Electro-Plating. Metallic coatings such as nickel plating, chromium plating, copper or zinc plating are the common methods of electroplating, and the coatings are given on the principles of electrolysis. The negative electrode is known as the cathode and the positive electrode as the anode. The article to be electro-plated is made to form a cathode while the positive electrode cousists of a piece of metal which is to be plated on the article. The process produces very bright surfaces.

8. STEEL ALLOYS

Alloyed steels are called *special steels* and ordinary steels as *carbon steels.* Certain elements, apart from carbon, are added to steel to improve its qualities ; very little of the iron employed in engineering constructions is pure. Alloyed steels give much better mechanical properties than simple carbon steels. The elements principally employed for alloying are manganese, chromium, nickel, silicon, tungsten and vanadium, and to a lesser degree cobalt, molybdenum and copper. Alloys are not now generally added singly.

Effects of Certain Elements in Steel

Nickel in Steel (Nickel Steel). Nickel is a very useful, but expensive, alloying element. Nickel improves tensile strength and elasticity and reduces the brittleness of steel, and also imparts hardness, ductility and ability to resist shocks and fatigue. Improves endurance and wear, and gives corrosion resisting properties to a certain extent. The best properties are developed by a heat treatment of quenching and tempering; it is more readily forged than carbon steel. The percentage of nickel usually added range from about 1.5 to 5 per cent with carbon 0.1 to 0.4 per cent for structural steels, and from 25 to 35 per cent, with less than 0.4 per cent carbon, in high nickel steels. High nickel steel is practically non-expanding as it has only 1/10th the linear expansion of ordinary steel and possesses great resistance to shock. Such steels are not used for engineering purposes. The addition of 1.5 per of chromium further increases the tensile strength, hardness, and ductility of steel. Nickel steel containing 3 to 4.5 per cent nickel is used frequently for long span bridge construction, bearings, castings and shafts, etc.

Chromium in Steel (chrome steel). Chromium is the key element in the wide range of stainless and heat resisting steels. Addition of chromium to steel makes it very hard and tough, increase elastic limit and strength of steel and also the ability to withstand wear and abrasion. Even a small percentage of chromium gives improved resistance to corrosion. Both nickel and chromium, or in conjunction with vanadium, added to steel give superior physical properties than when either element is added alone. Chrome steel is used for making chisels, drills, razors saw, blades, files, dies, ball bearings and rollers, safes, cutlery, etc.

Chrome-Nickel Steel has high strength, elastic limit and hardness combined with good durability, and is very tough. It is used for shafts, gears, axels, armour plating, etc. Chromium imparts greater strength and hardness than nickel. Chromium steels need careful heat treatments.

Stainless Steels. There are many varieties of stainless steels with vide range of mechanical properties. Stainless steels have varying proportions of chromium, nickel, carbon and other elements. Chromium is an essential constituent of all such steels and a stainless steel must have a minimum of 13 to 14 per cent of chromium to give it freedom from corrosion. Stainless steels usually have 18 per cent chromium and 8 per cent of nickel. The addition of nickel makes it more resistant to corrosion and gives it greater ductility and better working qualities. When only chromium is added its percentage is increased to 25 to 30 per cent. A stainless steel is very strong and tough, acid and corrosion resistant and withstands ordinary atmospheric conditions. These steels are of malleable character and can be readily cold rolled into sheets, deeply pressed, machined and drawn into wires and tubes. Most of such steels can be welded by either the electric process or by the oxy-acetylene method but will not weld by the usual method of heating in a smith's fire. These steels are not good in respect of thermal conductivity and are also poor conductors of electricity. The finished surface of stainless steel should be free from all scales, pits and cracks, otherwise it will rust. Stainless steel is of almost silver-white appearance.

Tests for distinguishing stainless steels from ordinary steels: A drop of nitric acid on common steel will dissolve the steel but on a stainless steel

the acid will merely etch the surfaee producing a gray appearaace. Magnets have reduced attraction with stainless steels.

Copper improves the properties of cast and maileable iron and steel, increases the strength and hardness of low and medium carbon steels and decreases the atmospheric corrosion of steel. In locations where structural steel is exposed to excessive atmospheric corrosion, copper bearing steel may be used. Copper present in such steels is 0.20 to 0.35 per cent. For rivet bars copper may be allowed up to 0.60 per cent. For bridge work the copper content of rivets should be in excess of the copper content of the steel members with which they come in contact.

Tool Steels are mainly of two classes; (*i*) Plain carbon steels having small amounts of alloying elements such as chromium and tungsten which are generally used for hard machine tools, and (*ii*) *High Speed Steels* which contain a high percentage of alloying elements, mainly tungsten, and vary widely in composition. Such steels may be run at a red head without losing their hardness and are used for cutting hard materials or for cutting at high speeds, and also used in parts which must withstand high heat and wear such as exhaust valves of gas engines.

9. NON-FERROUS METALS

Aluminium is a bluish silvery white lustrous metal found in abundance all over in the earth's crust as an oxide and is extracted from the ore called Bauxite. Its development is restricted because of some difficulties of its production. Pure aluminium is very soft, highly ductile and malleable and is therefore alloyed with other metals which increase its tensile strength and hardness while retaining its characteristics of lightness and durability. Aluminium has high resistance to corrosion (but is attacked by caustic soda and some alkalies undrr ordinary atmospheric conditions) and is one of the lightest metals of construction but very strong in proportion to its weight. Pure aluminium is highly conductive to heat and electricity, being second to copper (60 per cent of copper). As regards malleability it is second only to gold and can be hammered, rolled and drawn into various structural shapes. Aluminium is largely used in the forms of sheets, plates, bars, wires, structural parts—both cast and forged, utensils, pistons and cylinders of engines, door and window frames and sashes (suitably alloyed with other metals as pure aluminium is too soft for any practical purposes), and as fine powder for pigment in paints.

Copper. Pure copper is of lustrous red colour and is an ideal material for many purposes and next in importance to iron for engineering works. It is light, tough, strong, very malleable and ductile, possesses high thermal and electrical conductivity being second to silver, and is therefore very widely used for electrical wires and cables. This metal has good properties of resistance to corrosion in dry air, is attacked by acids but stands ·ell sea water. Copper can be forged, rolled or otherwise worked hot or cold and drawn into wires, but cannot be welded; joints are made either by soft or hard soldering. In addition to for wires and cables, copper is used for light gauge tubing for hot and cold water-supply, gas and sanitation services, roofing sheets, etc. Alloy of copper and zinc with over 50 per cent of copper is termed *brass*.

Lead is bluish grey in colour and has a silver like lustre when freshly cut. It is a very soft, highly ductile and malleable, plastic heavy metal with low fusion point and very low strength. This metal resists corrosion and acids and is not affected by soil and sewage effluents or industrial wastes. It also resists atmospheric attack but on exposure to moist air it becomes oxidized and losses its lustre, and does not retain a good polish. Lead can be easily worked or beaten into various shapes without annealing, and is used for gutters, flashings, cistern linings, water service pipes, soil and gas pipes, lead wool, damp-proof courses, cast lead fittings, sheets for roofing, cable coverings, type-metal for printing, foil, solders for plumping, etc. It is also widely used for making alloys and for paints in the form of oxides (white lead, red lead, litharge). Since lead resists acid actions, it is used for lining of tanks and pipes in contact with acids.

Lead is likely to corrode when in contact with fresh or damp concrete or mortar (more so with lime mortar), and also when in contact with certain woods (especially when damp). When lead has to be used in contact with mortar or concrete or wood, it should be heavily coated with a bituminous paint. A lead damp proof course laid without protection in the mortar joint of a brick wall may become severely corroded especially where the brick-work is in an exposed position and is excessively damp. Salts from bricks and alkaline reactions from lime and cement may promote corrosion.

Zinc is a bluish-white metal, very soft, light, possessing low tensile strength, good durability and good resistance to atmospheric action and corrosion due to the protective coating of an oxide film for which properties it is widely used in various forms for engineering works such as roof-sheets, weathering, gutters, flashings, kitchen table tops, electric batteries, and for galvanizing iron. Zinc oxide is extensively used in high grade paints, and for alloying with copper to make *brass*. Zinc requires no painting.

Like lead zinc is liable to corrode when in contact with damp cement concrete or lime plaster etc. A voltaic action is set up when zinc is placed in contact with iron, copper or lead, wihch destroys the zinc.

Tin is a silvery-white lustrous metal occurring as an oxide ore. It is extremely malleable with very low tensile strength and ductility; it cracks when bent and is extremely brittle at high temperatures. But tin offers excellent resistance to corrosion and acid action under many conditions and it is for this reason that it is so widely used in containing vessels for food, fruit and milk, and for protecting copper wires and cables, also for the process of tin plating and for making alloys.

Nickel is a brilliant silvery-white metal, in hardness like soft steel, but is more malleable and ductile. It does not tarnish or corrode in dry air and many acids have no action on it, and is capable of taking good polish. It is attracted by magnets. It is an important constituent of many alloy steels, and is also used for alloying with other metals. Nickel is extensively used for electroplating other metals, for corrosion protection, decorative purposes and for making coins.

Chromium is a silver-white metal which is harder than glass and most of the hardened steels. It has the appearance of nickel or silver (nickel

has a little more of a yellow tinge to it than chromium) and does not corrode or discolour even under intense heat. Chromium holds its colour indefinitely even where nickel will turn black. It is mostly used as alloy to produce stainless metals especially stainless steels and superior varieties of tool steels, and for plating (chromium plating).

Brass is an alloy of copper and zinc, often with the addition of small quantities of lead and tin, properties vary considerably according to the varying proportions of the metals. Its colour depends on the percentage of zinc it contains ; with 10 per cent zinc it is a golden colour, but the shade becomes lighter with additional zinc which is sometimes up to 40 per cent. Brass is stronger and harder than copper. Brasses containing more than 63 per cent of copper are ductile and malleable at ordinary temperatures and can be rolled into sheets, turned into tubes, drawn into wires or cast into moulds. Brass resists corrosion well but it need regular cleaning if a bright appearance is to be maintained. Uses of brass are well known such as stop cocks and valves in water works, bearings for machinery, fittings for doors and windows, and household utensils. It is also used in the form of other alloys such as gun metal and bronzes. Brass is available in the form of sheets, stips, wires, rods and castings.

Bronze is brass alloyed with tin to which lead is also sometimes added (or copper-tin alloys containing more than 93 per cent of copper). Bronzes are hardened copper and are superior to brasses for corrosion resisting properties and stronger too but more difficult to work and also more expensive. They are darker in colour than brass and are used both for engineering and decorative purposes. Boiler tubes in ships and bearings are also made where resistance is required against weather action.

Gun-metal is another type of bronze ; an alloy of copper (above 86 per cent), and tin to which lead and zinc are also sometimes added. This metal has high tenacity and elasticity, and is used for bearings and castings which are required to be stronger than iron ; or for parts subjected to corrosive influences. Can be cast into various shapes.

Corrosion of Non-Ferrous Metals is usually caused due to (*i*) contact between unprotected dissimilar metals where moisture is present, (*ii*) contact with masonry, lime, cement and some woods, which may release acids, alkalies or salts when damp. Both lead and zinc are liable to corrode. Copper is unaffected by cement or lime. A protective coating of bitumen should be given ; one coat of hot bitumen or two thick coats of bituminous paints.

10. WELDING

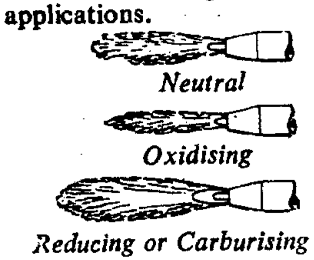
Uses and Advantages of Welding. (*a*) Welding requires much less time (which gives rapid production) than riveting, and is also more economical. (*b*) Entire cross-section of tension members is utilized as there are no rivet holes to be deducted and can thus take more load. (*c*) There is saving of materials for end connections as no gussets, etc., are required. (*d*) Reduction in weight of the structure. (*e*) In certain works welded fabrication is the most practical solution.

Different Processes of Welding. There are two principal forms of welding metals—Electric Arc Welding, and Gas Welding. The electric arc welding process is the most important and is most extensively used for mild steel structures with plate thickness over 1.5 mm. Gas welding is done using oxy-acetylene flame and is not adopted to structural steel work but is used generally for small jobs, and has a very wide application. It requires heating of the members to be welded along with the welding rod and is likely to create temperature stresses in the welded members.

Electric Arc Welding is a process whereby the metal of the two members to be welded is fused together through heat generated by an electric-arc. The electrode used is a specially prepared metal rod or wire which supplies the metal to be deposited in the weld. The heat produced by the electric arc, raises the work-piece at the point of welding and the electrode to a very high temperature, the end of the electrode is melted and fuses with contiguous metal surfaces to be joined. Fusion should be complete over the whole area of the joint surface. This type of welding can be done on most of the mild steels ranging from light articles with a wall or section thickness of 16 gauge to heavy fabrications.

Gas Welding is done with a flame produced by the burning of oxygen acetylene (acetylene gas burning in pure oxygen produces a flame) fed through a blow pipe which is ignited at its tip. The flame is played on the two pieces to be welded until the metal becomes hot enough to fuse together adding additional metal to the joint as necessary by melting into it a suitable electrode.

The proportions of oxygep and acetylene forming the oxy-acetylene frame is an important factor and the gases must be mixed in correct proportions otherwise defective welds will be produced. Three flame conditions arc developed, *viz.*, (*a*) Neutral flame, in which equal quantities of both the gases are supplied and which is required for most of the iron works, steel, stainless steel, cast iron, copper, aluminium, etc. In a neutral flame inner cone is sharply defined with a very slight haze or flicker at its end, disappearance of this flicker indicates excess of oxygen. This condition must be avoided as when excess of oxygen is supplied to the torch it becomes an effective cutting or burning tool—the heated steel burns in the presence of excess of pure oxygen. (*b*) Oxidizing flame, in which excess of oxygen is supplied by the blow-pipe. This flame is used for welding brass and some bronzes. (*c*) A Carburising flame in which an excess of acetylene is delivered through the blow-pipe. This flame is useful for hard-surfacing applications.



FLAME CONDITIONS

Agnition of the acetylene is first achieved at the tip without any flow of oxygen ; when the flow of the acetylene is such that the flame is about to leave the tip, the oxygen is turned on until the correct flame or welding or cutting is obtained.

The oxy-acetylene welding blowpipes are so designed as to bring the two gases together and mix them in correct proportions, and to project them through a special nozzle They are fitted with two small control valves, one for oxygen and the other for acetylene and serve as adjustment for control of the flame condition. The blow-pipe is fitted with a nozzle of the appropriate size which depends upon the type of the work to be done, the metal to be welded and its thickness, and the type of the joint. The size of the flame should be regulated by changing the nozzle and adjusting the gas pressure to obtain the necessary amount of heat. The flame must be carefully regulated and so held that the tip of the white cone is never brought into contact with the molten metal but is kept about 3 mm from its surface. The exact distance of the white cone from the molten metal depends upon the kind of metal being welded and its thickness. The welder should adjust the flame only after wearing the prescribed coloured glass goggles.

High carbon steels are difficult to weld and need pre-heating. Stainless steels can be welded with certain precautions. Wrought iron is well suited for oxy-acetylene welding. This is most suitable process for cast iron welding for which two methods are used : fusion welding and bronze welding. In the fusion welding process, cast iron filler material is used and pre-heating of cast iron is necessary. Bronze welding of cast iron gives a stronger joint and can be carried out more quickly and easily than fusion welding. Malleable iron should always be bronze welded. For all materials other than mild steel it is general to employ a suitable flux.

Oxy-acetylene Welding Equipment normally comprises (*i*) oxygen cylinders ; (*ii*) acetylene cylinders ; (*iii*) regulators and gauges ; (*iv*) welding blowpipes and nozzles ; (*v*) hoses ; (*vi*) welding rods and fluxes ; (*vii*) goggles, wire brushes, etc. Oxygen and acetylene gases are available in cylinders. Hose is of special type ; red colour hose is used for acetylene gas which is combustible and black colour hose for oxygen, so that they are not interchanged. Complete equipment fitted in an iron frame with travelling wheels is available.

Safety Precautions for the Use of Oxy-acetylene Equipment :

Handling and Storage of Cylinders :

(*a*) Do not allow cylinders to drop or strike each other violently. (*b*) Cylinders for oxygen, acetylene and other gases should be painted in distinctive colours and kept far apart, preferably in separate rooms which should be fire-proof, well ventilated and in a dry location. (*c*) Acetylene cylinders should not be stacked but should always be stored up-right. (*d*) Cylinders should be stored and used away from heat, exposed lights or fires and inflammable materials, and precautions must be taken against leakage. Gas cylinders should not come in contact with electric cables or where they might become part of an electric circuit. Dust and moisture should be expelled by opening the cylinder valve before the cylinder is put into use. Oxygen does not burn but supports and accelerates combustion. (*e*) Oxygen cylinders or tanks should be stored with valve end up ; never lay them on the side. (*f*) A blow-pipe should never be put down or hung from equipments or stands unless the gases are turned off.

A metal wire should never be used for cleaning the nozzle where back-firing occurs due to overheating, but the tip should be cooled in water taking care to close the acetylene valve and to leave the oxygen valve slightly open. By "back-fire" or "flash-back" is meant the momentary return of the flame into the blow-pipe tip which may relight immediately upon withdrawing the blowpipe away from the work or necessitate re-ignition.

Precautions Against Fire. Smoking shall be strictly prohibited. Sparks from a welding or cutting operation are often thrown a considerably distance, therefore, precautions should be taken to cover or remove any articles lying nearabout which might catch fire, before starting working. Lightning devices are convenient and safer than matches.

Before welding or cutting any vessels or tanks which may have contained petrol, oil, spirits or any other inflammable or explosive material, it should be made absolutely sure that they have been thoroughly washed and cleaned leaving no trace of the substance or its vapours. The vessels of tanks can be filled with water to within about 3 to 5 cm of the point where the flame is to be employed as a precautionary measure.

Oxy-acetylene is a slower process than electric-arc welding, especially on heavy sections, and is not now used for mild steel structural work if arc welding plant is available. This process is generally used for repair works and for metals less than 10 mm thickness as it has low operating and equipment cost.

Oxygen is supplied in tanks or cylinders. Acetylene gas is produced when calcium carbide, usually termed 'carbide', is dropped into water, which is a cumbustible gas. Carbide is a gray substance obtained by smelting coke and lime in an electric furnace. On small jobs the acetylene may be supplied in cylinders and on large works it may be made at the site.

Oxy-acetylene Cutting : When steel is heated to about 900 deg. C., it will readily burn if fed with oxygen. This principle is involved in the process of oxy-acetylene cutting and the same equipment is used as for welding with a special blowpipe. This oxygen cutting process can also be used effectively for cutting iron and steel under water.

Safety Precautions for Electric-Arc Welding. All welders should wear special goggles fitted with opaque side shields and suitable coloured non-splinterable and non-inflamable lenses and frames for protection from sparks and small flying pieces of slag, and to prevent eye-strain. Glasses of greenish or amber colour are considered to afford maximum protection to the eyes from the injurious rays. Hand-type protective screens are also used. When inspecting welded connections during the process of formation, the inspectors also must provide themselves with suitable goggles to protect their eyes from the direct rays of the arc which should never be looked at with the naked eye from a distance of less than 12 m. Failure to observe this precaution will result in serious straining or burning of the eyes. Welders employed on heavy works should wear fire proof gauntlets, leggings, high boots and apons, and head shields or helmets as a protection against radiated heat and sparks. Where welding is done in the interior of some structure or confined space all the outer clothings should be fire-proof.

Calcium carbide is liable to give off a highly inflammable gas if brought into contact with water. No naked light should be brought near, or any article of combustible or inflammable nature be kept in a carbide store. All unusable calcium carbide dust should be carefully removed and destroyed by immersion in not less than three times its weight of water in the open air. Drums containing carbide should be opened in a dry place with a special cutting tool. It is dangerous to leave carbide drums exposed to rain as they are liable to explode when opened.

Welding Rods or Wires (Electrodes) :

The use of proper welding wire is an important factor in making satisfactory welded joints and choice of the right size of welding rod is based on the thickness of the sheets or plates to be jointed ; the melting of both should take place in the molten pool and not in the flame. Coated electrodes should be preferred to bare wire electrodes.

A simple test indicate the quality of an electrode or welding wire can be made by laying the wire flat on a clean surface and applying the welding flame to it for a distance of about 8 to 10 cm by moving the flame backward and forward until the wire is red and then slowly melting the wire, moving the flame in such a manner so that the wire melts only half-way through its diameter. If the flame is withdrawn as soon as the rod metal begins to melt, the impurities can readily be seen being thrown off in the form of sparks, or a boiling action in the case of inferior metal When cold, an inferior metal will contain numerous spongy, volcano-like irregularities. A good metal welding rod will melt and flow evenly without any disturbing actions. (For *fluxes* see under "Soldering.")

Maximum Diameter of Electrodes for Welding

Average thickness of plate or section	Max. gauge or dia. of electrode to be used
Less than 5 mm	3.2 mm—10 SWG
5 mm to less than 8 mm	4 mm—8 SWG
8 mm to less than 10 mm	5 mm—6 SWG
10 mm to less than 16 mm	6 mm—4 SWG
16 mm to less than 25 mm	9 mm
25 mm and over	9 mm

(See also under Welding of Reinforcement Rods)

Cracks may occur in welding alloy steels owing to the rapidity with which these harden. This may largely be avoided by preheating the parent metal to 300 deg. C. or above in advance of welding to lower the normal cooling rate.

Welding Methods. (*a*) Welds should be made in the flat position as far as practicable. (*b*) Freedom of movement of one member should be allowed as far as possible. (*c*) The work should be securely held in position by means of spot welds, service bolts, clamps or jigs before commencing welding so at to prevent any relative movement due to distortion, wind or other causes. (*d*) The parts to be welded must be thoroughly cleaned and proper flux used. Any paint or rust and loose mill scales, etc. should be removed from the surfaces to be welded and surrounding materials for a distance of at least 12 mm from the weld. A coating of boiled linseed oil may be permitted. Steel to be welded should not be painted or oiled until after erection, unless all ends to be welded are left bare. (*e*) The sequence of welding should be such that when possible the members which offer the greatest resistance to compression are welded first.

Inspection of Welds. The metal in a good weld when cold should show its original colour. If the metal has a rusty or dull red colour or appears crystallized, it is an indication that the heat has been too great and the metal has been burned. A good weld will show an evenness of ripples or waves and well formed beads with good fusion along the edges of the welds. There should be no unfilled cavities, small pockets of slags or burned metal, and small air or gas pockets.

Usual defects in welded joints are : Lack of penetration or fusion of the metal to the bottom of the joint or welded members. Laps in the metal of the weld not properly fused together. Defects are most likely to occur at the root of the weld, and in this position they are liable to have the maximum effects in reducing the strength of the weld.

Flame Cutting : Oxy-acetylene, coal gas, hydrogen or an electric-arc is used. The most widely used is the oxygen cutting process which has already been described under oxy-acetylene welding, which is also called Gas Cutting ; a clean cut is obtained through any thickness of steel. In the electric arc process, the preheat flame is an electric arc. In this process the cutting can be done more quickly and is suitable for a wide variety of applications, including ordinary cutting and burning out rivets etc. It will cut any metal, ferrous or non-ferrous, stainless steel, and also cast iron. The current necessary for the cutting operations varies from 800 to 1200 amperes.

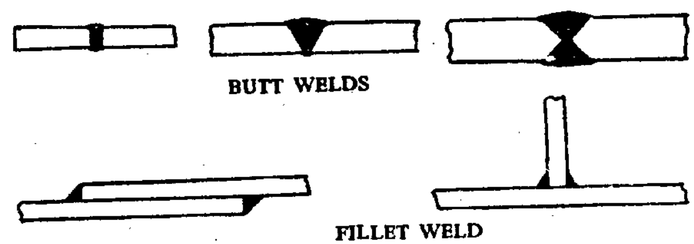
The strength of a welded joint may be taken only about 75 per cent of the stress usually allowed for common works, although tests have shown that if the welding is properly done it is possible to develop the full strength of the members joined.

Forge Welding or plastic welding is an old method in which the edges of the two metals to be jointed are heated to a fusible temperature and then beaten together with a hammer. The heating of the metals may be done through a blacksmith's forge fire. This process is confined to the welding of rods and small pieces. Wrought iron and mild steel can be thus welded without much difficulty, high carbon steel can be welded by a skilled workman, while cast iron cannot be thus welded.

Thermit Welding : This is a fusion welding process in which the faces of the parts to be jointed are pre-heated by the burning of iron oxide and aluminium. Is not a very commonly used method. Thermit process is used for welding heavy sections, such as rail joints, punch frames and large shafts which would be difficult to weld by other methods.

Forging is the process of heating steel to a high temperature and then hammering it to any form required. For some purposes hot forging is necessary before annealing. The directions of fibres of steel have to be considered in some of the forging operations especially where the part has to be subjected to shock or impact.

Types of Welds. There are in general two types of welds : Butt welds and Fillet welds. Butt welds lie within the parts to be joined and fillet welds are exterior to the parts to be joined. Fillet welds are mostly employed for structural works.



In the ordinary butt welding process the usual method is to level the edges of the steel plates so as to form a V-shaped trough. This trough is then filled with the molten metal. Materials of thickness less than 3 mm need not be bevelled. All materials from 3 mm to 10 mm or 12 mm are bevelled from one side only and the bevel should extend to the bottom of the plates. Thicker plates may be bevelled from both sides but care should be taken to see that full penetration and fusion is obtained into the roots of the welds. If it becomes necessary to reweld a joint previously welded, it is essential that the previous weld metal is entirely removed.

For oxy-acetylene welding fillet welds should be avoided as far as possible and butt welds employed wherever practicable. In lap joints the minimum amount of lap should be at least four times the thickness of the thinner member and weld should be provided at the end of each part. In built-up members in which parts are connected by intermittent fillet welds continous side fillet welds should be used at the ends for a length of not less than the width of the plate connected.

Welding of Reinforcement Bars

Oxy-acetylene welding should not be used. Mild steel grade electrodes can be used of size :

Bar size dia. mm	6	10	20	32—40
Electrode size mm	2	2.5	3	5

Electric are butt welding is most suitable for bars of diameter greater than 20 mm, and lap welding for smaller diameters ; and lap welding with longitudinal beads for 6 mm to 40 mm diameters.

11. SOLDERING OF METALS

Soldering is the joining of metals by the addition of molten filler metal of different composition at temperatures well below the melting point of the metals to be joined. The melting point of the solder (filler metal) should be less than that of the metal to be joined and should be able to flow freely ; the more nearly the melting point of the solder approaches that of the metal the better is the union obtained although the more difficult is the operation of soldering. The surfaces to be jointed should be thoroughly cleaned and the metals to be soldered should be heated to a temperature a little above the melting point of the solder to enable it to flow, and a suitable flux must be used to develop adhesion of the solder. Solders are alloys of metals.

Solders generally consist of two parts of lead and one part of tin for plumber's wiped joints and electric cables ; fine solder for tin-smith's work and for general purposes consists of equal parts of lead and tin. Solder consisting of two part of tin and one part of lead requires low heat for melting and is used for electrical, radio, and instrument joints where danger of over-heating and rapid solidifying of solder are important. Solders containing zinc are called hard solders and those containing lead are called soft solders. In ordinary works only soft solders are used. Solder wires are available for different types of works.

Brazing is more or less like soldering, because a harder filler material is used known as 'spelter' which fuses at a high temperature but below the melting temperature of the parts to be jointed. Borax is applied as a flux.

Fluxes for Soldering or Welding. The function of the flux is to dissolve the oxides formed during the operation of welding. No single flux is suitable for all metals and alloys. The following fluxes are commonly used :—

Iron or steel	Borax, Ammonium chloride.
Stainless steel	Zinc chloride and Hydrochloric acid in equal proportions.
Galvanized steel	Hydrochloric acid.
Zinc	Zinc chloride, Hydrochloric acid, Ammonium phosphate.
Lead	Tallow, Resin.
Copper, brass and gun metal	Ammonium chloride, Hydrochloric acid, Chloride of zinc.
Tin	Ammonium phosphate.
Tinned iron	Resin, Hydrochloric acid.
Lead and tin pipes	Resin and sweet oil.
Lead and brass pipes	Tallow, Resin.

Borax is applied as a paste with a brush to the surface of small articles and as a powder to those of large articles. Resin is usually applied by brushing a solution of resin in methylated spirit.

Electric and Thermal Conductivity of Different Metals

Metal	Relative Conductivity Electric	Relative Conductivity Thermal	Metal	Relative Conductivity Electric	Relative Conductivity Thermal
Silver	106	108	Nickel	25	15
Copper	100	100	Iron	17	17
Gold	72	76	Steel	13-17	17
Aluminium	62	56	Platinum	16	18
Zinc	29	29	Tin	15	17
			Lead	8	9

Total expansion of a structure = co-efficient of linear expansion × length of the structure × change of temperature in deg. If expansion is not allowed to occur, stress produced = co-efficient of expansion × change of temperature × modulus of elasticity of the structure metal.

The co-efficient of linear expansion is the change in length, per unit of length, for a change of one degree of temperature. The co-efficient of surface expansion is approximately two times the linear co-efficient, and the co-efficient of volume expansion for solids, is approximately three times the linear co-efficient.

Comparison of Hardness of Metals and Stones

Diamond	10	Glass	4.5—6.5	Marble	3-4	Copper	2-3
Quartz	7	Iron	4-5	Silver	2.5—3	Aluminium	2
Steel	6—8.5	Brass	3-4	Zinc	2.5	Lead : Tin	1.5

Co-efficient of Linear Expansion for 100 deg. F./C.
Per Unit Length

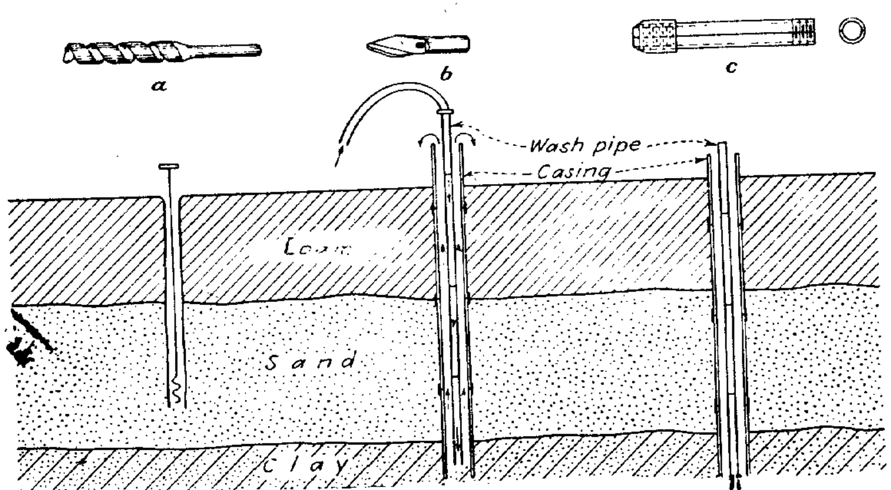
	°F. / °C.		°F. / °C.
Aluminium	.00128/.00230	Lead	.00160/.00290
Ashtar masonry	.00036/.00065		
Brass (cast)	.00104/.00188		.00036/.00065
" **(wire)**	.00107/.00193	Marble	to
Bricks (fire)	.00030/.00055		.00060/.00110
Brickwork	.00030/.00055		
Cement	.00080/.00144		00050/.00090
" Concrete	.00060/.00110	Sand stone	to
Copper	.00095/.00170		.00070/.00125
Glass	.00045/.00081	Steel (structural)	.00067/.00120
Granite (av.)	.00047/.00085	R. C. C.	.00065/.00117
Iron (cast)	.00060/.00110	Tin	.00120/.00125
" (soft forged)	.00070/.00125	Zinc	.00180./00325

Metal	Weight (Av.) Lbs./c. ft.	Gms./c. cm.	Ultimate Tensile Strength Tons/sq. in.	Kgs./sq. mm	Young's Modulus Tons/sq. in.	Melting-point Deg. F.	C.
Aluminium	165	2.64	5-9	7.9-14.2	4500	1220	660
,, (cast)	160	2.56	6	9.5	—	—	—
,, (sheets)	170	2.72	12	18.9	—	—	—
Brass (cast)	510	8.15	8-16	12.6—25.2	—	1650	900
,, (sheets)	510	8.15	13	20.5	—	—	—
,, (wires)	510	8.15	19-22	30.34.6	6700	—	—
Bronze (cast)	550	8.80	16-20	25.2-31.5	6700	1750	950
,, (annealed)	—	—	25-40	11.3-18.1	—	—	—
Cast iron, grey	440	7.00	8-14	12.6-22.0	6500—10200	2200	1200
,, ,, white	475	7.58	12-18	18.9.28.3		—	—
Chromium	428	6.85	—	—	—	3325	1830
Copper (bolts)	558	8.93	16	25.2	6700	1980	1083
,, (cast)	550	8.80	9-11	14.2-17.3	—	1980	1083
,, (sheets)	555	8.88	13-15	20.5-23.6	—	—	—
,, (wire)	555	8.88	20-27	31.5-42.3	8000	—	—
,, (wrought)	558	8.93	15	23.63	—	—	—
Gun metal	540	8.64	13-19	20.5-29.9	5800	1820	995
,, (cast)	540	8.64	18-22	28.3-34.6	—	—	—
Lead (cast)	707	11.30	0.81	1.23	—	620	326
,, (pipes)	707	11.30	1.0	1.57	1000	—	—
,, (sheets)	707	11.30	0.85	1.34	—	—	—
Manganese	500	8.00	—	—	—	2275	1245
Monel metal	554	8.85	31-55	48.8-86.6	11—12000	2475	1360
Nickel	550	8.80	38-45	59.8-70.9	11—13500	2650	1450
,, iron	548	8.78	38-51	59.8-80.3	10250	—	—
,, silver	548	8.78	22-25	34.6-39.4	8000	—	—
Platinum	1342	21.47	118	185.85	—	3250	1785
Silver	655	10.48	18	28.3	5100	1795	980
Steel	490	7.84	28-33	44.1-52.0	13400	2400	1315
Tin	454	7.26	.9-2.7	1.4-4.3	3450	449	232
Tungsten	1180	18.88	—	—	—	6130	3387
Vanadium	345	5.52	—	—	—	3130	1720
Wrought iron	480	7.68	20-25	31.5-39.4	12500	2750	1500
Zinc (cast)	428	6.85	2	3.15	—	786	419
,, (rolled)	416	7.15	8-11	12.6-17.3	5500	—	—
,, (sheets)	448	7.17	8	12.6	—	—	—

These figures are only intended as a guide to the average strengths or melting temperatures of the metals mentioned, as each metal varies to some extent, and the alloys to a greater extent, with varying compositions and other factors.

SECTION 6

FOUNDATIONS DESIGN & CONSTRUCTION



Soil-Boring Methods
(a) Auger borings, (b) Wash borings, (c) Dry-sample borings

1. ELEMENTS OF SOIL MECHANICS

Soil Mechanics is that branch of science which studies the structure, engineering properties and the reaction or behaviour of soils under loading and the changing weather conditions. The chief function of soil studies is to furnish some general principles to supplement and guide the practical experience and free judgement of the engineer, although it cannot give an exact solution to every practical problem. Soils and sites are so variable that it is not practicable to formulate any hard and fast rules.

No standard procedures for soil exploration, selection of foundation type in a given soil and its design have been evolved yet.

Soil is the naturally occurring loose or soft deposit forming part of the earth's crust, produced as a result of weathering or disintegration or decomposition of rock formations, or decay of vegetation, intermingled together. The top layer of the ground that supports vegetation is called 'soil' or 'top soil' and the undisturbed strata lying immediately below the natural top soil is termed *sub-soil.*

Constituents of Soils. Soils contain three components, *viz.*, air, water and solids. The solids are a mixture of mineral matters with particles differing in size, shape and structure and varying in chemical composition.

Types of Soils. There are six main soil types : Gravels, sands, silts, clays, fine-grained organic soils and peat. In view of the wide diversity of soil types they have been classified into groups or classes according to their particle size and cohesive properties. Clays, shales and silts are classified as cohesive soils (true silt has little cohesion). Sands and gravels are classified as non-cohesive or cohesionless soils as they possess no plasticity and tend to lack cohesion especially when in the dry state.

Sand is gritty, silt has a rougher texture than clay, and clay is smooth and greasy to the touch. Clay sticks to the fingers and dries slowly but silt dries fairly quickly and can be dusted off the fingers leaving only a stain. The individual particles of clay and silt are not visible to the naked eye. One of the greatest differences between clays and sands is in their permeability.

Organic matter in soil consists of the more or less decomposed remains of plant and animal organisms. It is of open spongy structure, swells or shrinks with increase or decrease of the moisture content and undergoes considerable volume changes under load. In general, dark colours of grey, brown or black indicate organic soils, whereas brighter colours are usually found with inorganic soils. Organic soils commonly have a distinctive smell and are undesirable constituent of a soil from engineering point of view. Deposits of silts and clays are often accompanied by a considerable amount of organic matter which makes itself evident by its odour when the deposit is disturbed. Due to pressure of organic matter the bearing capacity of the soil is greatly reduced.

Characteristics of Soils. Characteristics of a soil are useful in predicting the performance of the soil under load, which depends upon the grain size, shape, surface texture and chemical composition. The property having most influence on the physical characteristics is that of particle-size distribution, and therefore, it is essential to determine the extent to which each is present.

There is wide variation in the characteristics of different soils and the performance of each individual soil is affected by its moisture content and density. In general, the properties of soils composed largely of coarse materials are primarily controlled by the characteristics of the particles, but for soils composed largely of clays and colloids the properties are primarily controlled by the moisture content. Behaviour of soils containing 30 per cent or more clay depends solely on the characteristics of the clay.

Chemicals in Soils. Some soils and ground waters have a corrosive action on metals, particularly on cast iron, and also have damaging action on cement concrete. These may be due to industrial wastes, sea water and other saline waters, or sulphates which originate in clay soils, and acidic waters which are found in peat soils. A soil having pH value less than 7.0 is an acidic soil and that having pH value more than 7.0 is an alkaline soil. In such soils, the foundations should be built in aluminous cement instead of Portland cement; in less serious cases, a rich dense Portland cement concrete may be used.

Classification and Identification of Soils

Various systems of classifying soils for civil engineering purposes have been devised and the use of different systems has led to much confusion. The coarse-grained soils are usually classified mainly on the basis of their particle-size distribution and the fine-grained soils on the basis of their plasticity characteristics. The two most widely used systems of soil classification are those of U.S. Public Roads Administration and of Casagrande.

Particle-size Analysis or Mechanical Analysis of Soils is the process of separation of a soil into several fractions of different grain size.

Soils are divided into various size groups as, 'gravels', 'sands', 'silts' and 'clays' for the particle-size analysis which is carried out by combining sieving and sedimentation methods. Particle sizes for gravels and sands are determined by "sieve analysis". The gravel fraction is removed by sieving on the 2.36 mm sieve. The samples of soil which pass the 2.36 mm sieve are dried and then shaken through a series of sieves ranging from coarse to fine, and the amount (percentage of sample dry weight) retained on each sieve is weighed and recorded.

For the sedimentation test of the finer fractions than 75 micron sieve, the soil is shaken up in a test tube full of water and allowed to settle. The coarser particles soon settle at the bottom and the proportions of finer materials can be gauged from the thickness of the succeeding layers and the turbidity of the water. Such particle size is found by observing the rate at which the grains will settle through a liquid, as particles of different size have different settling velocities. This is done with a hydrometer or a pipette. It consists of determining the variation in density of the suspension with time.

Field Classification of Soils : Particle sizes :

	With 90% of parts Greater than	But less than or equal to
	mm	mm
Boulders	200	—
Cobbles	80	200
Pebbles	2.36	75
Gravels	2.0	60
Sand, coarse	0.6	2.0
„ , medium	0.2	0.6
„ , fine	0.06	0.2
Silt, coarse	0.02	0.06
„ , medium	0.006	0.02
„ , fine	0.002	0.006
Clay	—	0.002

Boulders, Cobbles, Pebbles and Gravels are subangular to rounded in shape.

Coarse grained soils—In these soils, more than half the total material is larger than 75 microns in diameter (retained on IS sieve 8).

Fine grained inorganic soils—In these soils, more than half of the material is smaller than 75 microns (passing IS sieve 8).

Gravelly soils—In which more than half the coarse grains are larger than 4.75 mm (retained on IS sieve 480).

Sandy soils—In which more than half the coarse material are smaller than 4.75 mm (passing IS sieve 480).

The following terms are used in describing the particle-size distribution of the coarse grained soils :

Well graded—extending evenly over a wide range of particle sizes, without excess or deficiency of any particular sizes.

Poorly graded—containing an excess of some particle sizes and a deficiency of others.

Uniformly graded—extending over a very limited range of particle sizes, *i.e.*, poorly graded but with an excess of only one small range of particle sizes and with a deficiency of all others. (It usually means all particle of about the same size.)

Closely graded—has the same meaning as "uniformly graded".

Textural Classification of Soils are based exclusively on the particle-size distribution and their proportion for a particular soil. The term "texture" indicates the particle size of distribution for a given soil.

The U.S. Bureau of Soils have classified soils into ten types according to their texture as follows :

Class	% Sand	% Silt	% Clay	
Sand	80—100	0—20	0—20	Proportions are determined by sieve analysis. The name denotes the constituent predominating in their composition.
Sandy loam	50—80	0—50	0—20	
Loam	30—50	30—50	0—20	
Silty loam	0—50	50—100	0—20	
Sandy clay loam	50—80	0—30	20—30	
Clay loam	20—50	20—50	20—30	
Silty clay loam	0—30	50—80	20—32	
Sandy clay	50—70	0—20	30—50	
Clay	0—50	0—50	30—100	
Silty clay	0—20	50—70	30—50	

The textural classification are mainly of value for describing coarse-grained soils ; they are not so suitable for classifying clay soils whose properties are less dependent on the particle-size distribution.

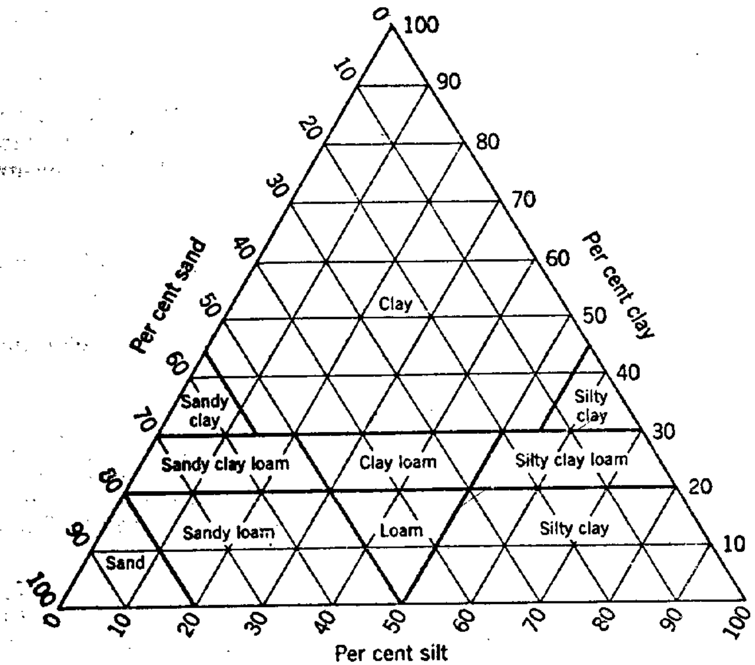
Classification and Characteristics of Soils for Roads and Airfields Based on the Casagrande Classification

Major divisions	Description and identification	Sub-groups	Value as a road foundation when not subject to frost action	Max. dry density at optimum compaction, grams/cu.cm and voids ratio, *e*
Boulders, Cobbles	Soils consisting chiefly of boulders or cobbles.	Boulder gravels	Good to excellent	
Gravel and gravelly soils	Soils with an appreciable fraction between the 75 mm and 2.36 IS sieves. A medium to high dry strength indicates that some clay is present.	Well-graded gravel-sand mixtures, little or no fines.	Excellent	> 2.0 $e < 0.35$
		Well-graded gravel-sands with small clay content.	Excellent	> 2.08 $e < 0.30$
		Uniform gravel with little or no fines.	Good	> 1.76 $e < 0.50$
	A negligible dry strength indicates the absence of clay.	Poorly-graded gravel-sand mixtures, little or no fines.	Good to excellent	> 1.84 $e < 0.45$
		Poorly graded gravel-sand mixtures with excess of fines.	Good to excellent	> 1.92 $e < 0.40$
Sands and sandy soils	Soils with an appreciable fraction between 2.36 IS sieve and 75 micron. Feel gritty when rubbed between the fingers. A medium to high dry strength indicates that some clay is present.	Well-graded sands and gravelly sands, little or no fines.	Excellent to good	> 1.92 $e < 0.40$
		Well-graded sand with small clay content.	Excellent to good	> 2.00 $e < 0.35$
	A negligible dry strength indicates absence of clay.	Uniform sands, with little or no fines.	Fair	> 1.60 $e < 0.70$
		Poorly-graded sands, little or no fines.	Fair to good	> 1.60 $e < 0.70$
		Sands with excess of fines.	Fair to good	> 1.68 $e < 0.60$
Fine grained soils having low plasticity (silts)	Soils with liquid limits less than 35 per cent and generally with less than 20 per cent of clay. Not gritty between the fingers. Cannot be readily rolled into threads when moist. Exhibit dilatancy.	Silts (inorganic), rock flour, silty fine sands with slight plasticity.	Fair to poor	> 1.60 $e < 0.70$
		Clayey silts (inorganic).	Fair to poor	> 1.60 $e < 0.70$
		Organic silts of low plasticity.	Poor	> 1.44 $e < 0.90$
Fine grained soils	Soils with liquid limits between 35 and 50 per	Silty clays (inorganic) and sandy clays.	Fair to poor	> 1.60 $e < 0.70$

having medium plasticity	cent and generally containing between 20 and 40 per cent clay. Can be readily rolled into threads when moist. Do not exhibit dilatancy. Show some shrinkage on drying.	Clay (inorganic) of medium plasticity.	Fair to poor	> 1.52 e < 0.80
		Organic clays of medium plasticity.	Poor	> 1.52 e < 0.80
Fine grained soils having high plasticity	Soils with liquid limits greater than 50 per cent and generally with a clay content greater than 40 per cent. Can be readily rolled into threads when moist. Greasy to the touch. Show considerable shrinkage on drying. All highly compressible soils.	Highly compressible micaceous or diatomaceous soils.	Poor	> 1.60 e > 0.70
		Clays (inorganic) of high plasticity.	Poor to very poor	> 1.44 e < 0.90
		Organic clays of high plasticity.	Very poor	> 1.60 e < 0.70
Fibrous organic soils	Usually brown or black in colour. Very compressible.	Peat and other highly organic swamp soils.	Extremely poor	

Simplified Classification of Soils

Type of soil	Plasticity characteristics		Content of clay particles, weight, %
	plasticity index, % $I_p = W_L - W_p$	diameter of soil thread, mm	
Clay	>17	<1	>30
Loamy soil	17-7	1-3	30-10
Sandy loam	<7	>3	10-3
Sand	Not plastic	Cannot be rolled out	<3



Effects of Seasonal Weather Changes. Movements are caused by the clay shrinking beneath the shallow foundations in dry summer seasons and subsequent swelling during the rains. The ground will dry most where it receives the greatest radiation from the sun. The movements associated with these seasonal changes are greatest at the surface and decrease with the depth, but may go up to even 3 m below the surface. Foundations on such clays should be placed at a depth at least 1 m below ground level as the shrinkage movements beyond that depth are likely to be small. Adjoining deeper excavations may dry out clays and cause settlement. Ground which is shrinkable will exhibit large cracks in the surface in dry weather and become very sticky during the rains.

Consistency of Soils. By consistency is meant the properties of stickiness, friability and plasticity. Soils are classified according to their consistency. Soils may be called plastic or friable depending upon the cohesion between soil particles. Friable soils are cohesionless.

Classification of Soils Based on the U.S. Public Roads Administration Grouping System

Group A-1

Well graded sandy materials from coarse to fine, passing the 2 mm sieve, mixed with clay as binder. (This combination of soil seldom occurs as such in natural deposits but is produced by combination). Highly stable irrespective of moisture conditions, and can be rolled to very high densities giving high bearing power. Makes excellent founations.

Clay %	5—10	Shrinkage Limit	14—20
Silt %	10—20	CME	15 (max.)
Sand % (total)	70—85	Shrinkage Ratio	1.7—1.9
Sand % (coarse)	15—25	Volume Change	0—10
Liquid Limit	14—35	Max. dry Weight	2.08 (min.)
Plasticity Index	4—9	Moisture %	9

Group A-2

Similar to group A-1, containing the same amount or more of binder but of inferior quality. Inferior to group A-1 soils due to poor grading. Rough and dusty in dry weather and fairly stable in wet weather. Can be compacted with either tamping or smooth faced rollers. Makes fair to excellent foundations depending on the mixture.

Clay %	0—45	CME	12—25
Silt %	0—54	Shrinkage Ratio	1.7—1.9
Sand %	55—80	Volume Change	0—16
Liquid Limit	14—35	Max. dry Weight	1.92—2.08
Plasticity Index	3—15	Moisture %	9—12
Shrinkage Limit	15—25		

Group A-3

Composed entirely of adhesiveless coarse materials-gravelly sands or sands with little or no fines. Low stability under loads when dry ; only slightly affected by moisture conditions ; have no volume change ; cannot be compacted by rolling. Make fair to good foundations when adequately confined.

Grading : 75 to 100 per cent sand. The fraction passing the 75 micron sieve is less than 10 per cent. These soils being non-plastic have no liquid limit or plasticity index. Optimum moisture is 9 to 12 per cent and max. dry weight is 1.92 to 2.08 grams/cu cm.

Group A-4

Fine grained soils having low plasticity, consisting predominantly of silt or silt loam soils, containing only moderate to small amount of coarse material with no appreciable amount of sticky colloidal clay. These soils vary widely in texture, composition, and range from the sandy loams to silt and clay loams. Make good foundations when dry but stability is lost when wet.

The soils in this group when wet become elastic and show considerable rebound on removal of load and have no stability. The more plastic

types will expand with increase in moisture in sufficient degree to cause warping at the joints in concrete slabs (of roads) if the soils are placed at moisture contents lower than the optimum.

Clay %, low	20 (max.)	Volume Change	0—16
Silt %	high	Max. dry Weight	1.76—1.92
Sand %	55 (max.)	Moisture %	12—17

Liquid Limit	varies from 20 for sandy loams to 40 for clay loams.
Plasticity Index	varies from 0 for coarse silts with no binder to 15 for clay loams.
Shrinkage Limit	varies from 20 for the better graded sandy clay loams with good binder to 30 for silts.

Group A-5

This group is similar to the A-4 group except that it contains very poorly graded soils and an appreciable percentage of materials such as mica and diatones which form highly elastic sub-grades which appreciably rebound on removal of load even when dry, and are of very low and doubtful stability and difficult to compact.

Grading : With a few exceptions the sand content is less than 55%. Silt percentage is medium and clay percentage is low.

Liquid Limit	more than 35.		
Plasticity Index	usually ranges from 0 to 20 but in some cases may be as high as 60.		
Shrinkage Limit	30—120. Usually exceeds 50 for the undesirable soils of this group.		
Volume Change	0—16	Moisture %	22—30
Max. dry Weight	1.28—1.60		

Group A-6

This group is composed predominantly of highly plastic colloidal clay soils with moderate to negligible amounts of coarse material. In the stiff or soft plastic state they absorb water only when manipulated and become fluid. They can be compacted to relatively high densities by the use of heavy rollers ; have good bearing capacity when compacted to maximum practical density ; are compressible and rebound very little upon removal of load. They are very expansive and if placed sufficiently dry to allow water to be absorbed in large quantities they may cause severe warping of superimposed concrete road slabs.

Clay %	30 (min.)	Shrinkage Limit	6—14
Silt %	medium	Volume Change	17 (min.)
Sand %	55 (max.)	Max. dry Weight	1.28—1.76
Liquid Limit	35 (min.)	Moisture %	17—29
Plasticity Index	18—20		

In the field this soil group is characterized by the presence of shrinkage cracks on all surfaces exposed to drying. When concrete slabs (in road work) are placed over these soils the sub-base should be blanketed with non-expansive materials or compacted to high densities at carefully controlled moisture content.

GROUP A-7

Plastic clay loam soils containing a high percentage of clay part of which is composed of very fine particles combined with organic matter or coarse grained mica. They are similar to those of A-6 group except that at certain moisture contents they are elastic and deform quickly under load and recover appreciably on removal of load, but have good stability and bearing capacity when well compacted to high densities. Alternate wetting and drying of these soils under field conditions leads to rapid and detrimental volume changes. These soils have produced more severe warping of concrete slabs (in road work) than have soils of other groups.

Clay %	30 (min.)	Shrinkage Limit	10—30
Silt %	medium	Volume Change	17 (min.)
Sand %	55 (max)	Max. dry Weight	1.28—1.76
Liquid Limit	35 (min.)	Moisture %	17—28
Plasticity Index	12 (min.)		

GROUP A-8

The soils in this group are composed of very soft peat or muck and dirt and contain excessive quantities of organic matter and moisture ; are usually brown or black in colour and highly compressible. They have very low stability and supporting power and are unsuitable for road sub-grades, fills, and foundations or embankments, as they will settle. Their use in any type of construction should be avoided whenever possible.

Grading : The grading is not significant, but sand content is 55 per cent maximum.

Liquid Limit	35—400	Volume Change	4—200
Plasticity Index	0—60	Max. dry Weight	1.44 (max.)
Shrinkage Limit	30—120		

NOTE—Max. dry weight is in grams/cu. cm. Moisture is optimum moisture. Percentage of dry weight (approx.)

The best foundation soils are those of Group A-1 to Group A-4. The less plastic varieties of Groups A-6 and A-7 are the next best and the more plastic varieties of Groups A-5 and A-6 are the soils that need special treatment. Generally, soils of Group A-1, A-2, A-3 and the better varieties of Group A-4 can be expected to perform satisfactorily to fills without regard to moisture content and special methods of consolidation. Medium plastic varieties of soils of Groups A-4, A-6 and A-7 require the use of the densification method of stabilization to increase satisfactory performance. More plastic varieties of soils and unstable soils, such as those of Group A-5, A-6, and A-8, should not be used in fills. The strength of soils of different type has a general tendency to decrease with decreasing particle size.

Where the dry density of a cohesive soil is less than 1.44 grams/cu. cm it should not be relied upon for the support of footings but should be compacted by pile driving or otherwise.

Effective size is defined as the maximum size of the smallest 10 per cent of the grains by weight.

Uniformity co-efficient is the ratio between the sieve size that will pass 60 per cent to the effective size. It is computed by first determining the size that is coarser than 60 per cent of the soil and dividing that size by the 'effective size.'

NATURE OF SOILS

Cohesion : Cohesion is the internal molecular attraction which resists the rupture or shear of a material. Cohesion is derived in fine grained soils from the water films which bind together the individual particles in the soil mass. Cohesion is the characteristic of the fine materials with particle size below about 0.002 mm (clay). Cohesion of a soil decreases as the moisture content increases. Cohesion is greater in well-compacted clays than in badly-compacted soils and is independent of the external loads applied.

Internal Friction : Internal friction is due to the resistance of grains to sliding over each other and is the characteristic of the coarse materials of particle-size larger than about 0.002 mm. The magnitude of the internal friction of a granular mass depends on the grading, shape, and surface texture of the particles, the degree of compaction and moisture content of the mass, and the load to which it is subjected. Frictional resistance is highest with angular grains having a rough surface and of varied size and shape, and increases with increasing load and is reduced in the presence of a lubricant such as water, present in excessive proportions. For the coarse material it is usually assumed that the particle-size distribution giving the greatest dry density has the greatest internal friction. The strength of a non-cohesive soil depends entirely on internal friction.

Angle of Internal Friction : The resistance in sliding of grain particles of a soil mass depends upon the angle of internal friction. It is usually considered that the value of the angle of internal friction is almost independent of the normal pressure but varies with the degree of packing of the particles, *i.e.* with the density. The soils subjected to the higher normal stresses will have lower moisture contents and higher bulk densities at failure than those subjected to lower normal stresses and the angle of internal friction may thus change.

The true angle of internal friction of clay is seldom zero and may be as much as 26°. The angles of internal friction for granular soils are given under "Shear Tests" in the pages following.

Capillarity is the ability of the soil to transmit moisture in all directions regardless of any gravitational force. Soils possess capillary action similar to a dry cloth with one end immersed in water. Water rises up through soil pores due to capillary attraction. The maximum theoretical height of capillary rise depends upon the pressure which tends to force the water into the soil, and this force increases as the size of the soil particles decreases. The capillary rise in a soil when wet may equal as much as 4 to 5 times the height of capillary rise in the same soil when dry.

Coarse gravel has no capillary rise ; coarse sand has up to 30 cm ; fine sands and silts have capillary rise up to 1.2 m but dry sands have very little capillarity. Clays may have capillary rise up to 0.9 to 1.2 m but pure clays have very low value. In coarse grained soils, the time required to reach the limit of the rise is much less than in fine textured soils.

Permeability of a soil is the rate at which water flows through it under the action of (unit) hydraulic gradient. The passage of moisture through the interspaces or pores of the soil is called "percolation". Soils porous enough for percolation to occur are termed "pervious" or "permeable" while those which do not permit the passage of water are termed "impervious" or "impermeable". In the majority of materials the rate of flow is directly proportional to the head of water, and the permeability is therefore a constant for the particular material. Permeability is a property of the soil mass and not of individual particles, and varies as the square of the diameter of the grains of the soil the ratio of the fine material and with the arrangement of the grain particles of the soil mass. The permeability of cohesive soils is, in general, very small. Sands drain readily whilst silts and clays are difficult or impossible to drain. A knowledge of permeability is required not only for seepage, drainage and ground water problems but also for the rate of settlement of structures on saturated soils. Soils yield under pressure when moisture content is increased. Ground water level depends upon a combination of the permeability of the strata and the causing the water to flow.

Elasticity : A soil is said to be elastic when it suffers a reduction in volume (or is changed in shape and bulk) while the load is applied, but recovers its initial volume immediately the load is removed. The most important characteristic of the elastic behaviour of soil is that no matter how many repetitions of load are applied to it, provided that the stresses set up in the soil do not exceed the "yield stresses" the soil does not become permanently deformed. This elastic behaviour is characteristic of peat.

Resiliency of a body is regarded as the extreme limit to which it can repeatedly be strained without fracture or permanent change of shape.

Compressibility : Gravels, sands and silts are incompressible, *i.e.*, if a moist mass of these materials is subjected to compression, they suffer no significant volume change. Clays are compressible, *i.e.*, if a moist mass of clay is subjected to compression, moisture and or air may be expelled, resulting in a reduction in volume which is not immediately recovered when the compression load is withdrawn. The decrease in volume per unit increase of pressure is defined as the "compressibility" of the soil, and a measure of the rate at which consolidation proceeds is given by the "co-efficient of consolidations" of the soil.

Compressibility of sand and silt varies with density and, compressibility of clay varies directly with water content and inversely with cohesive strength. Clays and other highly compressible soils are known to swell when overburden pressure is removed.

Density : The density or true weight of a soil is equal to the specific gravity of the solid materials × 1000 (weight or density of water per cu. m). A soil consists of solids, pores or voids and the moisture. The overall weight of the mass (including solid particles and the effect of voids whether filled with air or water) per unit volume, *i.e.*, total weight of soil ÷ total

volume of soil, is termed *Bulk Density*. Bulk density varies with the type of the soil, moisture content and its compaction. The weight of the dry solid matters contained in a unit volume of soil, *i.e.*, weight of soil particles ÷ total volume of soil, (determined after the water has been dried without bulk volume change) is termed *Dry Density*.

The usual method of measuring compaction in the field is to determine the dry density of the soil *in situ*. The maximum dry density of a soil is obtained by a specified amount of compaction at the optimum moisture content by the Proctor Compaction Test. For each compaction method, there is an optimum moisture content at which a given soil can be compacted to greatest density, and different soils have different maximum densities and optimum moisture contents. Dry density varies from about 2.00 grams/cu. cm for coarse grained well graded gravels and sands to about 1.45 grams/cu. cm for heavy clays, the corresponding moisture contents being about 4 per cent for the gravel and 26 per cent for the clay. The density of the solids alone is sometimes termed *absolute density*.

Effect of Density on Behaviour of Soils. No soils can be made to pack without voids. Owing to the effects of the films of water around the individual particles clays never pack so tightly as sands. The supporting power of any one soil usually increases with increasing dry density and decreasing moisture content, it is therefore, important to compact sandy soils to the maximum density possible, since the strength of a sandy soil depends upon its density. The effects of dry density and moisture content are, however, to a certain extent inter-related. As a particular soil becomes more dense, it will contain a greater number of particles, and the (pores) volume remaining for air and water will be decreased. When a soil is submerged, the effective density is reduced and with it its bearing capacity. High density assures high shear strength and greater imperviousness.

Voids Ratio is the ratio between the volume of voids or pores in the soil and the volume of the solid particles.

Porosity is the ratio of the volume of voids in a given soil mass to the total volume of the soil mass (solids plus voids).

Porosity varies with the texture of the soil. The ratio of voids is more in finer soils than in coarser soils ; in clayey and silty soils it is about 40 to 50 per cent and in sandy soils about 20 to 30 per cent.

Degree of saturation = volume of water/volume of voids.

Water content = weight of water/weight of soil particles.

The specific gravity of soil particles is defined as the ratio of their density to that of water. The specific gravity of soil particles may vary from 2.0 to 3.3, but usually is between 2.65 and 2.75. The usual soil weight (voidless) is between 2600 and 2700 kg/cu. m.

Effects of Moisture on the Performance of Soils

The properties of a soil mixture are influenced more by variations in moisture content than by any other cause. Saturated soils are improved in strength by drainage and dry soils lose strength by saturation. A water-logged ground is undesirable because of its low bearing capacity.

Fine grained (clayey) soils are most likely to suffer by water absorption. It is therefore important to ascertain the wettest condition in a given case and the basis of design should be the strength of the wet soil. Clayey soils are subject to a large amount of shrinkage but the loss of water that causes this shrinkage is slow, and the shrinkage might amount to as much as 20 per cent in volume.

In the case of sandy soils the detrimental effect of moisture is much less than in clayey soils. Granular soils do not hold water readily and do not shrink much when drying but they shrink more rapidly. When such soils are saturated with water and the water is trapped, the footing may be supported on hydraulic pressure. Under such conditions the soil is without shearing strength. The seepage out of this entrapped water will cause settlement, therefore this water must be drained out.

Ground-water. A rise in the ground-water level may reduce the safe bearing capacity of soils, and lowering of the ground-water over an area may result in differential settlement of structures.

The water-logged strata below the surface of the underground 'reservoir' of water is said to constitute the *zone of saturation*, the surface of the water is known as the *water-table*, and the level at which the water table occurs is known as the *standing water level* or *ground water level*.

In areas where considerable seasonal changes in moisture content occur, the resulting volume changes in clay sub-grades can be minimized to some extent by rolling in granular material.

The Moisture Content of a soil is defined as the ratio of the weight of water present in the soil to the dry weight of the solid soil particles and is expressed as a percentage of the solid particles.

Optimum : A condition which may be roughly defined as one in which the material will just bind together when squeezed hard in the hand.

Optimum Moisture Content : (OMC) That moisture content at which a specified amount of compaction will produce the maximum dry density in a soil ; it is expressed as a percentage by weight of the dry soil. For most soils there is a percentage of moisture at which the soil will compact to its greatest density. For sands and gravels the OMC generally occurs at about 8 to 10 per cent, which may be at 15 per cent for silts and 15 to 20 per cent for clays. It is about 3 to 4 per cent lower than the PL for cohesive soils. Variations of moisture content change the values of the angle of repose, the amount of compaction required and the cohesive strength of a soil.

Hydroscopic Moisture or Hydroscopic Water : Immobile soil mixture that can be driven off only by heat.

Free-water : Water in a soil in excess of hydroscopic and capillary water. Also termed "gravity water".

Fully saturated : All voids filled completely with water.

Partially saturated : Filled partly with water and partly with air.

Determination of Soil Plasticity

Plasticity is the property of a soil to undergo large deformations when stressed, without cracking or crumbling. Plasticity is a major characteristics of all cohesive soils and is due to the lubricating effect of the water films between adjacent particles. Information regarding the plasticity of a soil is very important in soil engineering and certain standard *limits tests* have been prescribed for determination of the same. Cohesionless soils are non-plastic.

Liquid Limit : (LL) It is the minimum amount of water required to be added to a soil, expressed as a percentage of the dry weight of the soil, that will just make it to flow like a liquid when jarred slightly. At the LL, soils have very small shear strength which may be overcome by the application of a little force, and cohesion is practically zero. The LL serves mainly to distinguish soils with respect to the amount of moisture necessary to make them to slide.

Most clay soils have LL of the order of 50 to 90 per cent. 'Fat' clays, which are highly plastic having a high content of colloidal particles, have high LL, which means they 'flow" only on the addition of large amounts of water. 'Lean' clays, which are moderately plastic, having a low content of colloidal particles, have correspondingly low LL. Presence of organic matter in clay increases the LL and comparatively lowers the PL. If sand or silt is added to clay, its LL is lowered. The commonest inorganic silts have LL less than 30 per cent and sands have about 20 per cent. A LL between 20 and 40 indicates a mixture with sand or silt predominating. Peats have a very high LL of several hundred per cent but a small PL.

Plastic Limit : (PL) The plastic limit signifies the percentage of moisture at which the soil changes, with decreasing wetness, from a plastic to a semi-solid state, or with increasing wetness, from the semi-solid to the plastic state. It is the lower limit of the plastic state. It is the moisture content at which a thread of soil can be rolled without breaking until it is only 3 mm in diameter, when it just begins to crumble under pressure exerted by the hand. A small increase in moisture above the PL will destroy cohesion and shear strength of the soil. (See figure at page 6/25).

Sands, gravel and peat do not possess plasticity and have no plastic limit and cannot be rolled into threads at any moisture content. Clays and colloids possess a high degree of plasticity and silts have only occasionally a PL. Clays have an average PL of 45, colloids 46, and silts 20. Both liquid and plastic limits are dependent upon the amount and type of clay present in a soil. A soil with high clay content usually has high liquid and plastic limits and a less cohesive soil gives low figures.

The liquid and plastic limit tests attempt to fix the moisture contents at which a clay soil passes from the solid to the plastic state and from the plastic to the liquid state.

Plasticity Index : (PI) Is the numerical difference between the liquid and the plastic limit of a soil and indicates the magnitude of the range of the moisture contents over which the soil is in a plastic condition as defined by the tests. The liquid and plastic limits are both dependent on the amount and type of clay in a soil, but the PI is generally only dependent on the amount of clay present. It indicates the fineness of the soil and its characteristics as regards plasticity and cohesiveness, *i.e.*, its power to change shape without altering its volume. The information regarding the type of clay in the soil may be obtained by considering the PI in relation to the LL. A high value of PI indicates excess of clay or colloids. When the PL is equal to or greater than the LL, the PI is zero.

Clay soils based on degree of plasticity :—

(Burminters' method)

Degree of plasticity	PI %	Descriptive name	Qualities
Non-plastic	0-1	silt	Friable
Slight plasticity	1-5	trace clay	Desirable
Low plasticity	5-10	little clay	Cohesiveness
Medium plasticity	10-20	clay and silt	Increasingly objectionable plastic displacement and compressibility
High plasticity	20-35	silty clay	
Very high plasticity	>735	clay	

Plastic limit and plastic index for representative soil constituents as determined by laboratory tests :

	LL	PL	PI
Sand	20	0	0
Silt	27	20	7
Clay	100	46	45
Colloids	399	45	0

Shrinkage Limit : (SL). Is the limiting moisture content, expressed as a percentage of the dry weight of the soil, at which a further reduction in the moisture (by evaporation) will not cause any further decrease in the volume of the soil mass but at which an increase in the moisture content will cause an increase in the volume of the soil mass. Evaporation of water causes shrinkage in a soil up to a certain degree beyond which decrease in volume does not occur ; at this stage the soil has reached its shrinkage limit. The SL represents the moisture content at the point at which the soil passes from the semi-solid to the solid state and is a means of describing the pore space present in a soil after it has been allowed to compact itself to the maximum density obtainable by shrinkage.

The SL considered in relation to the natural moisture content of soil in the field indicates whether or not further shrinkage will take place if the soil is allowed to dry out. The lower the SL of the soil, the greater is the possible volume change corresponding to a given variation in the moisture content of the soil. For friable soils, the SL may be anywhere between the LL and 50 per cent of the LL, and for feebly plastic, 25 to 30 per cent ; for medium plastic, 20 to 25 per cent ; for highly plastic, 15 to 20 per cent. There is no definite relation between the PL and the SL.

Consistency or Liquidity Index : (LI) of soils is defined as the natural moisture content of the soil in excess of the PL expressed as a percentage of the PL, *i.e.*,

$$\frac{\text{Natural moisture content—PL}}{\text{Liquid limit}-\text{Plastic limit}} \times 100 \text{ per cent}$$

and is a measure of the consistency of the soil. It merely describes the moisture condition of a soil with respect to its index limits. It shows in what part of its plastic range a given sample of soil lies. A cohesive soil with a natural water content of the same order as its LL will, in general, be a very soft material, while with a natural water content of the same order as its plastic limit will, in general, be a stiff material. Soft clays have a LI approximating to 100 per cent, while stiff clays have a LI which approximates to zero and may be even negative.

Field Moisture Equivalent : (FME) The FME of a soil is the percentage moisture content at which the demands for absorbed water are fully satisfied and at which a drop of water placed on the smooth surface of the soil will not be absorbed immediately but will spread out over the surface.

The above test for and FME is not now in much use.

General Properties of Soil Materials : The main properties affecting the mechanical stability of granular materials are internal friction and cohesion both of which depend on the moisure content of the soil as well as its grading. Factors such as swelling and shrinkage are only likely to be of importance in materials containing an appreciable clay fraction. Soil mixtures of clay and sand partake of both cohesion and internal friction resistance. Most of the sandy soils have some cohesion and most of the clayey soils have internal friction.

Non cohesive soils are described as loose or compact, uniform or well graded. Cohesive soils are described as soft, firm or stiff.

Clay : Is tenacious and plastic when wet, possessing considerable cohesive strength, very pronounced capillarity and practically no internal friction. Clay is usually recognized by its sticky and plastic properties and special odour. Clay particles in pure state are soluble in water, or remain as mechanical mixture in suspension in a colloidal state and gradually settle down at bottom in the form of stiff paste. When dry, clay forms hard lumps which cannot be broken down and powdered between the fingers, while lumps of silt can readily be broken down and powdered. Difference between clay and silt is not only based on the grain particle size but more on the plasticity and shear strength. 'Fat' clays are highly plastic and 'lean' clays are moderately plastic. Colour of clay may be black, white, red or yellow. For many engineering purposes a perfectly pure clay would be useless as it might be of so fine a texture and so highly plastic that it would shrink excessively on drying.

Clay is very absorbent and can swell to double its volume. Clays tend to hold free water in addition to their adhered water and do not drain nor do they dry out rapidly. Clays are subject to a large amount of shrinkage from loss of moisture which may be due to evaporation or transpiration from vegetation. Shrinkage in clays also results from external loadings (or consolidation) as under load the moisture in clay is forced out, and which can be forced out more rapidly if it is mixed with sand. Some clays lose their shear strength considerably when their structure is disturbed, such clays have a low liquid limit.

Natural clay deposits may contain up to 70 per cent or even more of material belonging to the sand and silt grades. A soil behaving essentially as a clay, but having an appreciable proportion of sand or silt is referred to as a *sandy clay* or *silty clay*.

Boulder clay. A deposit of unstratified clay or sandy clay containing stones of various sizes scattered irregularly throughout its mass. The stones are not necessarily all of 'boulder size'. Boulder clay may contain variable proportions of coarse material and stones, but there is usually sufficient clay present to impart cohesion.

Hard clay. A clay which at its natural moisture content requires a pick or pneumatic spade for it excavation or removal and cannot be remoulded with the fingers. Which can be indented with difficulty by the thumb nail.

Stiff clay. A clay which, at its natural moisture content, can be readily indented by thumb but penetrated only with great effort.

Firm clay or Medium clay. A clay which at its natural moisture content can be excavated with a spade and can be remoulded with substantial pressure with the fingers. Which can easily be penetrated several centimetres by thumb with moderate effort.

Soft clay. A clay which at its natural moisture content can be readily excavated with a spade or shovel and can be remoulded easily in the fingers.

Sand : Small mineral particles from natural sources, largely the result of breaking down of sandstones. Coarse sand frequently is rounded like the gravel with which it is found while fine sand particles commonly are more angular than coarse sand particles. Sand is gritty to the touch, possesses no plasticity and dry sands no cohesion and very little capillarity but high degree of internal friction, the sharper the sand grains the greater is this internal friction. A sandy soil if squeezed in the hand when dry, will fall apart when pressure is released ; squeezed when moist, it will form a cast but will crumble when touched.

The particle shape of sand grains or pebbles may be described as angular, subangular or irregular or as rounded.

Silt : A natural sediment of materials, usually deposited in water, consisting of an intimate mixture of fine particles of sand, clay and peat, etc. A soil which can be considered as intermediate between clay and sand. Silt has a gritty touch (but not very gritty) when bitten between the teeth. Wet silt if squeezed between the fingers moisture comes out. Silt possesses high capillarity, varying degree of internal friction and some cohesion depending on the moisture content, with very little plasticity. It can be rolled into threads between the fingers but crumbles readily when it dries. When dry, a silt may posses appreciable cohesion, but a lump is easily broken and powdered between the fingers. It dries moderately quickly and can be dusted off the fingers, leaving only a stain. Inorganic silt may be distinguished from clay by squeezing in hand, the surface appears to dry up and the specimen lack plasticity. Silt is darker in colour than clay.

Earth : This term is used synonymously with 'soil' in an engineering sense and in particular it refers to excavated material. Sometimes the term is used for clays of low plasticity ; but both the terms, earth, and clay, are often used very loosely by many engineers.

Peat and Muc : An accumulation of fibrous or spongy textured vegetable matter formed by the decay of plants more or less *in situ*. They are usually black or dark brown in colour, very compressible, and of open texture. Inorganic materials (sand, silt) may be sometimes present in varying amounts. Such soils are entirely unsuitable for load bearing ; have very high liquid limits but a small plasticity index. *Muc* is soft mud containing much vegetable matter. In muc the decomposition of organic material is more advanced than in peat.

Colloids : Gelatinous or gluey matter found in clays (of a sticky nature) consisting of ultra fine clay prticles of size below 0.002 mm, in the form of uncrystalline semisolid substance. The colloids absorb moisture, and soils containing large proportions of colloids have greater soil moisture capacity and slower soil moisture movement than an average soil. Colloidal clays are finer clay particles that remain suspended in water and do not settle under the force of gravity.

Black Cotton Soils : Are heavy clay soils, varying from clay to loam, with clay contents of 40 to 50 per cent, formed by the decomposition of rocks by long continued weathering. These soils mostly occur in the central and southern parts of India particularly in the Deccan Plateau where they are known as *regur*. The soils vary greatly in colour (light to dark grey, black or blue black), consistency and fertility. They are very unreliable for any structures as they become highly adhesive (sticky), very soft, and swell when wet losing bearing power considerably. When dry they have a high bearing capacity but contract greatly (to the extent of 20 to 30 per cent of original volume when wet) while drying ; the whole area shrinks and splits up and large cracks are formed even up to 15 cm wide at the surface and extending to 3 to 3.5 m deep where the soil is thick.

The thickness of these soils generally varies from 1 to 3.5 m or even more.

The black cotton soils have an affinity for water far beyond their ability to receive and contain it. This affinity belongs to the clay portion of the soil, so that the soil's tendency to swell and shrink with wetting and drying depends upon the proportion of the clay particles present. The capacity to swell thus also increases with the fineness of the particles. In soils with very high clay contents, the resulting imperviousness, howerver, reduces the capacity to swell. During summer months the soil loses water by desiccation, and shrinkage is caused. Deep and wide shrinkage cracks are formed.

Shale : A compressed and laminated clay (is more or less of the same composition as clay) with or without associated organic matter. Disintegrates on exposure to the air, is plastic when wet, but away from atmosphere it maintains a soft rock-like compactness. Consistency of shales usually ranges from soft plastic clay to very stiff tough clay.

Hard pan : A very dense accumulated mass of soil (clay, sand and gravel) that has been thoroughly cemented together to form a rock-like layer that will not soften when wet and must be excavated with a pick. Any material that cannot be classed either as 'rock' or as 'soil'. Soil that offers high resistance to penetration of drilling tools. Also called Gypsum

Hoggin : A natural deposit of a mixture of small stones, grit and sand, containing a small admixture of clay which acts as a binder and is sufficient to hold the mass togther without affecting the interlocking properties of the coarser particles. Clayey gravels.

Humus: A dark-brown earthy material usually formed in the soil due to partial or complete decomposition of vegetable matter. This is the organic component of the soil and forms an important constituent of agricultural top-soil.

Loam : A general term largely used to refer to soft deposits consisting of a mixture of different grades of sand, silt and clay in relatively equal proportions. A soil between sand and clay. It is mellow with a somewhat gritty feel yet fairly smooth, exhibiting slightly sticky and plastic characteristics. Squeezed when dry, it will form a cast which will bear careful handling, while the cast formed by squeezing the moist soil can be handled freely without breaking. Some loamy soils contain a considerable proportion of organic matter which are top-soils suitable for cultivation and plant growth.

It is 'Silty Loam', 'Sandy Loam' or 'Clay Loam', depending upon the properties proportionate to the contents of the main constituent.

Silty Loam is soil having a moderate amount of fine grades of soil and only a small amount of clay, over half of the particles being of 'silt size'. When dry, it may appear quite cloddy but the lumps can be readily broken, and when pulverised it feels soft and floury. When wet the soil readily runs together and puddles. Either dry or moist, it will form casts which can be freely handled without breaking. The silts and silt loams are relatively unstable at all moisture contents but especially so at high moisture contents, when they have very low bearing capacity.

Sandy Loam is a soil containing much sand but having enough silt and clay to make it somewhat coherent. The individual sand grains can readily be seen and felt. Squeezed in the hand when dry, it will form a cast which will readily fall apart, but if squeezed when moist, a cast can be formed which will bear careful handling without breaking.

Clay Loam is a fine textured soil which breaks into clods and lumps which are hard when dry. When the moist soil is pinched between the thumb and the finger it will form a thin ribbon which will break readily, barely sustaining its own weight. The moist soil is plastic and will form a cast which will bear much handling when kneaded in the hand. It does not crumble readily but tends to work into a heavy compact mass. The clay loams are quite stable at the lower moisture contents and higher densities, but under these conditions are likely to show detrimental volume change if the moisture content is increased. On heavy clay loams tamping rollers have proved more effective than rollers of the smooth faced type.

Marl : Indefinite term for any earthy crumbling deposit consisting of a natural mixture of sticky calcareous, silt and fine sands with a considerable proportion of organic matter. Quite often found in swamps and lakes.

Conglomerate : A rock consisting of rounded pebbles of other rocks in a cemented matrix, forming a consolidated gravel.

Bog : Soft water-logged ground composed largely of peat or mud.

Grit : A coarse-grained sands or sharp fine gravel, the grains of which are more or less angular.

Gravel : Subangular to rounded, water-worn stones of irregular shape and size occuring in natural deposits with or without sandy material. A 'well-graded' gravel contains both sand and stones, with a predominance of the latter. A 'uniform' gravel is one with a predominance of a single size.

Bed rock : Any hard rock bed underlying soft deposits classed as soil in the engineering sense.

SOIL TESTS

A number of methods are employed for testing soils which require special laboratory equipments. Certain of these tests are arbitrary and have been standardized. Brief descriptions and outlines of some of the more important tests are given below.

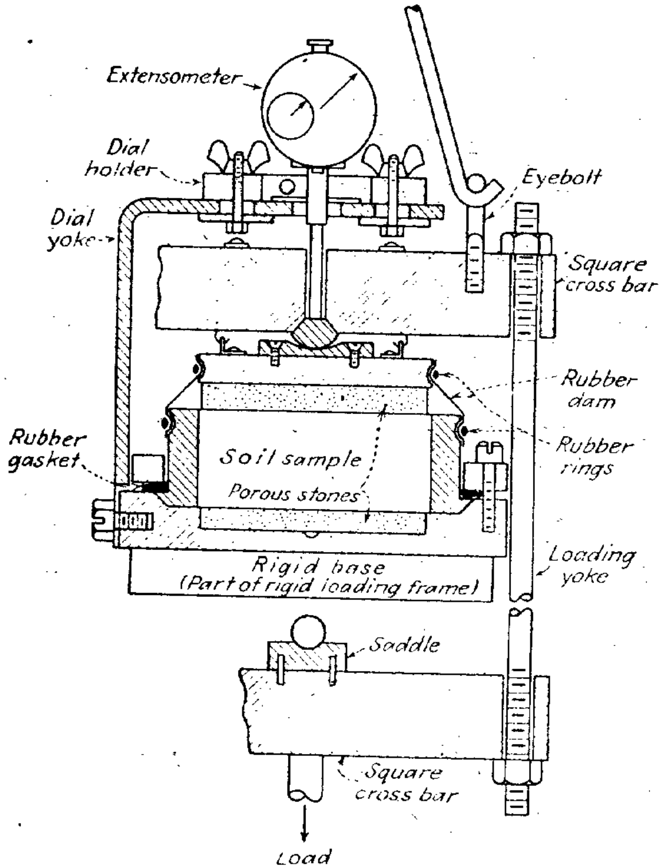
Determination of the Density/Moisture Relation of a Soil

Proctor Compaction Test. The apparatus used consists of a cylindrical metal mould with a detachable base and a detachable collar which fits on the top. The soil specimen is brought to a certain moisture content and compacted into the mould with a specified number of blows from a standard rammer. The dry density of the soil is calculated and procedure repeated with increasing moisture contents and a curve is plotted, until maximum soil density is obtained.

This test is made to determine the moisture content at which the soil should be compacted to obtain the maximum dry density, and the dry density likely to be achieved by compaction in the field.

The density of a soil can also be determined in the field by the *sand replacement method* in which a cylindrical hole of about 100 mm diameter is scooped out of the soil and then filled with sand of known bulk density. The volume of the hole is then computed from the weight of the sand required to fill it and the soil scooped out is weighed and its moisture content determined. A number of tests should be made.

Moisture Content Determination. A number of methods of determining soil moisture content both in the laboratory and in the field have been developed of which the most common are : Oven Dry Method ; Sand-bath Method ; Pycno-meter Method ; Density Method.



Consolidation-test apparatus

Measurement of Compaction in the Field. The usual method of measuring compaction in the field is to determine the dry density of the soil *in situ*. There are four main methods of making this determination, and the procedure involved in all cases is to determine the weight and moisture content of soil removed from an approximately cylindrical cavity whose volume is then measured. These Methods are : Core Cutter Method ; Sand-replacement Method ; Volumenometer Method ; Rubber-Balloon Method ; Water Displacement Method.

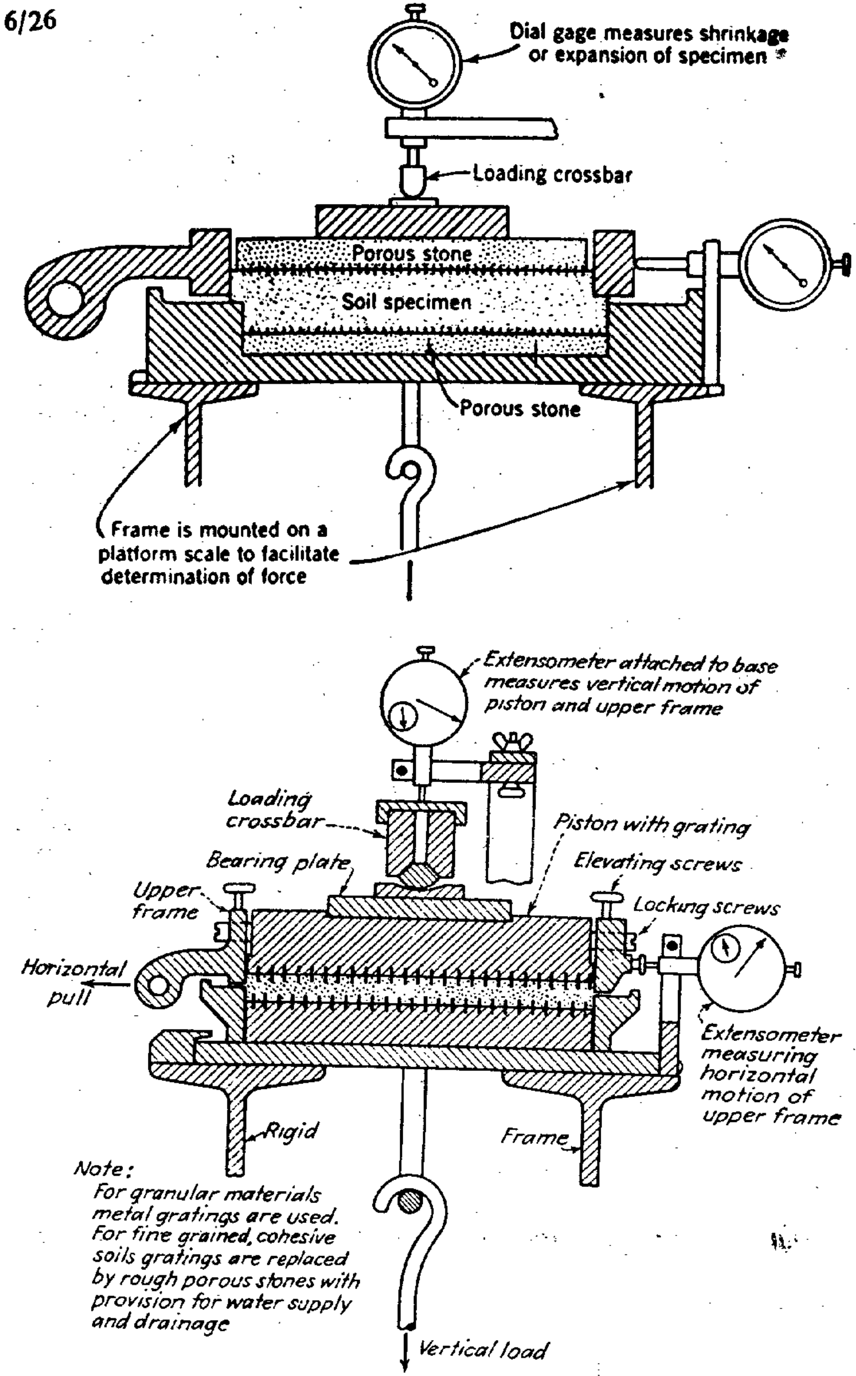
Consolidation Test is performed for the purpose of determining the total volume decrease as well as the time rate of volume decrease which a laterally confined soil sample will undergo when subjected to an axial load. The consolidation test is useful in all problems associated with settlement and is used principally with cohesive soils. This test is essentially a confined compression test.

An undisturbed sample of the soil is placed in a metal ring between two porous stones to facilitate drainage of the sample during the test, and a pressure applied to the upper stone. The magnitude and rate of compression of the sample under the pressure is observed. When equilibrium has been attained the load is increased to a greater value and the observations repeated. The volume change of the sample is measured by an extensometer. Each increment in the load is allowed to act till no further contraction takes place under it. For each increment of load the progress of volume change corresponding to appropriate time intervals is recorded. The data thus obtained is plotted in the form of a dial reading virsus time-curve. Time required for consolidation is proportional to the square of the thickness of a soil stratum. Since the time rate of consolidation is a function of the permeability of a soil, it determines the co-efficient of permeability indirectly.

Liquid Limit Test. The soil sample weighting 30 grams is placed in a porcelain evaporating dish about 115 mm dia., shaped into a smooth layer approximately 10 mm thick at the centre and divided into two portions by means of a grooving tool of standard dimensions. The dish is held firmly in one hand and tapped lightly ten times against the palm of the other hand. If the lower edges of the two soil portions do not flow together the moisture content is below the liquid limit. If they flow together before ten blows have been given, the moisture content is above the liquid limit. The test is repeated with more or less moisture as the case may be until the two edges meet exactly after ten blows have been given.

A mechanical device, called "Crank and Cam Device" is used in most of the laboratories. In this device a brass cup is raised 1 cm above a flat base and then dropped by rotating a handle. The LL is the moisture content when the soil sample flows together for 12 mm along the groove with 25 shakes at 2 drops per sec. Weight of the sample is 100 grams and passes the 425 micron (No. 36 BS) sieve.

$$LL = \frac{\text{weight of wet soil} - \text{weight of dry soil}}{\text{weight of dry soil}} \times 100$$



Apparatus for Shear Test

Plastic Limit Test. A sample weighting about 15 grams is taken from the material passing the 425 micron sieve and is thoroughly mixed with water on a glass plate until it is plastic enough to be rolled into a ball. The ball of the soil is then rolled between the hand and the glass plate so as to form the soil mass into a thread. When the diameter of the thread becomes less than 3 mm, the soil is kneaded together and rolled out again. This process is continued until crumbling of the thread occurs at a diameter of 3 mm. The portions of the crumbled soil are gathered together and the moisture content of this soil determined.

Testing Soils for Strength

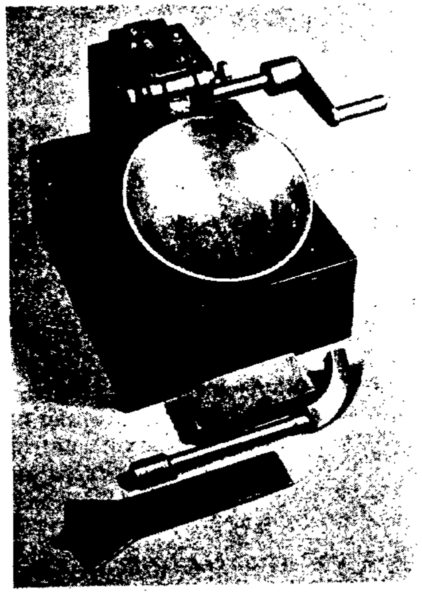
The behaviour of soils under loading is very complex. Soils of different types differ considerably in their resistance to deformation when stressed and such deformation depends upon the moisture content, bulk density, angle of internal friction of the soil, and the method in which the load is applied. Furthermore, all the affected soil beneath a foundation is seldom homogeneous and large variations in strength may occur in both the vertical and horizontal planes.

The tests used to determine the strength properties of soils can be divided into three broad groups : Shear tests, Bearing tests, and Penetration tests.

Shear Tests. The object of shear tests is to determine (*i*) the ultimate bearing capacity of the soil mass for the design of footings and other foundations, (*ii*) the stability of earth slopes, (*iii*) the estimation of earth pressure on retaining walls, footings and sheet piling, etc., and (*iv*) for the design of thickness of airfield and road pavements. It determines the values of the apparent cohesion and angle of shearing resistance of soil under known test conditions. All stability in soil is derived from shearing strength. The shear resistance is composed of two parts, the resistance of soil grains to sliding over each other and the cohesion existing between the soil particles. The resistance to sliding is dependent upon the angle of internal friction. A granular soil develops friction between the soil particles only under the application of a normal load but in cohesive soils there is resistance against sliding even when no normal load is applied. This resistance against sliding of the cohesive soils is called *cohesion*. Clays have a resistance to shearing due to their cohesive strength. Shearing strength is dependent on and varies directly as the co-efficient of friction and cohesion of the soil. To understand the shear strength of soils is one of the most complex problems of the soil mechanics.

There are several methods of testing the shear strength of a soil in a laboratory, the most common being (*i*) Shear Box test, (*ii*) Triaxial test, (*iii*) Unconfined Compression test, and (*iv*) Vane test.

Shear Box Test. In the shear box, failure is caused in a pre-determined plane of the soil, the shear strength or shearing resistance and the normal stress both being measured directly, as it is a direct shear machine. The essential feature of the apparatus is a rectangular box divided horizontally into two halves, the lower half box is fixed and the upper half is movable. The soil to be tested is enclosed in the two half boxes.



LIQUID LIMIT DEVICE AND TOOLS

DETERMINING THE PLASTIC LIMIT OF SOIL

and porous stone plates or metal plates are placed above and below the specimen. While a constant vertical compressive force is applied, a gradually increasing horizontal force is applied to the upper half of the box, thus causing the soil prism to shear along the dividing plane of the box. This measures the horizontal load required to shear a soil corresponding to any vertical normal compressive load. The test is repeated on other identical specimens under different vertical loads and the results are plotted as shearing resistance against normal vertical load (shear stress is plotted vertically and the normal stress horizontally) and straight line is drawn through the points. The equation of this line is :

$s=C+n \tan \phi$ where :

s=horizontal force divided by the area A of the cross section of the soil specimen, *i.e.*, the unit shear resistance ;

C=cohesion per unit area=the horizontal shear force under no vertical load. Cohesion for a granular soil (dry sand) is zero. Can be read off from the graph ;

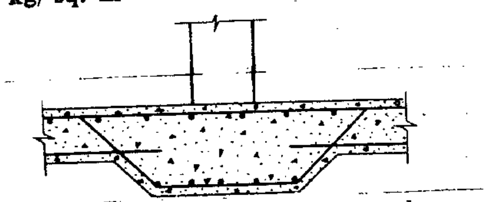
n=vertical normal load per unit area ;

ϕ=angle of shearing resistance or the angle of internal friction. Can be read off from the graph.

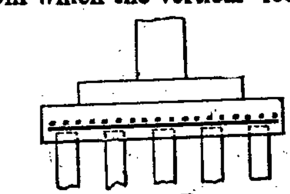
In the case of undrained saturated clays the angle of shearing resistance is zero. The true angle of internal friction of clay is seldom zero and may be as much as 26 deg.

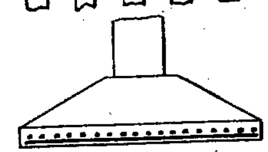
Direct shear tests are of two kinds : (*i*) Immediate tests, in which the horizontal load is applied as soon as the normal vertical load begins to act, and the specimen is enclosed between metal plates. (*ii*) Slow tests, in which the soil is allowed to consolidate completely under each increment of vertical load. The specimens are enclosed between porous plates which allow the soil to drain.

The inter-relatlonship between cohesion, internal friction and stability are determined from the above equation. The unit shear resistance is composed of two parts, that furnished by the resistance of soil grains to sliding over each other and that furnished by the cohesion existing between the soil particles. By experiments the cohesion C in kg/sq. m and ϕ have been ascertaind as given in the table below, from which the vertical load n in kg/ sq. m can be computed :



Increased depth may occur at column base or be a band continuous between columns.





Soil	C kg/sq. m	ϕ Deg.
Clay liquid	490	0
,, very soft	980	2
,, soft	1950	4
,, fairly stiff	4880	6
,, stiff	7300	8
,, very stiff	9800	12
Silt	0	20
Sand wet	0	10
,, dry or unmoved	0	34
,, predominating with some clay	1950	30
Cemented sand and gravel, wet	2450	34
Sand—gravel mixture cemented with clay, dry	4880	34

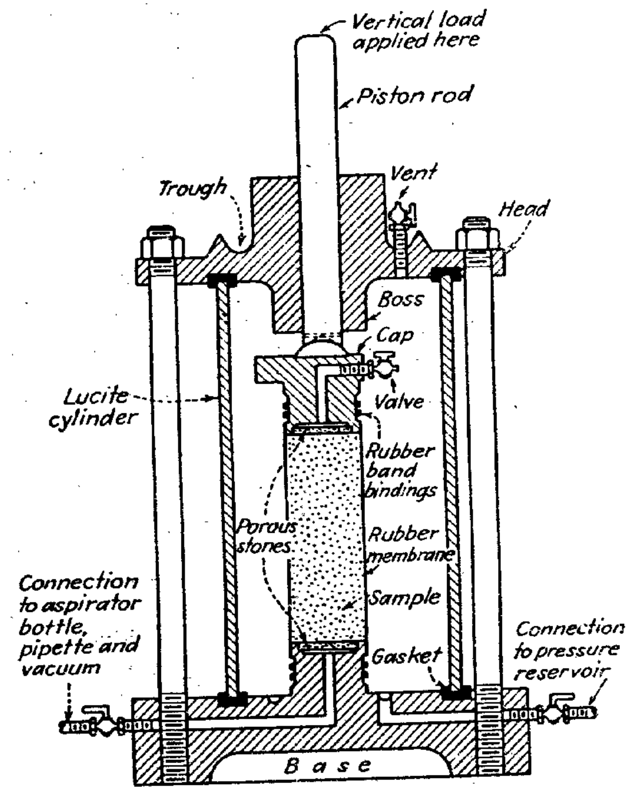
(A safety factor of 2 should be applied to the values obtained)

	Dry Density grams/cu. cm	ϕ Deg.
Compact well graded sands and gravel-sand mixtures	1.76—1.92	40—45
Dense well graded gravel	1.84—2.00	45—50
Loose well graded sands and gravel-sands mixture	1.60—1.76	35—40
Dense sands	1.68—1.84	35—46
Compact uniform sands	1.60—1.76	35—40
Loose uniform sands	1.44—1.60	30—35
Loose fine sands	1.44—1.60	28—34

Immediate shear strength of clays is taken as follows :—

Type of Clay	Shear Strength in kg/sq. m
Very stiff boulder clays and hard clays	greater than 14650
Stiff clays and sandy clays	14650—7300
Firm clays and sandy clays	3660—1830
Very soft clays and silts	less than 1830

The soils subject to the higher normal stresses will have lower moisture contents and higher bulk densities than those subjected to lower normal stresses and will thus have increase in shearing resistance and cohesion with increasing normal stress.



Apparatus for Triaxial-Compression Test

Triaxial Compression Test. This test is used where more precise values of the cohesion and angle of internal friction of a soil are required than determined from a shear box test. The specimen of the soil is subjected to three compressive stresses at right angles to one another, and one of these stresses is increased until the specimen fails in shear. The test differs from the shear box test in that the stresses determine the plane of shear failure which is not predetermined.

In this test a cylindrical specimen usually 40 mm dia. and 75 mm long is enclosed in a thin rubber membrane and is subjected to radial fluid (water or glycerine) pressure. Increasing axial stress is applied at the top until failure occurs. The test is repeated with different pressures and the results are plotted in the form of Mohr's circles. The triaxial apparatus is probably the most useful for research into the fundamental properties covering the strength of soils but is elaborate.

The undrained triaxial test is, in general, used as a basis for estimating bearing capacity, earth pressure and slope stability of cohesive soils. Unconfined compression test is used for predominantly clayey soils which are saturated or nearly saturated.

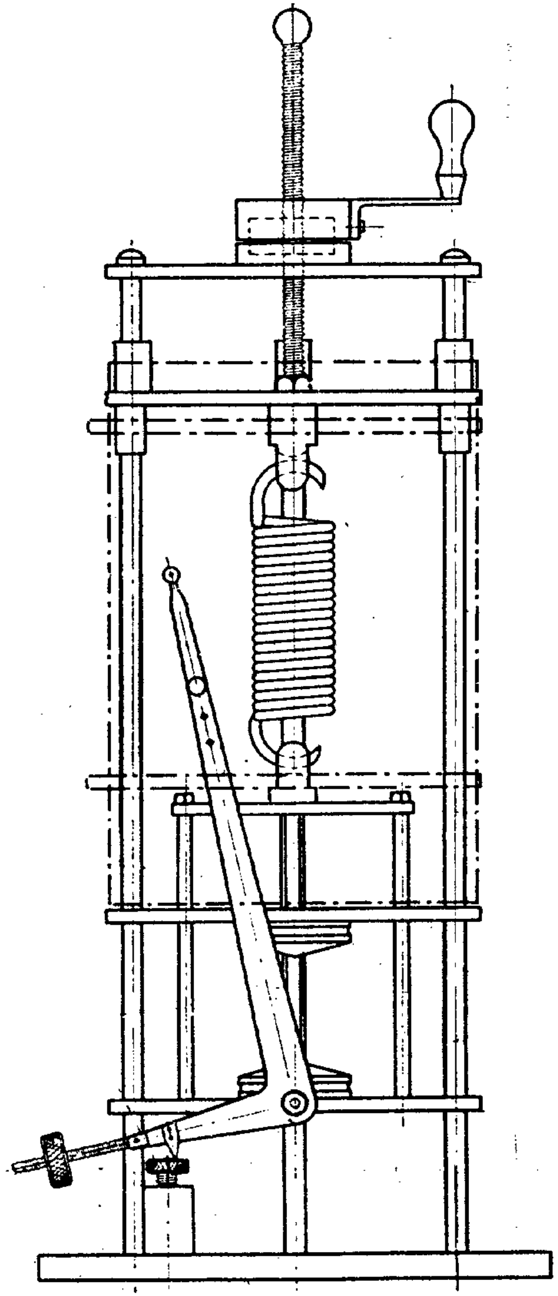
Bearing Tests. Are loading tests made in the field on the surface to know the elastic and unrecoverable compressibility of the soil while *penetration tests* are made to know the resistance of the soil to deformation by shearing. Such tests may be used in order to determine the ultimate bearing capacity and load/settlement characteristics of the soil under the foundations. Bearing plates are generally used for bearing tests and have been described under "Loading Tests on Foundations" in the following pages.

It is usually assumed that the effectively stressed zone beneath a loaded area extends to a depth of 1.5 times the width of the area. Bearing tests should only be carried out on material which is homogenous to the depth which will be stressed by the proposed structure, and in the case of non-homogeneous materials tests should be carried out on each stratum in turn. If the surface stratum is underlain by softer materials with lower bearing capacities, *e.g.*, a sand bed over soft clay, loading tests on the surface will indicate an excessive bearing capacity and low settlement.

The pressure from a load is rarely distributed evenly over the whole base of the footing, but for rigid bases, the distribution of pressure is quite different according to whether it is founded on cohesionless or cohesive soils. On sands, the bearing pressures are higher at the centre than at the corners and edges, whereas on clays the higher pressures come at the edges and corners. But for practical purposes the total load is assumed to be distributed evenly over the whole base of the footing.

Unconfined Compression Test is similar to a compression test performed on concrete cylinders and is made by applying an axial load to cylindrical or prismatic soil samples and measuring the deformation corresponding to the stress as the load is increased. When the stress reaches the ultimate strength, the sample may fail either by a grandual bulging or by a sudden rupture. The ultimate strength of a test specimen prepared from an undisturbed sample (at unaltered moisture content), is a relative measure of the ultimate bearing capacity of a soil. A comparion of the stress-strain characteristics of a soil tested in the undisturbed state, and then in the remoulded state at unchanged moisture content, is indicative of the structural damage caused by remoulding. A simple apparatus intended for field use has been developed. This test is generally the most convenient for immediate tests on saturated or nearly saturated clays.

The Vane Test. Is a field shear test for clays in which a vane consisting of two or four blades fixed at right angles, is attached to the end of rod and pushed into the soil at the bottom of a borehole. The torque required to cause rotation or shear the soil is measured. This torque is approximately equal to the moment developed by the shear strength of the clay acting over the surface of the cylinder with a radius and height equal to that of the vanes. This test has the advantage over the unconfined compression test in that the shear strength of a soft and sensitive clay at

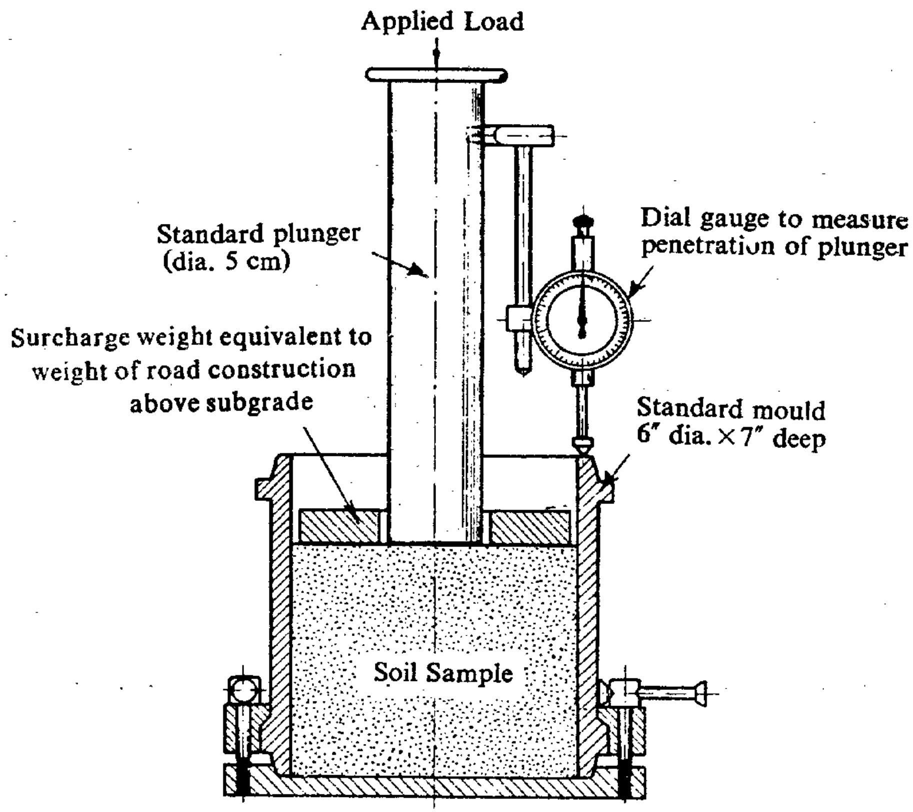


UNCONFINED COMPRESSION APPARATUS

considerable depths (say up to about 30 m) can be determined *in situ* without obtaining undisturbed samples. This test is still in the process of development.

Penetration Tests are small scale bearing tests and determine the strength properties of a soil mass. The most common being :

The C.B.R. Test which is an *ad hoc* penetration test developed by the California State Highways Deptt. (USA) for the evaluation of sub-grade strengths. It is a measure of the shearing resistance of a soil to penetration under controlled density and moisture conditions. The strength of a soil is found by causing a plunger (or piston) of standard size to penetrate a specimen of the soil prepared to the density and moisture conditions of the soil to be tested in a standard mould. The resistance to penetration is measured and then expressed as a percentage of the known resistance to penetration of the plunger in a crushed aggregate. The California Bearing Ratio Test can be made on nearly all soils ranging from clay to fine sand. This test is generally used for the design of road pavements.



CALIFORNIA BEARING RATIO –C.B.R.– TEST APPARATUS

North Dakota Cone Test. It is a cone penetration test similar to the C.B.R. developed by the North Dakota State Highways Deptt. This test is simpler and more rapid than the C.B.R. but its use is restricted to fine-grained soils and is considered reliable only for clayey soils. The penetrometer consists essentially of a shaft with a sharp cone attached to one end. The cone is loaded during the test by placing weights on a disc fixed to the top of the shaft and its penetration measured. The test is made directly on the sub-grade.

There are several other forms of penetration tests such as : **Cone Penetration Test** and **Proctor Penetration Needle Test** which are, in general, only of use in cohesive soils. These penetration tests are in the nature of small-scale loading tests and only measure the strength of the soil in the immediate vicinity of the soil. They do not measure directly any fundamental property of the soil, but it is possible to co-relate the results with the shear strength or density and moisture content for each particular cohesive soil.

Load Bearing Capacity of Soils

The bearing capacity of a soil depends upon the physical characteristics of the soil particles, (*i.e.*, size, shape, cohesive properties, frictional resistance and the power to retain moisture, etc.), moisture content and the changes brought in by the atmospheric influences (heat, rain, etc.). The finer the soil particles, the more variable are the cohesive and frictional properties of the soil under field conditions. In general, the heavier the unit weight of soil the greater the strength, and also the lesser the voids, the greater the strength.

With structures built on sands and gravels the settlement is likely to be partically completed at the end of construction, but when the site is underlain by clays or silts, settlement is likely to continue for a long time after construction and cracks may appear many years after completion.

All foundations settle under load and the general tendency is for some parts of a structure to settle more than others causing relative movement. The critical factor in the settlement of a structure is not the *amount* of settlement but the *differential* settlement between the different parts of a structure itself. Excessive pressure is comparatively uncommon cause of settlement.

Investigation of all layers under a foundation should be made as even thin layers which are weak in shear can cause settlement.

Load Bearing Properties of Rocks

Rocks have a high safe bearing capacity except when decomposed, heavily shattered, or steeply dipping. On a non-level site dangerous conditions may develop with stratified rocks if they dip toward cuttings or deep basements. Reduction in the permissble load should be made if the beds are steeply inclined. Slips or cleavage planes may cause trouble if there are cuttings or deep basement close to the foundation. With limestones, the possibility of caves or swallow-holes should not be overlooked.

Load Bearing Properties of Clayey Soils

The most marked characteristics of cohesive soils from the engineering standpoint is their susceptibility to slow volume changes. All soils containing greater proportion of clay expand in bulk when wetted and contract on drying, the volume changes cause both vertical and horizontal movements of the ground, the vertical movement being generally the more important. Shrinkage and expansion of clay of high liquid limit may be very detrimental. When subjected to load, even if the pressure is below the maximum safe bearing capacity, some of the contained water is squeezed out with consequent diminution of volume, resulting in consoli dation settlement. The possibility of these movements is an important consideration even for light buildings. The behaviour of clayey soil is most treacherous when in great thicknesses as the total settlement depends on the thickness and compressibility of the layers. Care is also necessary when building on relatively steep slopes of clay, since downhill creep is appreciable and often leads to serious trouble.

Settlement on foundations of clayey soils increases with the size of the footings and some soils are of such a nature that settlements do not take place immediately after the load is applied, there is a time lag in the settlement following the loading. Clays settle somewhat if reloaded and may swell if load is removed ; there are also definite periods of time for these settlements to become complete. Shear failure in the stiffer clays is immediate, but some soft plastic clays fail very slowly and short term loading tests on them may be deceptive.

Load Bearing Properties of Sandy Soils

Because of the internal frictional resistance of granular soils their shearing strength depends largely upon the magnitude of the applied load, which increases in relation to the normal pressure to which they are subjected. The bearing capacity of sand increases under heavy load. Sand is almost incompressible if compact and kept confined, and has great crushing strength. Settlement in sand occurs immediately the load is applied (whilst it is slow in clay) and it may still contain water in the voids. Wet sand has less bearing power than dry sand and fine sand has less bearing power than coarse dry sand.

Loose uniform sands are apt to be treacherous since they suffer very appreciable settlement and loss of stability if subjected to vibrations which cause them to fall into more dense packing. This has been explained under "Quicksand formation." When designing structures on such soils, unit load should be reduced as much as possible and never more than 22 t/sq. m, and the foundations should be so confined that there can be no escape of any material from below the foundations. The density of the soil *in situ* should be compared with its "critical" density which can be determined in the laboratory. No danger exists if the natural density is greater than the critical.

Load Bearing Properties of Silts. Silts are generally found in loose state but if dense they are not necessarily a poor foundation. Micas, diatoms and organic matter are elastic and rebound when pressure on them is removed and since they cannot be permanently compacted, they make bad subgrades.

Field Methods of Exploration of Foundation Strata Site Exploration and Soil Survey It is desirable to make a preliminary reconnaissance by walking over the ground or the site to obtain a broad indication of the work required. A soil survey comprises the exploration of soil conditions over the site by boring or other means, and preparation of sections indicating the soil profiles and the ground water levels.

Extent and Depth of Exploration. A soil survey should be sufficiently extesive to furnish an idea of the degree of variation of the soil in both horizontal and vertical directions. This is particularly important where there can be differential movements of one part of the structure relative to another due to variation of loads as differential settlement between the adjacent parts of structure are more important than total settlement of the structure as a whole. The position of a soft layer relative to the surface is important as frequently a deep seated soft layer has been the cause of movement. The large settlements occurring with structures founded on compressible soils are due almost entirely to consolidation of the underlying strata. The thicker the soft layer the greater will be the ultimate movements due to consolidation. A close study of the hydrostatic pressure (uplift) and the changes in gronund water level due to weather conditions are of great importance as these factors modify the stress-distribution appreciably. Presence of water-bearing sands or silts in beds of clay are particularly important where dams or cofferdams are to be constructed.

As a result of research it has been indicated that under the loaded area the stress is about one-fifth of the applied pressure at a depth equal to about one and a half times the breadth of the loaded area. Hence the larger the loaded area the deeper is its influence felt. The larger the footing, the greater the settlement, because the area of the pressure will increase in proportion to the size of the footing. (Relative settlement is proportional to the depth.) It does not, however, increase the danger of failure because the intensity of shearing stress is not greater in the deeper region. If a number of footings are in close proximity, the effects from each are additive. The shear distribution indicates that the maximum shear stresses are appreciable at depths of one-half to one and a half times the breadth of the footing. Therefore, it is necessary to explore the foundation strata to greater depths than these.

For roads and runways, borings 1.5 to 3 m below the proposed formation level are generally sufficient ; for footings a depth of approximately 1.5 times the width of footings is required, and for dams a similar depth is generally needed.

For ordinary building structures it will usually be sufficient precautions, after having dug and levelled off the foundation pits or trenches to a depth of 0.9 to 1.5 m to test them by making a few holes to a depth of about 1.2 to 2.4 m, or by sounding with an iron rod and measuring the resistance to penetration, to ascertain if the soil continues to be firm to that distance. Any ordinary building with a stratum of firm soil from about 1.5 to 2.4 m thick below the foundation bed will generally be safe against settlement.

Sounding and Probing. A most simple method of soil exploration is by means of sounding rods which is suitable for shallow foundations in common soft soils. A steel rod or bar of about 20 to 30 mm diameter pointed at one end and threaded at the other to receive additional lengths which are joined together by couplings, is forced vertically into the ground with blows from a hummer and turned after each blow. The relative hardness of the strata penetrated can be judged by the resistance to penetration durring driving. It may also determine the ground water level and a little practice will enable one to distinguish sandy from clayey soils by the sound given out when the bar is twisted. There will be some penetration through all soils except when rock is met with. On a rock the rod will quiver when struck and make a different sound. To avoid mistaking a large boulder for soild rock, further soundings should be made nearby. This is only a rough guide method which does not produce a soil sample. Soundings up to about 9 m can be made in this manner.

Another better method is by driving in a hollow tube with open end of 40 to 50 mm diameter for about 30 cm each time and withdrawing it for examining the material caught in the tube. The tube is split at the bottom for about 45 cm with a slit of 30 mm to facilitate removal of the material. A depth of about 3.6 to 4.6 m can be penetrated by this method.

Trial pits are preferable to shallow bore holes in dry and stiff ground which requires little support as they give a more accurate idea of the strata in their undisturbed state. They are very commonly used for exploration to depths up to 3 m to 6 m beyond which the relative cost of of pit sinking to that of boring will increase. The type of soil within a few feet of the bottom of the pit may be determined by exploration with an earth auger or by driving a tube to the additional depth.

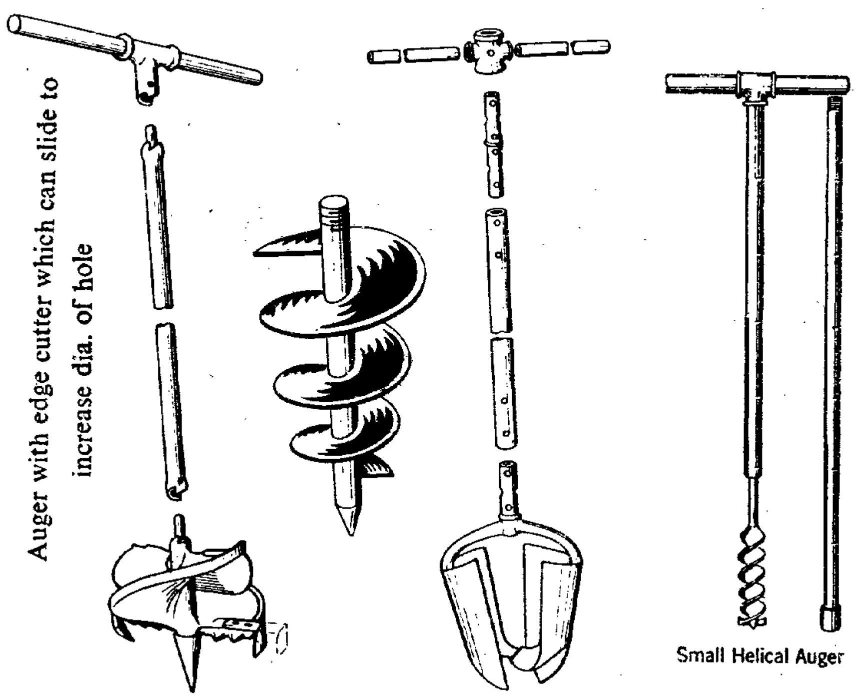
Boring is especially useful where it is suspected that a low bearing soil underlies a hard bearing soil. Borings will also determine variations in ground water pressure due to flood conditions or draughts. If water is found in a bore-hole, the hole should be left for 24 hours for the water to rise to its final level.

Hand Augers

Post-hole augers. This simple tool is used for putting down holes to a depth of about 6 m in soft soils which will stand unsupported but it may also be used with lining tubes if required. Augers of 100 or 125 mm dia. (150 mm max.) have been found most successful ; for very dry noncohesive soils or for ground below water-table level, augers with self-closing flaps are especially useful. In cohesive soils free of stones the common *wood auger* or *corkscrew* 40 mm dia. may be used. These augers are turned by two men with 0.91 m long handles. Samples

of the soils brought out are examined. About 15 to 30 m of boring can be done in a day. Where auger boring are made without the use of casings it is often impossible to determine from which soil stratum a given auger sample came as the sides of the hole sometimes cave in, and the soil picked up by the auger at the bottom of the hole will be the soil that has fallen from the sides of the hole at some higher elevation.

Shell and auger boring. The tools consist of augers for clay, and shells or sand pumps for sandy strata ; these are attached to sectional boring rods. Hand rigs can be used for vertical boring up to 20 cm in diameter and to 24 m in depth. Small boulders and thin strata of rock can be broken up by a chisel bit attached to the boring rod, or the auger may be supplemented by a crowbar in shallow borings. The boring rods are raised or lowered by means of shear legs and a which and are turned by hand ; casing is driven by means of a 'monkey' suspended from a winch.



Shell auger can be manufactured locally from a hollow tube of about 1.2 m length and 100 mm diameter with cutting edge at bottom and split along the length for about 60 cm. The top end is bent so that a square rod of 40 mm × 40 mm can be fixed with a round threaded end for attaching lengthening pieces. It is used like a screw and driven down into the soil with the help of a handle.

Where the testing has to be made to a considerable depth, ordinary well-boring methods can be used. In firm ground no precautions may be necessary to pervent the sides caving in but if boring passes through a stratum of quick-sand, marshy earth, etc., a casing of pipes must be provided. The pipes generally used are from 3.7 to 4.6 m long and of varying diameters from 75 mm upwards, so as to pass one within the other. They are provided with bayonet joints or screw joints. The largest pipes are first placed in the boring, successive lengths being added at the top until the required length is got in and are forced down if necessary by a monkey or a small pile engine acting on a block of wood fitted on the head of the pipe. When the resistance of the earth prevents the pipes being driven any further, another pipe whose external diameter is equal to the internal diameter of the pipe already placed (but slightly less), is lowered. The earth inside of the pipe may be removed by a small scoop with a long handle. For deep borings the rods are replaced by a rope, one length of rod only being used next the chisel and boring done entirely by jumping. Great care must be taken to prevent breaking of the boring rods.

(Boring methods have been described in detail under "Tube Wells".)

Undisturbed (with no change in moisture content) samples of the soil are taken out to measure the shear strength, compressibility and density of the soil which are the three most important properties of a soil.

LOADING TESTS ON FOUNDATIONS

For testing the bearing capacity of a soil the conditions of the moisture content and dry density of the test area should be those which are likely to exist when the structure has reached a state of relative equilibrium subsequent to the construction of the structure. In area subject to water-logging and floods the bearing capacity of a soil should be determined under the wettest conditions that are likely to occur and the soil to be tested should be brought under such a condition by soaking, and the basis of the design should be the strength of the wet soil. Clayey soils are most likely to suffer by water absorption. Soaking can be done by making a small pond of bricks over the area to be tested and keeping it filled with water till the oil attains the expected moisture conditions; required degree of compaction can be obtained by hand tamping in thin layers.

Bearing Plate. In general, iron plates either 60 cm sq. or 75 cm dia. and 16 mm thick are used. Smaller plates of 12 or 45 cm dia. may be used where much accuracy is not desired. Size of the plate should not be less than 1/5th of the width of foundation trench. The plate is placed in a pit dug below the bottom of the foundation trench. Where the test plate is to be placed on the surface (for road works), it is preferable to remove the top 23 cm of the natural soil before placing the plate. It is very important to seat the plate accurately over the area and the ground should be levelled as much as possible. The plate should be rotated over the area and any irregularities over the surface trimmed off; the plate should be in contact with the soil over all its area. On coarsely-grained soils which are difficult to level accurately, the plate can be seated on a layer of fine dry sand 6 mm thick.

The load can be applied to the test plate either by the actual superposition of load or by jacking against a reaction. The following simple method of loading the plate may be adopted : Load the soil four times the proposed design load and read settlement every 24 hours until no settlement occurs in 24 hours. Add 50 per cent more load and read settlement every 48 hours until no settlement occurs in 48 hours. Settlement under the test load should not show more than 20 mm or increment of settlement under 50 per cent overload should not exceed 60 per cent of settlement under test load. If the above limitations are not met, repeat test with reduced load and in which case the reduced load will be taken for the safe loading capacity of the soil. At least two tests should be carried out, preferably, with different size of plates.

A small area will stand a heavier unit pressure for a short time than a larger area permanently, therefore, the test area should be as large as practicable, and the test continued for as long a time as possible.

It is usually assumed that the load spreads out in the underground in the form of a truncated pyramid whose sides slope at an angle of 45 deg. The high load intensity under the test plate at the ground surface decreases rapidly with the depth in the underground and practically vanishes at a depth amounting to only a few times the diameter of the loaded area. In small-scale loading tests the stresses due to the test load are confined only to the top few feet of the soil and reflect only the properties of the soil a foot or two below the test plate and give no idea of the properties of deeper layers. Since the depth of the stressed soil depends upon the width of the footing, the loads from wide footings will produce high stresses in the underground up to considerable depth. As such, the load test on the small area can give a completely misleading answer as to the ability of the soil to support the structure safely. The soil-load test alone is, therefore, not a sure test of the bearing capacity of a soil, these tests much be accompanied by suitable borings.

2. LOADS AND WEIGHTS ON FOUNDATIONS

Safe Loads on Common Soils

Rocks	tonnes/sq. m	*Rocks*	tonnes/sq. m
Hard rock	above 220	Moorum	20 to 45
Ordinary rock	above 110	Clay shales	110
Sandstone	130 to 220	Marl & firm shale	65
Limestone	100 to 200	Hard chalk	45 to 65
Soft rock	20 to 90	Soft chalk	17

Intensity of pressure on rock foundation should at no point exceed one-eighth pressure which would crush the rock.

Cohesive soils :

Very stiff boulder clays	65
Hard or stiff clays and sandy clays ...	30 to 44
Firm clays and sandy clays	20
Ordinary clay	20

Sand and clay mixed or in layers	...	20
Red earth	... ...	30
Moist clay	... ...	10 to 20
Soft clays and silts	... ...	10
Very soft clays and silts and peat	...	5 to nil
Black cotton soil	... ...	5 to 10
Alluvial soil	... ...	3 to 9
Alluvial loams	... ...	9 to 17
Made ground (consolidated)	...	5
Hoggin (compact)	... ...	65

-cohesive soils :

Compact gravel or sand well cemented	...	55 to 80
Compact gravel or sand and gravel	...	43 to 55
Loose gravel or sand and gravel	...	30
Compact coarse sand (confined)	...	45
Loose coarse sand	... ...	20
Compact fine sand (confined)	...	32
Loose fine sand	... ...	10
Sand with clay	... ...	20
Kankar	... ...	32

Note :

(*i*) The above values are only approximate and the allowable bearing pressure for individual soils may differ considerably. The figures have a factor of safety of 2 to 3.

(*ii*) If the ground water level in sand or gravel soils is likely to approach foundation level the safe bearing pressure should be reduced to about one-half the values given.

(*iii*) For accentric loads the maximum safe pressures may exceed by about 10 per cent.

(*iv*) The safe bearing pressure can be exceeded where the foundation is taken well down in the ground by an amount equal to the weight of the material which is displaced by the foundation itself.

In the case of non-cohesive soils the bearing pressure may be increased by one-eighth of a tonne for each 30 cm of depth of the loaded area below the lowest ground surface immediately adjacent.

For foundations supported on cohesive soils the settlements of footings for a given unit pressure increase with the linear dimensions of the footings. There would therefore be a different allowable pressure for each size of footing on a given cohesive soil if uniform settlement were required.

(*v*) The ultimate bearing capacity of soils under long rectangular footings should be taken only 3/4th of the bearing capacity under square footings.

Average Weights of Soils and Masonry in kg/cu. m

Earth, dry to wet	1600—2400	Brickwork, ordinary	1800—1900
Sand, dry to wet	1450—2000	„ sundried	1600
Sand and clay	2000	Stone masonry	2550
Gravel	1450	Dry stone masonry	2080
Gravel and sand	1750	Lime concrete	1800
Silt, dry to wet	1600—1750	Cement concrete	2080—2400

Superimposed (Live) Loads on Floors—kg/sq. metre

1.	Floors for residencies, hostels, hospitals, etc	200*
2.	Floors for offices, schools, assembly halls, banking halls, etc.	250—400*
3.	Floors for factories, warehouses, etc., for light weight loads and office rooms for filing and storage	500
4.	Floors for factories, warehouses etc., for medium and heavy weight loads	750—1000
5.	Stairs, landings, corridors of residencies, balconies, not liable to over-crowding	300
6.	Ditto. subject to over-crowding	500
7.	Telephone exchange rooms, transformer rooms	1000

*Subject to a min: total load of 2.5 times for slabs and 6 times for beams, uniformly distributed. Beams, ribs and joists spaced not more than one metre centres may be calculated for slab loadings. Also see page 11/3.

"Superimposed loads" consist of persons occupying the rooms, furniture, equipment, etc.

Average weight of men is 68 kg each. 5 men stand in a space of one sq. yard (0.84 sq. m).

Experiments show that it is possible for a crowd of people to exert as high a weight as 880 kg/sq. m of floor area. A weight of 680 kg/sq. m is quite possible where there are throngs of people. A load of 400 kg/sq. m is quite frequent in buildings and private houses at social gatherings. A crowd of men pushing against a balustrade may exert a horizontal pressure of about 250 kg/m run when they are three deep.

Minnimum load for slabs of less than 6 sq. m area should be taken as for 6 sq. m and for beams of less than 2.5 m span, load should be taken as for 2.5 m. Beams, ribs and joists spaced at not more than 90 cm centres may be calculated for slab loadings.

Weights to be taken for the Design of Foundations :—

(*i*) Dead loads of : foundation concrete, walls, roofs, floors, projections and any concentrated loads.

(*ii*) Superimposed loads on floors, roofs, staircases, etc. Vertical components of any horizontal thrusts in warehouses or workshops, or from arches.

In buildings where the superimposed load does not exceed 500 kg/sq. m the following reduction is made in the superimposed loads for the design of columns and foundations in multi-storey buildings :—The seperimposed load on the topmost storey is in accordance with the above table but a reduction of 10, 20, 30, 40 and 50 per cent is made on the superimposed load on the 1st, 2nd, 3rd, 4th and 5th storeys respectivcly below the topmost storey, and 50 per cent for all the succeeding storeys where the building consists of more than 6 storeys. It is highly improbable that full load will ever be imposed over all the floors at one time except in the case of warehouses in which full load conditions might occur. Some engineers take only 50 per cent of the total superimposed load on all the storeys except the 1st.

An ordinary single storey residential building imposes a weight on the foundations of about 10 to 15 tonnes per sq. metre, a 2-storey building, of about 15 to 20 tonnes, and a 3-storey building of about 20 to 25 tonnes per sq.metre. A 3-storey building of the heavier type may have a weight of about 40 to 45 tonnes per sq. metre.

3. DESIGN OF FOUNDATIONS FOR BUILDINGS

Depth of Foundations—Minimum depth of foundation is given by :

$$h = \frac{p}{w}\left(\frac{1-\sin\phi}{1+\sin\phi}\right)^2$$ Rankine's formula applicable to loose soils.

Where : h=min: depth of foundation in m below ground level ; p = safe permissible pressure on base in kg/sq. m ; w=weight of the soil in kg/cu. m ; ϕ=angle of repose of the soil material.

For tall structures such as chimneys and towers, 3/4th of safe load on the soil should be taken and the depth h increased by 1/3.

For values of $\left(\frac{1-\sin\phi}{1+\sin\phi}\right)$ see Section 7—"Masonry Structures".

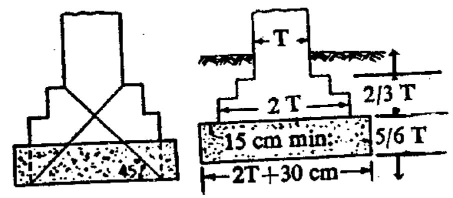
Foundations are generally taken down to about 90 to 120 cm for main walls, 45 to 60 cm for partition walls and 30 cm for boundary walls in ordinary soils 2 to 3-storey building weights. But foundation must be taken down to a firm soil and below weathering effects. When part of a footing is in weaker soil, that part should be taken down deeper and separated (by a gap), or measures adopted for equal distribution of pressures according to the bearing capacity of the respective soils.

Width of Foundation Concrete

For heavy buildings the width of the concrete in the foundation should be determined by the bearing capacity of the soil. The resistance of concrete to tensional and shear stresses is very low, therefore, the projection of the concrete beyond the wall (or offsets) must be so limited, and depth must be sufficiently great, as to prevent cantilever action and to give the maximum shearing area. Crushing tests on cubes of concrete show that the plane of greatest shear occurs at an angle of 45 deg. It is cousidered usually sufficient to fix the depth of the concrete by drawing lines at 45 deg. from the base of the wall to intersect the side of the

concrete bed as shown in the illustration. Projection of the concrete beyod the offsets should be not less than 15 cm on either side since the edges are not generally well consolidated, and its thickness should be not less than its projection beyond the offset and in no case less than 15 cm. Minimum size for 1:3:6 cement concrete (or good lime concrete) wall foundation would be twice the thickness of wall plus 30 cm for the width. The illust.ations show "rule of thumb" for ordinary buildings.

Concrete in Foundations



Cement Concrete. Mass-concrete foundations in good ground stressed in direct compression only, should not be leaner than 1:4:8, or 1 part cement and 8 parts all-in aggregate. (For cheap buildings, a leaner concrete, as much as 1:8:16 may be used.) Foundations stressed in bending in addition to direct compessions in weak ground should not have leaner than 1:3:6, or 1 part cement and 6 parts all-in aggregate. Tension in the concrete up to 4.2 kg/sq. cm may be permitted.

Lime Concrete. Concrete should be laid (not thrown) in horizontal layers, not more than 20 cm thick, and consolidated until the layer is 15 cm thick. The rammers should weight not less than 4.5 kg (5.4 kg is the usual weight) and not more than 320 sq. cm. Square rammers should be used for the edges. Consolidation is not complete until a skin of pure mortar covers the surface and completely hides the aggregate, and until a stick dropped endways from a height rebounds with a ringing sound. If after ramming the mortar does not come up on the surface, the layer should be rejected and relaid. No mortar or water should be added during ramming. (Also see Section 7).

Where new concrete has to be jointed with old, the lower layer should be swept and washed clean and watered before the next is laid. Where joints in layers are unavoidable, the end of the each layer should be sloped at an angle of 30 deg. Where vertical joints occur in the upper and lower layer they must at least be 60 cm apart horizontally. Concrete should be kept wet for at least ten days and no masonry laid on it.

Test for good concrete : Two days after ramming is completed a hole is made in the concrete and filled with water. If the water stays in the hole the concrete is good, if the water runs through it the concrete is either not properly rammed or insufficient mortar has been used.

Lime concrete is now being rapidly replaced by lean cement concrete. Suitable proportion are given in the Section "Estimating".

Where soft places are encountered reinforcement will be required. Where the 'soft' place is not apparent in the excavation but may develop anywhere under the wall, then the foundation concrete will need reinforcing both at top and bottom of the section. An empirical rule for calculation is to allow for full wall load over a 3 m span fixed at both ends. (This has been further discussed in detail under "Piles" and "Lintels".)

Where the foundations are laid at more than one level, at each change of level the higher foundations shall extend over and unite with the lower foundations for a distance not less than the thickness of the foundations and in no case less than 30 cm.

Strip foundations on *sloping sites* should be on a horizontal bearing and stepped. At all changes of levels they should be lapped at the steps for a distance at least equal to the thickness of the foundation or twice the height of the step whichever is the greater. The steps should not be of greater height than the thickness of the foundation unless special precautions are taken.

Footing for Simple Buildings

It is generally assumed that load from a column or a wall spreads out in the form of a truncated pyramid whose sides slope at an angle of 45 deg., which is called the "angle of dispersion". Therefore, footings should lie within a line drawn at 45 deg. from the outer edge of the wall or column. For masonry foundation, the projection of any footing course should not exceed half the depth of the course. Usually each projection is taken a quarter length of brick but not more than half brick, and laid in header courses with reference to the face of the wall. See figures at page 6/45.

Heavy Foundations. Where heavy loads from single columns are transmitted to a stratum of low bearing value, the footings have to be extended much beyond the 45 deg. line of load dispersion. Such foundations have to be of the form of a grillage, raft, or of reinforced concrete. The footings or projections in this case are designed as cantilevers resisting an upward load equivalent to the pressure on the foundation. The moment to be resisted at the centre plane (centre of the wall or column) is taken $= \frac{W}{8a}(a - b)^2$ which is equal to the moment of resistance of the concrete at the footing, viz : $\frac{fd^2}{6}$, ,from which d can be calculated. This formula applies to masonry walls and columns which are not considered as rigidly fixed at the bottom with the foundation.

Where : W is total load due to the column or wall on the footing ; a is the full width of the footing ; b is thickness of the column or wall ; d is depth of the footing ; f is the bending tensile stress of the footing concrete (which may be taken as 5 kg/sq. cm for a 1:2:4 concrete). Calculation is made for a "unit" length of the footing. See pages 8/59, 60. Depth of the footing concrete block can be fixed for the safe bearing capacity of the soil as follows for a 1:2:4 cement concrete for a particular projection p :

Safe bearing capacity of soil in tonnes/sq. m	5	10	15	20	25
depth d=p×	0.55	0.77	0.95	1.1	1.2

The footing is also checked against "punching shear." This has been further explained under RC Columns.

Design for bending under eccentric loads has been discussed in Section 7.

4. CAUSES OF FAILURE OF FOUNDATIONS AND REMEDIAL MEASURES

(*a*) Unequal settlement of the sub-soil : Masonry should be raised uniformly over the whole area. A slow progress of masonry work makes stronger joints and has more uniform settlement.

(*b*) Unequal distribution of the weight of the structure on the foundations due to eccentricity of loads : In continous wall foundations reinforcement should be provided whenever an abrupt change in magnitude of load or variation in ground support may occur.

(*c*) Horizontal movement of the earth adjoining the structure. This is effective in the case of clayey soils and black-cotton soils. Such soils become soft and swell when wet loosing their bearing power considerably and shrink and crack when dry. (See under "Improving the Bearing Power of Soils.")

(*d*) Atmospheric Action : Rain and the sun are the main agents with the change of seasons. Rise and fall of the sub-soil water level, increasing or decreasing the moisture content which is especially effective when the underground water is near the surface, or in damp soils overlying a layer of porous material like sand ; the sub-soil shrinks or expands causing cracks. Soakage of the rain water in the sub-soil also produces a number of changes as above and sometimes bring in salts which react chemically on the lime and bricks in the foundations and cause them to disintegrate. Underground open drains should be provided to drain out the excess water when the subsoil water level rises. Method has been explained in detail in the Section on "Roads and Highways." Deep foundations with sides (of the trenches) well filled and consolidated with good slope away from the walls given to the ground surface will help against rains. A plinth protection of about 60 to 90 cm width with concrete or flat brick flooring will give further protection. Cement or hydraulic lime should be used with stone or over-burnt bricks up to the plinth level in damp locations.

(*e*) Transpiration of Trees and Shrubs : This is a very important factor which is not generally considered. The drying effects of the transpiration of trees and shrubs are superimposed on the seasonal conditions mentioned above whenever the root system approaches the shallowly founded structures. The root systems of isolated trees generally spread to a radius greater than the height of the tree and have been observed to cause significant drying of fat clay soil to a depth of 3 m. The fast growing trees are especially dangerous and within 5 or 6 years the roots extend to a distance of 15 or 18 m and dry out the clay abnormally below the foundations of the nearest part of the house. Sometimes a permanent depression of the ground is produced during the early period of the rapid growth when the tree demands more water than is available on the ground. Abnormal spells of dry weather have much more serious effects. It has been found that the differential shrinkage below the foundations can be sufficient to produce cracking in the brickwork. Such cracking can be most common at the western and southern sides of the house since these parts receive more radiation from the sun than the northern or eastern sides. The drying which progress from the outside towards the inside edge of the footing causes outward tilting of the walls and corners ; a movement which becomes magnified at the top of wall to 25 to 50 mm in the worst cases. The outward movement of the wall may drag the upper floor and roof with it and cause cracking to spread throughout the interior of the house.

For overcoming the damaging effects of shrinkage the foundations should be taken sufficiently deep. A depth of 90 cm is necessary to avoid cracking in normal brick houses away from fast growing trees. It has also been suggested that fast growing and waterseeking trees should not be planted within 18 m (7.6 m min.) of buildings.

Trees planted alongside roads have caused marked depressions along the edges of the roads and underneath cement concrete paths.

(Based on the research made at the Building Research Station, Watford, England.)

Safety of Existing Structures may be affected by :

(*i*) Excavations in the immediate vicinity which may cause a reduction in support to the structure ;

(*ii*) Mining or tunnelling operations in the neighbourhood ;

(*iii*) Adjacent structures which may impose additional loads on the foundation strata or additional stresses in earthwork and supporting structures ;

(*iv*) Vibrations and ground movements resulting from traffic, piling or explosions in the immediate vicinity ;

(*v*) Shrinkage of clay soils due to weather, transpiration of plants ;

(*vi*) Lowering of the ground-water level by pumping from wells may cause settlement of the ground surface over a wide area ;

(*vii*) A rise in the ground-water level may cause movement of the foundation strata.

5. IMPROVING THE BEARING CAPACITY OF SOILS AND MAKING FOUNDATIONS ON WEAK SOILS

If the foundations are left open for one rainy season it will enable the soil to settle down, and it will also be known whether the natural movements of the soil below due to increment of moisture are likely to cause any damage. Foundations in bad soils can be improved by :

(*a*) Increasing the depth of the foundation except when the material grows wetter as the depth increases.

(*b*) Compacting the soil by ramming.

(*c*) Ramming in sand, gravel, moorum, broken stone or brick bats in situ between the foundation concrete and soil. This is useful for silt or black cotton, soils and also clayey soils.

(*d*) Removing the poor soil and filling the gap with sand, rubble stone, gravel or other hard material. This will increase the bearing power to about twice its original value. In this method the foundation trenches are excavated to a depth of about 1.5 m, and 45 cm wider, and filled with the hard material to a thickness of about 30 to 45 cm and heavily rammed with water so as to force the hard material into the soft soil. If the filled material is buried completely, then another layer of the hard material may be filled in to a depth of about 15 to 23 cm and well rammed. This method is especially useful for black cotton soils. (See under "Foundations in Black-cotton Soils" in the following pages).

Cement grouting the rammed materials will make the foundations much harder.

(*e*) Draining out water from wet foundations. (This method has been explained in detail in the Section "Roads and Highways.")

(*f*) Driving piles, either of wood or concrete, or driving and withdrawing piles and filling the holes with sand or concrete. This will increase the density of the soil. (The method has been explained in detail in the following pages.)

(*g*) **Artificial Stabilization** can be used to seal off permeable strata for deep excavations, or to give soft soils additional strength if they are likely to flow.

Cement grout. Water bearing gravel and coarse sand can be made very much less permeable by pumping cement grout into them. The process is successful only on coarse sands and gravels where the grout can fill up the voids; finer sands necessitate some form of chemical or bituminous emulsion treatment. Grouting is of much use for deep excavations, such as tunnels.

Making Foundations on Weak Soils

(*a*) **Grillage** footings consist of single or double tiers of steel beams or rails. The top tier is laid at right angles to the bottom tier. The beams are held in position by spacers placed between them 1.22 to 1.52 m apart. The stanchion is usually bolted to the top tier and the entire footing is filled solidly with concrete and encased in concrete with a minimum cover of 75 to 100 mm ; a layer of concrete 15 to 20 cm in thickness is placed under the lower beams. The maximum spacing of beams should not be more than 45 cm centre to centre. Overhang ends are designed as cantilevers subject to an upword uniform load equal to the pressure on the foundation. The working stresses of the encased beams may be increased by 33.3 per cent. It is necessary with grillage beams to check the strength of the web for resistance to buckling, and also shear strength for short spans. This type of foundation is generally suitable for single column loads. Steel grillage footings have been largely replaced by reinforced concrete footings known as "Mat Foundations."

(*b*) **Column footings**. For light loads the column footings may be of plain concrete, but most column footings are reinforced concrete footings with two-way reinforcing. Small-diameter closely spaced bars with hooked ends should be used to provide greater bond strength. (This has been fully explained under R.C. Columns.)

(*c*) **Raft or Mat Foundations.** These usually consist of either : (*a*) thick reinforced concrete slabs covering the entire area occupied by the building and reinforced with layers of bars running at right angles to each other about 15 cm below the top surface of the mat, and another layer about 15 cm above the bottom, or (*b*) inverted T-beams of reinforced concrete, with the slab covering the entire foundation area. The beams run under both directions and intersect under columns and support wall loads, if any. Slab and beams are formed into a monolithic structure and act as a unit. Reinforcement is provided in the beams to support walls, if necessary. The basement floor is placed over the beams. Before the basement floor is placed, the space between these beams may be filled with cinders or some other such material. This kind of foundations are used on soft natural ground or fill where the power of the soil is very low and where piles cannot be used advantageously. A raft should be so shaped and proportioned that the centre of area of the ground-bearing should, where practicable, be vertically under the centre of gravity of its imposed load.

Raft foundations are stable so long as the underground conditions are undisturbed. Rise and fall of the underground water-table is dangerous for such type of foundations. Where ground water pressure is likely to occur relief holes should be left in the mat to relieve the water pressure.

Foundations of the above type are sometimes called *floating foundations* and the term is applied where the earth is excavated to a depth that will make the weight of the earth removed about equal to the building load. The total vertical pressure on the soil under the building is about the same after the building is completed as it was before the site was excavated and the settlement is reduced to a minimum.

(*d*) Inverted arches method is now getting obsolete.

(*c*) Piles have been described in detail in the following pages under "Piles and Pile Driving."

A structure should be erected in such a manner that its whole weight is evenly distributed over the solid foundation below to avoid unequal settlement of the sub-soil. All settlement cannot be eliminated because there is a tendency for the central portion of the building to settle more than the outer portion. In order to reduce differential or uneven settlement to a minimum, foundations must be made very rigid. Heavily loaded parts of a building should be separated from the rest, and the higher and heavier parts treated as separate units with independent foundations fitting in such a manner that the whole structure will have equal settlement. The foundations have also to be separated if the soil underneath is of varying nature and different bearing capacities.

The axis of the loads of a unit *i.e.*, the vertical line passing through

the centre of gravity of the weight of the whole unit structure, should coincide with the area of the foundation of the unit. If there is an eccentricity, the intensity of pressure becomes uneven at the two ends producing more compression at one end and less at the other (or even tension and lifting up of the structure) and the structure thus assumes an inclined position resulting in vertical cracks. Eccentric loads are produced by inclined members such as, a pitched roof, thrust from an arch or wind pressure, balconies or brackets. The inclined load is resolved into its vertical and horizontal components and the resultant of all the loads is found, and which should coincide with the centre of the base or should lie within its middle-third for stability. Vertical component divided by maximum permissible vertical load plus horizontal component divided by maximum permissible horizontal load. must not exceed unity.

Sand Piling of Foundations

If the foundation soil is unsatisfactory it can be improved by sand piling. Holes are made into the foundations with wooden pegs 15 cm diameter and 1.2 m long, driven 60 cm into the ground. These holes are filled with sand. The holes are spaced diagonally so that each hole is 60 cm apart from those adjacent to it. Work should proceed from the centre of the trench outwards. Sand piling must never be resorted to in foundations subject to occasional floods by the rise of sub-soil water, or in foundations where water is met within the course of excavation or bottom of driven pegs.

For big structures, holes are made about 30 cm diameter and 3 m deep which are filled with sand. The spacing may be about 2.5 to 3 m according to the arrangements of the columns of the structure. The filled-in sand is thoroughly consolidated and a concrete slabs laid on top of the piles. The concrete is also let into the pile holes for about 15 to 30 cm so as to be monolithic with the slab. A loading test should be taken for the safe bearing capacity of the piles.

On shrinkable clays, it may be more economical to use short bored piles and beam foundations to support the external walls. This has been described under "Piles".

Foundations in Black Cotton Soils. The following methods are generally adopted to meet the characteristics of this soil :—

(*i*) Foundation loads are limited to 5 tonnes/sq. m if water finds access to the foundations, otherwise it may be about 10 tonnes/sq. m.

(*ii*) Foundations are taken down to such depths to which the cracks do not extend.

(*iii*) Trenches are dug on either side of the foundation and filled with sand or other material to prevent intimate contact of the black cotton soil with the concrete and masonry of the foundations.

If the thickness of the black soil is only up to 120 cm it should be completely removed and foundation laid on the soil below.

(*iv*) For important buildings "raft" foundations of reinforced concrete are provided.

For ordinary buildings, the foundations trench should be about 120 cm wide and taken down to at least 15 cm below the depth at which the cracks cease. The bottom of the trench should be well watered and throughly rammed with heavy rammers. On the rammed bed a 30 cm layer of good hard moorum or other soil is spread in 15 cm layers, well watered and rammed. On top of the moorum about 45 cm of sand is spread. Before spreading the sand and in order to keep it from running, when dry, into the cracks in the black cotton soil, a half wall in mud or a thin skin of stone masonry is built along both sides of the trench. On top of this sand the concrete foundation of the building is laid, the masonry to start 15 cm below ground level. Or alternatively, boulder filling may be done underneath the foundation concrete and sides filled with sand. Sand filled around the foundations is about 15 cm for compound walls and unimportant buildings and 45 to 60 cm for main walls. Another method similar to the above is :

Trenches are excavated to a depth of from 150 to 180 cm and width greater than the width of the bottom of footings by 45 cm. Cement concrete is filled in to thickness of 23 cm on both sides of the trench bottom for a width of 23 cm on either side, thus leaving a space equal to the width of the bottom of the masonry and 23 cm high which is filled with sand. On the top of this (for full width of the trench) RC slab is built 15 cm thick. Masonry (foundation footings) is built on the RC slab and the 23 cm space left on both sides of the foundation masonry is filled with sand. A vertical pipe of 75 mm diameter is passed through the plinth masonry to the sand under the RC slab (through the masonry and the slab) which is kept filled with sand. The sand in the tube will filt up the hollows created at the bottom. Such tubes can be built from 120 to 150 cm apart and inspected at every change of season and filled up with sand if required.

Practice now adopted by some departments in cases where black cotton soil is encountered and good foundation is at a greater depth than 120 cm below the surface, is to put in shallow foundations and use two reinforced concrete courses of bands each 10 cm thick, one at the plinth level and the other over doors and windows. Where the second band acts as a lintel it should be adequately reinforced. This prevents cracking of the masonry.

Black cotton soil can be improved by blending it with granular material, or white clay and coarse sand in equal proportions, which is spread on top and rolled.

6. BUILDING ON MADE-UP GROUND OR FILLINGS

The support afforded by made-up ground depends on the composition of filling material, its depth, the manner in which it was placed and the degree of consolidation it has attained. Fills of fine grained materials like very fine sands, silty soils, clays, loosely tipped and not properly compacted during placing, take a very long time for consolidation. Occasional rolling on the top surface makes very little difference, since this can only compact the upper 30 cm or so and leaves the main body of the fill loose. It is always advisable to put test pits down into the fill and by inspection of the sides to estimate the extent to which natural compaction has taken place If the fill is composed of hard granular materials it is likely to give good support. If large voids are found, the consolidation is obviously poor. Most fills will be found inadequate to support any heavy structures. Where bearing capacity of made-up ground is considered to be good, wide strip footings may be sufficient to distribute the load ; they should be well reinforced in both top and bottom. When the fill is loose and of poor supporting value it is usually best to take the foundations through to firm natural ground below by means of piers or piles and to carry the structure on beams spaning between them, well tied together with the piers or piles. Placing a building partly on natural ground and partly on fill should be avoided.

Filling depressions. When placing new fills, water in ponds or depressions should be drained away. The first layer placed in a new fill should preferably be of granular nature so that it may serve as a drainage layer at the base of the fill. If there is only a limited amount of good granular material, it will be best to use the granular material in layers interposed between layers of poor fill. With clay fills, the top layer also should consists of good granular material. Fills should be rolled in thin layers of 23 cm to 30 cm.

7. PUMPING WATER OUT OF EXCAVATIONS

In order to estimate the number of pumps required to drain an excavation or to carry out a ground-water lowering job, it is necessary to estimate the yield of water which depends upon the permeability of the soil and the hydraulic gradient. Whilst no guide can be given as to the number of sumps necessary in an excavation of given area, it is generally considered that, unless the area is very small, two sumps are better than one. Pumping from a number of well-points spread over an area is preferable to heavy pumping from one central pump. It is preferable, where the site permits, to locate the sumps outside the main excavation area and water led to them.

Many types of pumps have been developed. The type of pumps in most common use are :

(1) Reciprocating, (2) Diaphragm, (3) Centrifugal, (4) Pulsometer, and (5) Plunger pumps.

Reciprocating pumps give good results only with water free from grit.

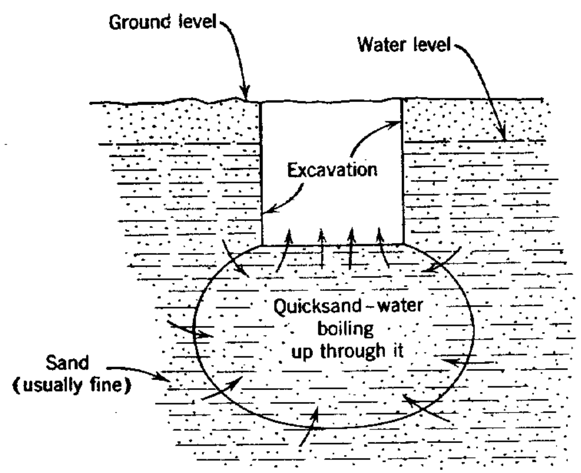
Diaphragm pumps will withstand rough usage and handle dirty and gritty water, though their power and capacity are limited. They are convenient on small sites.

Centrifugal pumps, which are lift pumps, are made with either horizontal or vertical spindles, the latter being especially suitable for sinking. They are simple, easy to drive and maintain, and will cope with variety of conditions, including multi-stage work for high delivery heads. With some loss in efficiency they can be made with wide passages to handle dirty water. For small lifts with large quantities of water, a centrifugal pump is likely to give the best results. Bottom of the suction pipe should be at least 60 cm from the bottom of the sump hole, since any gravel drawn up the suction pipe will injure the pump. The suction pipe should be as short as possible with few bends and should be laid with a slight slope from the pump to the sump. A screen or strainer should be fitted at the lower end of the suction pipe. A foot-valve is required above the strainer.

The pulsometer pump, which is a force pump, is very convenient for foundation work as it requires no staging and can be slung in any position, by a flexible pipe. Its range of pumping is practically unlimited.

Double acting plunger pumps are excellent for sandy, gritty and dirty waters. They are mostly used horizontally. (Pumps have been described in detail in the Section on "Water Supply".)

Heavy pumping in excavations should be avoided as is may tend to remove the fine material from under the foundations or ground adjoining and cause "blowing" or "quicksand formation". Special care is necessary where there is a sand layer within a cohesive soil, especially if such a layer is water-bearing.



Quicksand

Quicksand Formation. Quicksand is a condition and not a soil type. This condition is created in saturated thick layers of loose fine sandy soils when disturbed either due to vibration such as, from pile driving in the neighbourhood, or due to pressure of flowing water. The particles in trying to achieve a closer packing will force the pore water upwards and out at the surface, and if this has sufficient velocity to cause a floatation or "boiling up" of the particles, the sand particles begin to move horizontally and get lifted up, the bottom sand rising up and its space is

occupied by the adjoining particles, thus making a regular movement. The finer the sand the more readily it is affected by a current of water, especially if it contains a little clay. A particular form of this known as "piping" is met with in coffer dam failures. Under such conditions the material may be carried off from under a structure, which can result in the settlement of buildings at a considerable distance. Even if a full flow is not created, the stability of the soil is lessened due to the upward seepage pressure. The condition can be corrected by lowering head of water by underground drainage.

If there is any chance of excavation or pumping on adjoining sites causing a "loss of ground" beneath the structure by releasing a layer of running sand, this layer should be effectively confined by sheet piling.

Running Sand. Sand below the natural ground water level, which is carried into the trenches, trial pits or boreholes by the flow of ground-water as excavation proceeds.

8. MACHINE FOUNDATIONS

There is no standard practice for fastening and bedding machine bases to their concrete foundotions. The foundation block must be made of sufficient bulk that will stand all the weights, thrusts and vibrations, and should generally be heavier than the machine itself. The mass of the concrete block is usually about from 600 to 900 kg per B.H.P. and the depth should be at least 5 to 6 times the diameter of the cylinder bore for large and medium size engines.

The anchor bolts should be strong enough to resist sliding due to horizontal thrusts. The size of anchor bolts and dimensions of the foundation concrete block are usually supplied by the machine manufactures. Many types of anchor bolts are used which are either removeable or fixed. Removable bolts are usually provided for large machines and are fastened at the lower ends by an anchor plate nut. To give access to the lower ends of bolts, pockets or band-holes are cast in the sides of the concrete foundations. The fixed bolts are cast in the fresh concrete. A straight headless bolt will develop sufficient bond with concrete to equal to tensile strength of the bolt if it is embedded in the concrete to a depth of about 45 times the diameter of the bolt. Small bolts can be hooked at the lower ends and large bolts may be provided with a nut and washer or plate, to secure further mechanical anchorage.

Wooden templates are used to hold the bolts in correct positions and alignments. Where it is not possible to fix the anchor bolts at their exact positions while making the concrete block, holes can be left in for the full depth of the bolts and of section of a frustum of a pyramid, with bigger size at the bottom. These holes are filled in with concrete after the machine has been fixed in position. In placing concrete in the foundation, care should be taken to puddle or vibrate the concrete around the bolts without disturbing their position. When bolts are to be placed in hardened concrete it is necessary to drill holes larger than the bolt, then fasten the bolt in the hole. Holes can be made larger at the bottom by tilting the drill. Cement grout is used to fill the annular space around the anchor bolts. A minimum clearance of 6 mm around the bolt is desirable for grouting although bigger clearances are better. Bolts should be moved up and down a few times to free the grout of air and obtain consolidation. Bored holes in bed plates are usually 3 mm larger than the bolt diameter for bolts up to 22 mm, 6 mm larger for 25 mm to 63 mm diameter bolts, and 10 mm for larger diameter bolts.

Vibrations. Foundations should be insulated from vibrating machinery as heavy vibrating machinery may cause settlements on sands and gravels. Vibrations can be minimized by the following methods :

(*i*) By providing felt, rubber, timber, cork or lead sheets between the bed plate and the foundation block.

When rubber is used it should be of the highest grade, and space should be provided for elastic flow at the sides. Rubber should not be used where the temperature rise is more than 49 deg. C. and it should also be not allowed to come in contact with oil. Rubber also helps damping to a certain extent. Wooden sleepers are put crosswise and spiked. Cork is not very reliable.

(*ii*) By providing rubber or lead sheets between the foundation block and the lower soil.

(*iii*) Sometimes metal springs are fixed between the machine and the bed-plate.

(*iv*) By filling in sand or saw dust between the foundation block and the side soil.

(*v*) The machine should be fixed rigidly with anchor bolts.

(*vi*) Soft soils or loose sands and gravels which are subject to settlement and compaction when vibrated, should be compacted by means of piles or other means.

9. PILES AND PILE DRIVING

Definitions of terms :

Anvil. The part of a power operated hammer which receives the blow of the ram and transmits it to the pile.

Composite pile. A pile whose length is made up of more than one material, *e.g.*, timber at bottom and concrete at top.

Dally. A cushion of hardwood or other material placed on top of the helmet to receive the blows of the hammer.

Driving cap. A temporary cap placed on top of a pile to distribute the blow over the cross-section and to prevent the head being damaged during driving.

Drop or stroke. The distance which the weight is allowed to fall on to the head of the pile.

Drop hammer. A hammer, Ram or Monkey (which are identical terms) raised by a winch and allowed to fall by gravity. A *single-acting hammer* is raised by steam, compressed air, or internal combustion, and allowed to fall by gravity. A *double-acting* is operated by steam, compressed

continued over to page 6/69

air, or internal combustion for lifting the ram and for accelerating the downward stroke. (Also see under "Pile Hammers".)

Helmet. A temporary steel cap placed on top of a reinforced concrete pile to retain the packing in position and to prevent the head from being damaged during driving.

Pile bent. A number of piles projecting above the ground up to the bottom of bridge girders. The piles are connected by capping beams on which the bridge decking rests. (Also see under "Trestle bent".)

Ram. The rising and falling part of the hammer which delivers the blow.

Set. Is the penetration of the pile per blow during the final stages of driving.

A piled foundation is normally provided where the soil material under the base of a structure has insufficient bearing power to take the load of the structure, and the soil near the ground surface is also incapable of supporting a mat foundation. Piles are used for erecting foundations of structures in order to transmit load to the underlying soil and to enhance the load bearing capacity of weak soils.

Types of Piles. Several types of piles are used : (*a*) End Bearing Piles ; (*b*) Friction Piles ; (*c*) Compaction Piles ; (*d*) Batter or Raking Piles ; (*e*) Anchor Piles ; (*f*) Sheet Piles ; (*g*) Screw Piles ; etc. The load taken by each pile depends upon the soil characteristics and the natural setting of the soil layers.

End Bearing Piles are driven through soft strata and go deep to rest on hard surface and support the load by the resistance developed at their points by end-bearing, and act as long columns. In these piles the cross section should be comparatively greater to resist the buckling effect.

Grooved piles made of timber, RC and steel in cross-sections of various shapes are used for making coffer-dams to protect excavations and trenches against soil and surface water and landslides to build sustaining or retaining walls.

Strength of a pile as a Column. The effective length of a pile is considered to be from one third to two thirds of the length in the ground plus the length projecting above ground. The proportion taken depends upon the firmness of the surrounding strata. In no case should the effective length be less than the projecting length plus 1.5 m.

Friction Piles or Floating Piles are driven into hard strata and their bearing capacity is provided chiefly by friction of their side surface against the soil (skin friction).

The following is considered the frictional resistance offered by various soil materials to the pile surface :—

Sand and gravel	...	4.9 to 8.8	tonnes/sq. m
Stiff clay	...	3.9 to 5.9	,, ,,
Clay and sand mixed	...	1.9 to 3.9	,, ,,
Dried and compact silt	...	1.0 to 1.5	,, ,,
Silt and soft clay	...	0.25 to 0.50	,, ,,

Compaction Piles are used to compact loose granular soils to increase their bearing capacity.

Influence of Type of Soil. The load carrying capacity of piles is affected by the structure, water content, frictional and cohesive properties of a soil.

Soils with upper soft and plastic layers with stiff layers below: If the soft upper layers cannot be consolidated by driving piles into them, the piles must be taken down to the stiffer layers below. But the piles should not, however, be driven through the firm layer unless they can reach another more suitable firm stratum lower down. Where the upper layers are compressed to some extent, it is not then necessary to go down deep into the stiffer lower layers. The procedure in such soils is to start driving piles at widely spaced intervals and then to drive intermediate ones until the bearing capacity is increased to the desired degree. In some cases clays are weakened by driving piles through them.

Load Carrying Capacity of Piles. It is very essential to explore the foundation strata before deciding for pile driving and the safe loads the piles will carry. Resistance of a particular soil to pile driving is not always the correct indication of its load carrying capacity, When ground water levels fluctuate, there may be considerable variation in the soil resistance especially in the case of permeable soils (sand, gravel) where water from the adjacent soils may lubricate the sides of the pile. Piles driven into loose fine sands or silts may sustain a much larger steady load than that indicated by the final set per blow. Fine sands with some water will show permature refusal to driving, sand and water both being incompressible, but after a little rest the piles can be driven further when the materials have adjusted themselves. On the other hand, in the case of clays, a pile will show a higher loading capacity immediately after it has been driven, which will be lowered after a few days when the clay particles have been adjusted and set. In some cases clays are weakened by driving piles through them. Piles driven into clay if left overnight, will set up and be difficult to start.

Special care and investigation is necessary for piles driven through soft sensitive clays, as appreciable settlements may occur with driven piles embedded wholly in clays. In such cases bored piles may be perferable to driven piles. Occasionally, piles which give a large set under the hammer will be found to acquire much greater resistance after a few days' rest. Therefore, full load tests should be carried out for all doubtful cases after the piles have been finally driven. The longest practicable time should be allowed to elapse between driving and testing to allow the recovery of soil conditions around the pile. A factor of safety of 2 to 3 is generally allowed. Driving to "refusal" or driving so that an exceedingly small set is obtained should be avoided.

In the case of concrete piles cast-in-situ, tests should be carried out by actual loading after 2 weeks of concreting, when it has set.

The load at which a pile begins to show settlement should be taken as the ultimate strength of the pile. In other words, the maximum load which can be carried by a pile is, at which the pile continues to sink without further increase of load. A suitable factor of safety has to be applied to such loads to arrive at the allowable load.

Pile groups. The load carrying capacity of a group of piles is not always a multiple of the capacity of a single pile. For group of piles depending mainly on frictional resistance in cohesive soils, an appreciable reduction in the bearing capacity should be anticipated. In soils with deep deposits of fairly uniform consistency and which are compressed by piles, the computation of pressures should be made on the assumption that the load is spread uniformly at the bottom of the piles for a distance of 0.58 times the length of the pile. (In some of the soils the individual bearing capacity is reduced only to one-third.) As this determines the spread of the load due to the action of the piles, the number of piles required and their spacing under a specific load can be fixed. Piles must have at least two diameters clear space between them in all directions. Test loads should be applied to groups of at least four piles placed at the intended spacing rather than to single piles.

For piles depending mainly on end support in non-cohesive soils, no corresponding reduction in individual bearing capacity need be allowed, while in loose sands and in some silts the bearing capacity of a group of driven piles may be higher due to the effect of compaction. The bearing capacity cannot be accurately forecast except by test loadings on the whole group.

Testing Piles for Loads. Ordinarily one-third of the total piles on an area should be tested, but not less than two piles for the entire site. A suitable platform should be built on top of pile which has been in place for at least 24 hours after it has been finally driven. The total test load should be twice the proposed working load on the pile, (some authorities recommend only 1.5 times) which should be put in about four to six increments starting with half the working load. The next load should be put after about 12 hours when there is no settlement. Allow final load to remain at least 48 hours after there is no settlement and which should not exceed 0.3 mm in 48 hours (total net settlement after deducting rebound). If the settlement is more, reduce the load.

Determination of Ultimate Bearing Capacity. The bearing capacity is most accurately determined from test loading. The probable bearing capacity in noncohesive soils (gravels, coarse sands and similar deposits) may be deduced from one of the dynamic pile formulae. Many of these formulae are very unreliable and should be used with caution. The formulae are not applicable to systems which provide an enlarged base to the foot of the pile.

Formulae for Determining Safe Load on Piles :

$$P=\frac{16.7Wh}{S+2.54}$$ ......for piles driven with freely falling drop hammer.

$$P=\frac{16.7\,Wh}{S+0.254}$$ ......ditto. with single-acting steam hammer.

$$P=\frac{16.7\,h\,(W+ap)}{S+0.254}$$ ......ditto. with double-acting steam hammer.

P=safe load on pile in kg,
W=weight of monkey in kg,
p=mean effective steam pressure in kg/sq. cm,
a=effective piston area in sq. cm,
h=height of fall in metres,
S=average penetration of the pile in cm per blow measured as the average of the last of 5 to 10 blows under a drop hammer or as the average of the last 20 blows under a steam hammer.

Formula for timber piles :

(*a*) Piles driven with steam hammer $P=\dfrac{16WH}{S+0.25}$

(*b*) Piles driven with drop hammer $P=\dfrac{16WH}{S+2.5}$

H=free fall of monkey in metres, and
S=penetration of pile in cm to be taken as the average of three blows.

Safe loads on isolated single piles or isolated pairs of piles should be reduced to allow for accidental misplacement during driving or inaccurate positioning.

Safe Loads on Piles in Tonnes

Penetration of piles in mm	150-kg monkey			300-kg monkey			1-tonne monkey		
	Height of fall of monkey in metres								
	1.22	1.83	2.44	1.22	1.83	2.44	1.22	1.83	2.44
6	0.96	1.44	1.92	1.92	2.88	3.84	6.40	9.60	12.80
13	0.80	1.20	1.60	1.60	2.40	3.20	5.33	8.66	10.67
19	0.69	1.03	1.37	1.37	2.06	3.74	4.57	6.86	9.10
25	0.60	0.90	1.20	1.20	1.80	2.40	4.00	6.00	8.00

Pile Hammers

Pile hammers are of three main categories : (i) Drop hammers ; (ii) Single-acting hammers ; and (iii) Double-acting hammers. Drop hammers may be used for driving all kinds of piles but are normally used for driving light and steel sheet piling. They are usually of cast iron with a lifting eye and require a leader guide.

Single-acting hammers are used for driving heavy piles in compact or hard soils. The hammers are usually 2 to 4 tonnes in weight and have a stroke up to 1.5 metres. The mass of the ram of a single-acting hammer (including diesel hammers) should be : for piles longer than 12 m—not less than the mass of the pile ; for piles up to 12 m long—when driving piles in cohesive soils, not less than 1.5, and in soils of average cohesiveness, not less than 1.25 times the mass of the piles, including the pile cap.

A double-acting hammer operates with rapid blows in succession which for the small size may be as many as 300 per minute if the hammer weighs less than one tonne. Double-acting hydraulic hammers are simple to operate less noisy compared to diesel hammers and much more economical in operation than steam and pneumatic hammers. These hammers are generally used to drive piles of light or moderate weight in soils of any cohesiveness (soils of average resistance against driving).

Diesel hammer is a small, light weight self-contained and self-activating type, worked on petrol. For vibrating hammers, the driving unit vibrates at high frequency.

It is desirable that the weight of the hammer should be at least half that of the pile. With pre-cast concrete piles, the weight of the hammer should not be less than 30 times the weight of 30 cm of pile. With a single-acting or drop hammer, the stroke should be limited to 1.4 m or less for reinforced concrete piles. The weight of the hammer should also be sufficient to ensure a final penetrations of not less than 0.3 mm per blow. It is always preferable to employ the heaviest practicable hammer and to limit the drop or stroke, so as not to damage the pile.

The weight of a hammer for driving short concrete or wooden piles is about 250 to 500 kg and for driving big and heavy piles it may be about 2 to 3 tonnes, which gives about 80 blows per minute through a height of about one metre. When there is any uncertainty about the proper weight of a hammer it is advisable to use a heavier rather than a lighter hammer.

A comparatively heavy ram with a shorter fall is found practically to be better than a light ram with a great fall, the latter having the tendency to shiver the pile instead of forcing it down. A heavy ram with small fall is best for sand ; a light ram with high fall for clay. A great number of light blows are preferable to a small number of heavy blows especially in sand.

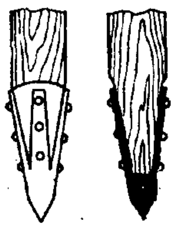
Timber Piles

Timber piles are extensively used as they have the advantage of flexibility and lightness, and in many places they are cheaper than other materials. Their disadvantage is lack of durability in certain conditions. Durability depends on the type of wood, its moisture content, and its position. In general, timber piles are durable in permanently-wet or permanently-dry positions, but not where they are alternatively wet and dry or where the moisture content is widely variable. Timber piles if used below ground level last for a very long period, but ordinarity they last not longer than 30 years or thereabouts and as such are usually preferred for temporary works and also for semi permanent marine structures. Timber piles should be impregnated with solignum, creosote or treated with some such anti-rot compounds to enhance their service life. Preservative treatment may not be necessary for piles which will be completely and permanently submerged in water-logged ground ; in this case seasoning is not necessary and piles may be stored in water prior to use.

Indian timbers suitable for piles are : Teak, Sal, Deodar, Babool, Khair, Ippi, Kumbia, Jamba, Rayani.

Timber piles should be sound and free from sharp crooks and bends or decay, and sufficiently straight so that a line drawn from the centre of the head to the point at the bottom will be wholly within the pile.

Timber piles are generally 15 to 45 cm (up to 30 cm common) in diameter or square in section. Round piles are made of tree trunks with bark stipped and square piles are cut from the heartwood of long logs. Length 4 to 12 m and not greater than 20 times the diameter (or width) at the top. They are either made tapering throughout the length or the upper half is kept straight and the lower half is tapered to about 15 cm square size. The bottom is shaped conically for a length of from 1.5 to 2 times the diameter or about 45 cm. To facilitate driving, the bottom ends of piles are pointed and fitted with a steel shoe. The pile head is slightly tapered and set in a metal ring (pile band) which prevents damage to it during driving. The band is made of strip steel 10 to 15 mm thick and 40 to 75 mm wide to protect it from splintering under the blows of a hammer. After driving, the heads of the piles should be cut off square to sound wood and treated with preservative before capping. For lengthening a timber pile, a piece can be added at the top by straps and bolts. For increasing frictional resistance, small battens can be fixed on the sides lengthwise. The bearing capacity of piles can be enhanced by binding them into packages of 2 to 4 pieces by coupling bolts.



Usual spacing of timber piles is 70 to 80 cm. They should never be driven to "refusal". Piles are considered to be sufficiently driven when five blows fail to drive more than 12 mm or when the last blow does not sink the head more than 6 mm. Timber piles forming the foundation of a building should be cut off below the lowest ground-water level. If concrete cap is provided, the piles should be embedded for a depth sufficient to ensure transmission of load. The concrete should be at least 15 cm outside the piles and be suitably reinforced to prevent splitting.

Timber grooved piles are manufactured of boards 4 to 10 cm or beams up to 28 cm thick. The shapes of the tongue and groove may be triangular or trapezoidal.

Testing Existing Timber Piles (in water) for Deterioration—such as under Bridges :—

If a small bore is made with a carpenter's auger (about 20 mm diameter) at the ground line where decay first sets in, it will disclose rot which is not apparent on the surface. A pointed rod of about the same diameter thrust into the pile will also indicate the position.

Pre-cast Concrete Piles

Reinforced concrete piles are widely used in construction practice—with and without pre-stressed reinforcement. They are usually of square or rectangular cross-section, as they are easier to cast than a round section. The usual size is 15 to 60 cm but piles have been made up to 90 cm size with cylindrical holes inside. Hollow piles have advantages where exceptional lengths are required; they provide stiffness and large perimeter with

lesser weight than solid piles. For piles larger than 36 × 36 cm an octagonal section is preferable to a square section. Square piles should have chamfered corners.

The maximum lengths of piles are usually 12 m for 30 cm square, 15 m for 36 cm square, 18 m for 40 cm square, and 21 m for 46 cm square. It is preferable to keep the lengths less than 40 times the side for friction piles and less than 20 times the side for bearing piles. Where the piles are considered to act as columns, stresses should be calculated as for ordinary columns. To prevent damage to the head of a pile, the top edges should be chamfered liberally and additional lateral reinforcement provided and kept back from the head about 50 to 75 mm according to the diameter.

Concrete piles should be cured for at least one month. Lifting holes should be made at one-fourth to one-fifth the length of the pile from each end and a toggle bolt hole one metre from the head at right angles to the lifting hole. 2.5 cm diameter gas pipe ferrules may be fixed in the holes.

Reinforcement for Concrete Piles. The area of the main longitudinal reinforcement may be 1.5 per cent of the gross cross-sectional area of the pile for piles of length up to thirty times their least width and 2 per cent for lengths 30 to 40 times, which may be increased to 3 per cent for longer lengths.

One rod is provided at each corner in square piles, and one rod at each angle of octagonal or hexagonal piles. All main longitudinal bars should be of the same length and level at the top, and should fit tightly into the pile shoe following the taper of the shoe. Joints in longitudinal bars, if unavoidable should be made by butt welding or as explained under "Lengthening of RC piles". Transverse reinforcement should be provided in the form of hooks or links of not less than 5 mm diameter, or one-quarter the diameter of the main bars whichever is greater, and the quantity should not be less than 0.4 per cent of the gross concrete volume, spaced not more than half the least width of the pile. The links usually are 6 mm dia. up to 12 m and 10 mm dia. above 12 m and are spaced 50 to 75 mm for lengths up to 3 times the side at each end of the pile, lengthening to 15 to 20 cm at the centre.

When attack by soil water is expected, the RC piles can be protected by impregnation with bitumen. The cover over all reinforcement, including binding wire should not be less than 40 mm of concrete, but where the piles are exposed to sea water or other corrosive influences, the cover should be nowhere less than 50 mm.

In RC piles with pre-stressed reinforcement, the longitudinal reinforcement is placed not at the corners but in the centre of the cross-section (it obviates the need for transversal strengthening).

Reinforced concrete piles should be of 1 : 1.5 : 3, or richer mix., with well graded aggregate, of maximum size limited to 12 mm, and a slump of about 40 mm.

Point of the Piles. For plastic soils a blunt point is suited ranging from no point at all to a diameter at the tip of 1/4 of the pile diameter and a length equal to 1.5 times of the pile diameter. For sand and gravel or where hard strata are to be penetrated, a long tapered point is desirable; the tip may have a diameter of 1/4 and a length of 3 times the pile diameter. Points should have cast steel shoes where penetrating hard soils.

Lengthening RC Piles. Timber trial piles should be driven at various places over the site to ascertain the exact lengths required for the piles. Should the driven piles require to be lengthened, the concrete at the heads should be hacked off until the longitudinal rods are exposed for a length of at least one metre. Ferrules or sleevs of water-tubing or similar piping should be placed at the heads of the rods, new lengths butted on, and joints stiffened by fish bars at least 1 m to 1.5 m long, and the whole well laced by steel wires. Longitudinal bars are also butt welded. The old surface of concrete must be well cleaned and brushed, the column shuttering erected and the additional length cast and allowed to cure for at least one month before further driving is continued. Where steel rods are required to be cut off, it can be done with an acetylene torch.

It is generally advisable to use a heavy monkey and a low fall for RC piles. With final set of 12 mm for 10 blows, a 2-tonne monkey with a 40 cm fall or a 1.5-tonne monkey with a 0.6 m drop might be used where the load on the piles would not exceed 30 tonnes. For piles designed to carry a load of up to 40 tonnes with the foregoing final set, 1.5-tonne monkey with 0.8 m drop, or a 2-tonne monkey with 0.5 m drop would be suitable.

Concrete piles are generally used for a design load of 70 to 80 tonnes.

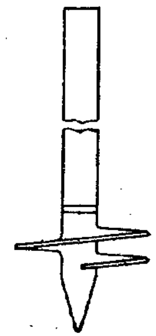
Cast-in-situ piles are made by driving hollow tubes or heavy steel pipe castings and then withdrawing them, or by boring and filling the holes formed with concrete. The tube is placed on top of a loose cast iron point before driving into the ground and is slowly and steadily drawn out of the ground as concrete is filled in. Piles are also formed by driving in a steel shell, leaving it permanently there and filling it with concrete. The shells should be strong enough so that they are not distorted by soil pressure or the driving of adjacent piles. Such piles are built in diameters of 0.4 to 2 m and in lengths of up to 50 m. The bearing capacity may be as high as 600 tonnes per pile. These piles can also be made with bulb toes giving greater bearing value. There are many patented processes for these piles such as, Franki, Simplex, Vibro.

No driving pile should be withdrawn until all piles within 3 m radius have been driven. Built-in-place piles are made without vibrating the soil, which is particularly important for construction work near or inside existing buildings and installations.

As these piles are not intended to carry impact loads under normal condition no reinforcement is necessary and where required it is placed in the top part only. Any reinforcement used should be made up into cages sufficiently well wired; the bars should be openly spaced, and the lateral ties should not be closer than 15 cm centres. The reinforcement should be exposed for a sufficient distance to permit it to be adequately bonded into the pile cap.

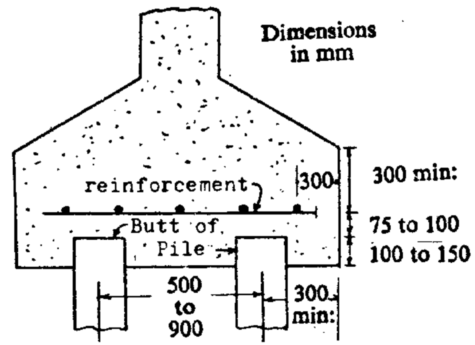
In bored piles care should be taken to prevent the influx of soil into the castings during boring. Before placing the concrete the holes should be inspected by lowering a light for any undesirable materials or water. Placing of concrete should not be started until all the shells in a group have been driven and, in general, until all driving within a radius of 4 5 m has been completed. Bored piles unless sunk into hard and compact ground should be test loaded.

Protection against corrosion. Steel shells which are to be filled with concrete should be coated externally with bituminous composition or tar, etc., before they are driven. In other cases all surfaces should be coated. If tar is employed, it should be neutralized with slaked lime.



Screw Piles. A screw pile consists of a shaft with a steel screw blade attached to the lower end. The diameter of the shaft varies from 7.5 cm to 25.5 cm and the blade is 45 cm to 150 cm diameter. When the blade is 150 cm it is called a *Disc-pile*. Sometimes there are no blades and only the shaft is screwed at the bottom.

These piles can be screwed down to great depths in clay or such soils and also penetrate through small broken stones. The base area of screw does most of the weight bearing. These piles are useful where shocks of driving other types of piles are injurious to the neighbouring structures. They are screwed down by long bars at the top by manpower. Not much used now.



Making Foundation over Piles. (*Capping*). The pile tops should be extended into foundations of the structure for 15 to 23 cm and embedded in concrete, or a space 15 to 30 cm below the top of the piles and 30 cm outside the piles is excavated and concrete placed around the above the piles. Sometimes concrete is stripped off from the top of the piles for a length of about 60 cm and the rods are bent and incorporated with those of the caps or footings to form a monolithic whole. Steel rods are left protruding over the top of a pile to be embedded subsequently in the foundations. Isolated stanchions or piers supported by fully loaded piles should for the sake of stability, preferably be supported by groups of not less than three piles with pile-caps designed to transmit the load to each pile.

Test Piles should be of the same material and dimensions as the working piles and driven with the same type of plant. Whenever possible, test piles should be driven and installed near to the borings so that the driving records can be studied in conjunction with the samples and the boring records.

Test piles should ordinarily be not less than 6 m long and during driving necessary observations should be made to determine the supporting capacity of the piles. From this the number and length of piles for a particular load can be determined. For a foundation covering a large area, it is well to drive test piles at frequent intervals.

Test that piles have been driven to a safe bearing—one of the following conditions to be satisfied :—

	Weight of Monkey	Fall of Monkey	Penetration with last blow (av.)
Either :	400 kg	1.5 m	5 mm in 30 blows
or	760 kg	4.6 m	6 mm in 10 blows
or	400 kg	9.0 m	5 mm in 10 blows

Pile Foundations for Bridges

Pile load is limited to 50 tons for RC piles and 20 tonnes for timber piles.

Spacing of Piles—minimum 1 m.

Reinforcement for RC piles not less than 1 per cent. Driving hammer not less than the weight of pile. Drop of hammer 90 to 120 cm.

Piles must be driven below bed level to a depth not less than twice the maximum depth at Observed High Flood, ordinarily subject to a maximum of 3 m. Their projection above the bed should not be greater than the buried depth.

Driving Piles

When the soil is soft to a great depth the area should be enclosed with sheet piling before the main piles are driven to consolidate the ground better. Piles should not be more than 2% out of plumb and not more than 75 mm out of place.

The top of a pile must be cut square so that the impact of the hammer is distributed uniformly and the pile is driven truly vertical. Where a pile has been driven out of alignment, forcing pile head back into line should not be permitted unless the ground around the pile (in the direction of the pull) has been first excavated.

Pile Driving Methods

Impact and Vibration Pile Driving

Piles are sunk by driving using various types of hammers ; by vibrations using vibrating hammers ; screwing by capstans ; thrusting ; or hydraulic jetting. Hammer driving is employed for sinking wooden, reinforced concrete and steel piles into any soil. Vibration driving is suitable for sand and water-logged soils. Screwing is used to sink piles

with a helical head by rotation into soils free from large solid inclusion. Thrusting and vibration is suitable for sinking short (not more than 6 m) piles in both loose and solid soils. Water jetting is used for driving piles into sand, gravel and other non-cohesive soils amenable to jetting. In some cases, combined methods for sinking piles are employed, such as driving with jetting and vibration thrusting with jetting.

When selecting a pile driving method, account should be taken of the fact that impact methods (driving, vibration thrusting) entail vibrations and jarring of the soil, which, in its turn, may lead to setting and failure of existing structures near the pile driving sites. This can be avoided by using impact-free pile driving techniques (such as, static thrusting, screwing, hydraulic jetting) or built-in-place (filling) piles.

The pile driving should always progress away from an existing structure and not towards it. In driving piles close to old bents, walls or piers, they should be started leaning slightly away to prevent the lateral pressure crowding the points over. In the case of a river, pile driving is done in the direction going towards the river bed.

Cushions for Pile Heads. The top of a pile is damaged by the impacts of the blows, therefore, piles must be protected by cushions (pads) of some resilience material whilst they are being driven, to absorb the shocks. With steam hammers a suitable driving head made of cast iron is provided to fit the top of the pile. A thick packing of felt, bags of sawdust, gunny bags, old rags, ropes or such like material, is placed over the pile head and under a block of hardwood, which is placed on the top of the cast iron driving hood. The cushion should not normally be more than 75 mm thick but should give enough protection to the pile head and should not absorb too much of energy of the blow. Two layers of soft wood boarding have also been found to be satisfactory. Where pile heads are made with the longitudinal bars protruding, the driving head should be designed accordingly ; a steel helmet is made to fit on the top of the pile and sand filled in to form a cushion.

Sinking piles with the help of water jet : The work is sometimes much facilitated by the use of a water jet. It is a means of avoiding very hard driving and vibrations in materials such as sand. A small diameter jet pipe is taken down the side of the pile and the pipe is kept working up and down if required. Water pressure of about 0.35 to 0.50 kg/sq. cm in sand and 2 to 2.8 kg/sq. cm in clay is considered sufficient. A jet tube can also be cast into the pile and connected to the pile shoe which is provided with jet holes. At least two jet holes are necessary on the opposite sides of the shoe. The jetting pipes should have an internal diameter of not less than 50 mm terminating in a nozzle or fishtail of reduced area. Jetting should be stopped before completing the driving, which should always be finished by ordinary methods. The ground should not, however, be very much disturbed.

For sinking in sand, flooding the surface round the pile is quite helpful.

Batter or Raking Piles are driven to resist large horizontal or inclined

forces and eccentric loads, and are generally in addition to vertical piles.
Piles may be driven to a batter as great as 1 in 4. Piles on a larger batter
are difficult to drive and the batter tends to increase. As far as possible,
raking piles should be supported during driving right down to the level at
which they enter reasonably solid ground. Bearing value of raking piles
is reduced by about 1 to 6 per cent.

Extracting Piles. Piles can be extracted either : (*a*) by a direct pull
from a winch in the case of short and easily removable piles ; (*b*) by hy-
draulic jacks acting on a large grip surrounding the pile ; (*c*) by an inverted
double-acting hammer. The piles to be pulled out should be kept lubricated
with water to reduce friction of the soil. Pulling force is calculated from
the frictional resistance of the soil. Safe uplift strength of friction piles in
sand, clay or gravel is generally taken half of the safe bearing load.

Driving Piles Without Engine. Where a pile driving engine is not
available, the following method for driving the piles can be adopted.

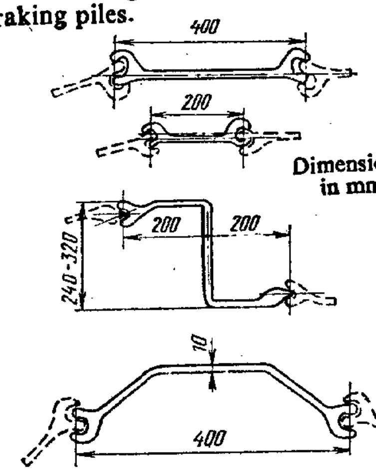
Set an iron rod 50 mm in diameter and 2 to 2.5 m long, about 30 cm
into the centre of the head of each pile truly parallel to the length. Up
and down this guide rod is worked a wooden monkey about 25 cm
diameter and about one m long provided with four handles of 20 mm iron,
screwed by wood screws to the block under 50 mm iron rings shrunk on at
either end. Down the centre of the monkey is a 63 mm hole to allow it to
slide freely up and down the guide rod. The monkey is worked up and
down by men who stand on a platform fixed to the pile.

Another method is to erect a frame-work (similar to a tripodal lifting
tackle) around the place where a pile is to be driven. A pulley is fixed
over the top and the hammer is tied with a rope passing over the pulley
and carried to a hand operated winch. A timber frame-work is erected
round the pile to guide the hammer and the pile and also to keep the pile
vertical.

This is quite a slow process and hardly 15 to 20 blows can be given
in an hour and only one pile may be driven in one day.

Sheet Piling. Sheet piling is used for coffer dams, holding up the
faces of excavations, for quay walls, retaining river banks, etc. For
retaining walls sheet piling acts as a vertical beam loaded by the earth
pressure behind it. At the lower end the piling is supported by the passive
resistance of the earth in front ; at the top it is generally supported by a
waling and the tie rods. For walls of small heights the tie rods may be
omitted ; the wall then acts as a vertical cantilever, the fixing moment being
derived from the earth resistances in front and behind the wall. Earth
pressure has been explained under "Retaining Walls". In most cases,
however, tie rods and anchorages are essential (to provide anchorage
against horizontal pull) and the piling then spans as a beam between the
level of the ties and a point some distance below the ground level in front
of the wall, depending on the nature of the soil. The tie rods are normally
anchored to mass concrete blocks, groups of steel sheet piles or other
appropriate forms of anchorage located at a suitable distance from the
face of the retaining wall (called anchor piles).

The spacing of the tie rods is generally 3 to 4.5 m to suit an even
number of piles. In order that full resistance be obtained, it is essential to
place the anchorage within completely stable soil. Other types of ancho-
rages can be designed when site conditions preclude the use of tie rods of
normal length. In some cases the most suitable type is in the form of
raking piles.



Dimensions in mm

Sheet piles are either of steel,
concrete or timber. Steel piles are
generally standardized or patented and
are provided with longitudinal inter-
locking joints for water tightness and
are driven with supporting guide piles
at internals. Concrete piles are with
tongue and groove joints. Timber piles
are generally about 3.5 m long, 230 to
300 mm wide and 75 mm thick. They
are made of 3 planks of 25 mm thick-
ness and bolted together to make a
tongue and groove sheetings or cut
from a single plank, and are driven
along guide piles which are driven
first at intervals of 1.5 to 3 m.

The advantages of steel sheet piling over other forms of piling are
that the section of the pile gives greater strength for the same weight and
permits them to be driven and easily extracted. Steel sheet piling is liable
to deteriorate owing to the formation of rust. The average rate of corrosion
per year is considered 0.07 mm in sea-water and 0.05 mm in fresh water.
A protective coating of tar or some similar anti-corrosive paint should be
applied periodically.

Safe Loads in Tonnes for Pre-cast Reinforced Concrete Piles

Size of pile in mm	Max: load in tonnes	Max: length in metres	Dia. in mm of main bars fo[r] length of pile in metres 6	9	12	15	18	2[…]
250×250	20—25	9.1	16	20	—			
300×300	35—40	12.2	20	22	25	—		
350×350	50—55	16.8	20	22	25	28	—	
400×400	65—75	19.8	—	22	25	28	32	—
450×450	80—90	22.9	—	—	25	28	32	3[…]

The least width and cross-sectional area of a taper pile should b[e]
based on the dimensions at a point two-thirds of its exposed length fro[m]
the top of the pile, the exposed length being increased as mentioned abo[ve]
if the top stratum consists of very soft clay or mud.

Piles should not generally be loaded above 15 to 20 tonnes exce[pt]
for bridges.

Safe Load in Tonnes for Timber Piles and Struts

Length in metres	Diameter				
	100 mm	150 mm	230 mm	300 mm	380 mm
2.0 m	5.3	13.8	33.5	61.5	97.7
3.0 m	4.0	11.5	20.3	57.6	93.2
6.0 m	—	7.2	22.0	45.9	79.0
9.0 m		—	16.2	36.2	64.9
12.0 m			—	28.8	53.8
15.0 m				23.6	45.2

Reduction Factors for Piles Acting as Columns

Ratio of l/r	Timber	R.C.	Steel	Cast Iron
0	1.00	—	1.00	1.13
10	0.98	—	0.95	0.94
20	0.95	—	0.89	0.80
30	0.93	—	0.84	0.64
40	0.89	—	0.78	0.50
50	0.82	1.00	0.73	0.39
60	0.72	0.88	0.68	0.31
70	0.61	0.76	0.62	—
80	0.50	0.67	0.57	—
90	0.41	0.59	0.51	—
100	0.34	0.52	0.46	—
110	0.28	—	0.41	—
120	0.24	—	0.36	—
130	0·21	—	0.32	—

l is effective length and r is least radius of gyration. Based on British Standard Code of Practice 4 : 1954.

Choice of pile. The choice of the type of pile is governed largely by site conditions. Under normal conditions a driven pile is usually employed. But where vibrations and noise have to be avoided or where the headroom is limited the use of bored cast-in-place piles is preferable. Either bored or driven cast-in-place piles are likely to derive additional carrying capacity when formed in soils such as coarse sand or gravel owing to the friction developed between the tamped concrete and the surrounding soil.

Spacing of piles depends upon the distribution and magnitude of the loads to be carried, the width of the piles, the soil structure and the manner in which the piles transfer their load to the ground. With end bearing piles, the minimum spacing should not be less than 75 cm centre to centre or twice the least width of the piles, whichever is greater, and one metre spacing should be aimed at. Friction piles should be not less than 110 cm or the perimeter of the piles, whichever is greater. For heavy piles, the maximum spacing varies from 1.5 m to 2.5 m. In the case of screw piles the spacing should not be less than twice the diameter of the screw in soft ground but may be slightly less in ground of good bearing value. There should be no tendency of the side soil rising up due to the driving of adjacent piles which is caused by driving closely-spaced piles into relatively incompressible strata, such as clay or dense sand and gravel. Spacings may be closer in loose sand or fillings.

Short Bored Piles and Beam Foundations

On shrinkable clays it may be more economical to use short bored piles and beam foundations to support the external walls. This system is suitable on sites where firm to hard shrinkable clays occur and where such clays do not overlie softer clays and peat, the system is not suitable for very stony sites. The pile holes are bored to a depth of 2.5 to 3.5 m by an auger. The most suitable hand auger is the bucket type post-hole auger. Average spacing of the piles is about 2.5 m depending upon the locations of doors and windows under which no piles need be bored. Piles should be cast immediately after the hole has been bored and concrete tipped through a hopper so that no soil falls into the hole. Immediately before placing the concrete, the bottom of the hole should be well-punned and also made dry so as to ensure a firm base. The lifts should be about 30 to 60 cm deep and each lift should be thoroughly compacted before the next is poured. A lightly reinforced concrete beam about 300 mm wide and 150 mm deep spans between and is anchored to the piles. The bottom of the beam trenches should preferably be blinded with ashes or clinker (Design of Beams has been explained under "Lintels" in Section 7.) Reinforcing rods about 1.2 m long and 20 mm diameter should be set 60 cm in the head of each corner pile and bent over and cast in the beams.

The load carried by a pile depends on the diameter and length of the pile in addition to the type of clay.

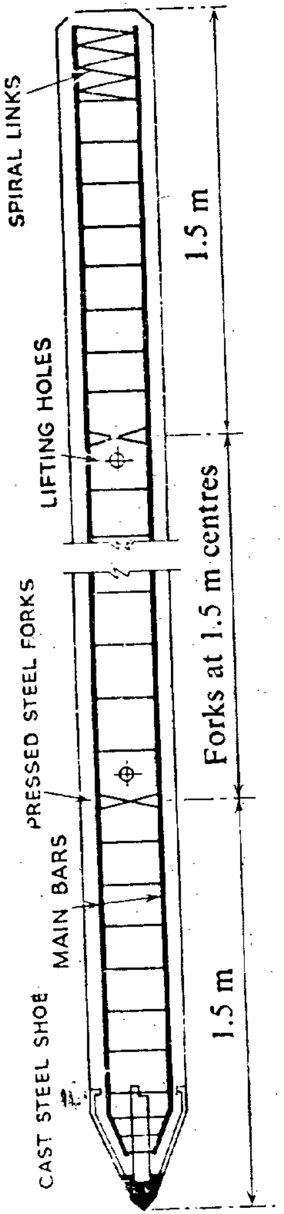
Sufficient bore-holes should be made to determine the nature of the clay and in all cases the depth of the bore should be 60 cm greater than the anticipated length of the pile, with a minimum depth of 3.7 m.

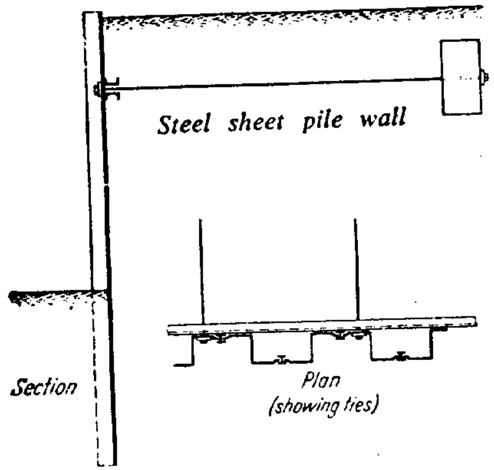
Load Bearing Capacity of Bored Piles in Tonnes

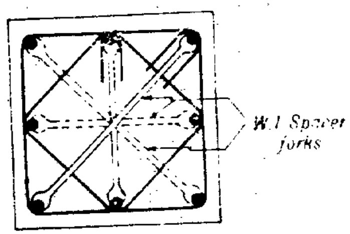
Strength classification	Dia. of pile	Length of pile			
		*1.83 m	2.44 m	3.05 m	3.66 m
Firm at 0.61 m and stiff at 2.44 m	250 mm	2	4	5	3
	300 mm	3	5	6	7
	350 mm	4	6	7	8
Stiff at 0·61 m and hard at 2.44 m	250 mm	4	6	8	
	300 mm	4	7	9	
	350 mm	4	9	11	

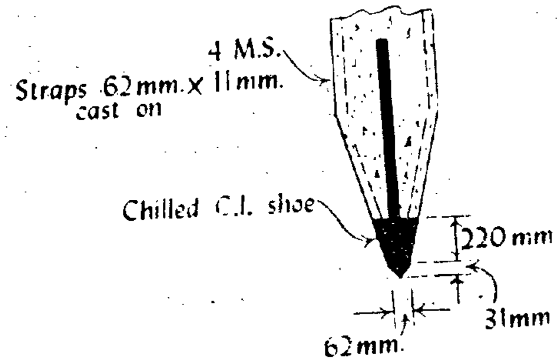
*1.83 m piles are advised only for internal situations given adequate shelter by a solid concrete floor or the oversite concrete.

(Based on Building Research Station, Watford, England, Digest No. 42).







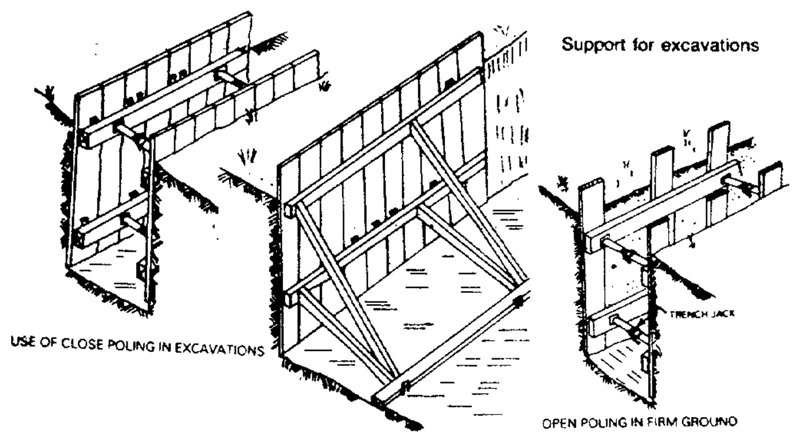


Pile shoes

SECTION 7

GENERAL MASONRY STRUCTURES DESIGN & CONSTRUCTION

Support for excavations



1. PERMISSIBLE LOADS ON MASONRY

Average for good class materials

(Safe permissible loads on Ground or Soil are given in Section 6)

		tonnes/sq. m
Cement concrete	1 : 1 : 2	420
—ditto.—	1 : 1.5 : 3	350
—ditto.—	1 : 2 : 4	310
—ditto.—	1 : 3 : 6	200
—ditto.—	1 : 2.5 : 5	165
—ditto.—	1 : 4 : 8	110
Mass cement concrete	1 : 6	200
—ditto.—	1 : 8	165
—ditto.—	1 : 10	110
—ditto.—	1 : 12	55
Lime concrete		45
Brickwork in cement	1 : 3	100
—ditto.—	1 : 4	90
—ditto.—	1 : 6	55
—ditto.—in lime		45
—ditto.—in mud		27
—ditto.—sundried		10
—ditto.— country, in lime		35
—ditto. ditto. in mud		16
Stone masonry, ashlar, in cement	1 : 3	175
—ditto.— —ditto.—	1 : 6	90
—ditto.— —ditto.—in lime		80
Coursed rubble masonry in cement	1 : 4	110
—ditto.— —ditto.—	1 : 6	55
—ditto.— in lime		50
Random rubble masonry in cement	1 : 4	90
—ditto.— —ditto.—	1 : 6	45
—ditto.— in lime		35
Block masonry in 1 : 3 cement mortar, average crushing strength of block not less than :		
35 kg/sq. cm		27
70 ,, ,,		65
140 ,, ,,		105
Solid cement concrete block masonry in cement 1 : 3		165

The above permissible minimum loads (on walls) may be exceeded up to 20 per cent where such increased pressure is only of a local nature, as at girder bearings, column bases, lintels or other concentration of loads, and should be calculated as uniformly distributed pressures under the contact area. For occasional loads such as wind and earthquakes, the allowable stress may be increased by 33.33 per cent. In eccentric loadings, the compressive stress may be increased by 25 per cent but there should be no tension (as far as practicable) in the masonry.

Permissible stresses in shear and tensile should be taken at not more than one-tenth of the allowable stresses in compression. The following safe working stresses in tension have been recommmended by I.R.C. :—

	Tonnes/sq. m
Ist class dressed stone or cement concrete block masonry in cement mortar 1 : 3	14.0
—ditto.— in lime mortar 1 : 2	7.0
Ist class brick masonry in cement mortar 1 : 3	14.0
—ditto.— in lime mortar 1 : 2	7.0
Lime concrete masonry with stone metal and hydraulic lime mortar.	7.0

2. DAMP-PROOFING AND WATER-PROOFING

Dampness in buildings is generally due to bad design, faulty construction and poor materials used. Structures built on high ground and well drained soil are far less liable to suffer from foundation dampness than those built on low-lying water-logged areas where a sub-soil of clay or peat is commonly found through which dampness will inevitably rise unless properly treated. A sub-soil through which water can easily pass such as firm gravel, sandy soil or a soil containing light clay, will usually keep the foundations fairly dry.

In coastal towns buildings are particularly prone to seepage because of the high humidity and salt particles in the atmosphere. Since salt absorbs water, the walls become damp. When that happens, the plaster peels off, exposing the steel reinforcement. In course of time, the steel is corroded, further weakening the structure. The sand mixed with cement for the reinforced concrete also is said to be salt-contaminated, endangering the life of the structure.

Leaks generally occur from the sanitary fittings. In new constructions, leaks are due to inadequate curing of the concrete, substandard quality of the cement used, and the use of salt-rich sand.

Certain chemicals have been developed for use as water-proofing agents. These effectively plug the leak by expanding to fill the recess in the wall.

Treatment of Foundations on Bad Soils. Where the sub-soil water is not properly drained (in clay or peat soil) the structure should be disconnected from the face of the ground excavation and a trench made all round for a width of about 60 cm taken down to a point at least as low as the underside of the concrete footings. The bed of the trench should be provided with a good slope at each end and the trench filled with coke, gravel, or stone, graded with fines to fill the voids. An open jointed land drain may be laid at the bottom to collect and drain out the sub-soil water. A water-proof coat should be given outside the structure foundations (on the external face of the walls) and continued through the thickness of the walls (under the walls over the foundation concrete) and under the floor. A 75 mm layer of water-proofed cement concrete can be laid all around. Dampness can also sometimes be reduced by leaving out an air gap around the external wall of the foundations.

Where sub-soil drainage has been ignored and necessary precautions have not been taken, water will stand about in the foundations, and the warmth of the interior of the building acting through porous concrete floors will set up suction of moisture which will eventually give rise to dampness in the floors and the walls. Where the sub soil water is near the ground surface and cannot be lowered by underground drainage owing to the flatness of the ground or any other reasons, the level of the floors of the buildings should be kept sufficiently high. It is considered that the height of the plinth should be kept at least 1 8 to 2.4 m minus the difference of level between the ground level and the sub-soil water-table level.

Damp-proof Course. One of the following specifications may be adopted for a damp-proof course, according to the type of the construction and the nature of the ground : –

(*i*) Two courses of dense bricks in 1 : 3 cement mortar. Bricks should have a water absorption of not more than 4.5 per cent. It is advantageous to leave the vertical joints unfilled as moisture rises through the mortar joints.

(*ii*) A layer of well burnt bricks soaked in hot tar and pitch will suit for cheap class buildings.

(*iii*) Non-porous stone slabs about 50 mm thick laid for the full width of the walls over a bed of cement mortar.

(*iv*) Two layers of non-porous slates laid to break joint, each layer being bedded and set solidly in cement mortar 1 : 3.

(*v*) 12 mm cement plaster 1 : 2 with some water-proofing compound laid above the plinth masonry with one or two thick coats of hot coal tar applied over the mortar after the mortar has fully dried. Dry sharp sand should be sprinkled over the hot tar. Five per cent of Pudlo by weight of cement can be used for water-proofing the mortar.

(*vi*) 40 to 50 mm cement concrete 1 : 2 : 4. Two coats of asphalt or hot coal tar should be applied over the cement concrete when the concrete has been fully cured and dried. A coat of 7 asphalt mixed with 3 parts of clean sharp sand may be laid 6 mm thick over the concrete. A layer of tough asphalt about 10 mm thick is often used instead of hot asphalt. Mastic asphalt in one or two layers is generally considered best where hydraulic pressure is encountered. The asphalt used should not melt or soften in the hottest days and should not get squeezed out due to pressure of the masonry over it.

The damp-proof course should be laid flush with the floor surface and should not be carried across doorways or other openings. The upper layer of cement concrete floors should be continued over such openings and should be laid at the same time as the floors. The asphalt or tar layer should be laid under the concrete at the openings. Where concrete is laid on bitumen or tar, the surface of the bitumen or tar must be sprinkled with dry sand.

The position of the damp-proof course is also an important factor and it should be laid at such a height that it is above the normal level to which water splashes from the ground when it is raining. A damp-proof course should not be less than 15 cm above the highest level of the ground. In Northern India plinths are usually kept 45 to 60 cm above ground level for good class buildings under normal conditions.

Treatment of Floors. The floors should be filled with some dry filling. A hard-core filling of stones with smaller stones to fill in voids is quite suitable. The filling should be well rammed but not unduly consolidated. It is considered that a thin layer of cinders and coal tar well rammed under a tiled floor prevents the rise of damp and "kalar" or efflorescence. A filling of 75 to 150 mm of dry coarse sand under the floor masonry is usually specified, but this is suitable for dry locations only. Where there is possibility of moisture penetrating the floor, it will be necessary to lay a liquid-proof membrane before a concrete floor is laid. Porous concrete attracts moisture from wet soil. Even dense cement concrete mixed with water-proofing compound is not a complete barrier to moisture; the passage of water as liquid may be prevented, but moisture can still reach the top of the concrete as vapour and condense there if an impervious finish covers the surface.

Treatment of Walls. Rain can penetrate through solid brick walls as there is a limit to the amount of rain that a wall can keep out, moisture is conveyed from the exterior to the interior due to the porosity of the bricks. More rapid penetration is through the mortar joints, and an efficient pointing on the exterior will greatly resist the passage of water. The simple flush pointing will offer good protection. Sometimes the soffits of all horizontal courses are slightly throated. Cavity walls afford sound protection and ensure a dry interior even if porous material is used for outside. The application of a porous rendering on the external surface will do much to prevent direct penetration.

A porous finish will absorb water in wet weather and will permit free evaporation when the weather improves. A dense impervious rendering is less efficient than a porous one as it will more effectively prevent moisture drying out rather than prevent it getting in, and is also more liable to crack. A porous rendering is less liable to crack and will not cause the entrapment of moisture within the wall. An external treatment unless it is porous will also be liable to aggravate dampness if it is dne to rising ground moisture, indirect penetration of rain or due to deliquescent salts. A mortar of cement : lime : sand in the proportions of 1 : 2 : 9 or 1 : 1 : 6 is usually recommended.

Basements. To ensure dryness the whole of the structure below ground level should be provided with a continuous membrane of asphalt supported on the inside. Other treatments described above under "Foundations" will also be essential.

Condensation. Dampness due to condensation can often be identified by drops of moisture clinging to the whole area of the walls, ceiling and floor which is different from the damp patches that result from rain penetration or rising ground moisture. Condensation is also encouraged by deliquescent salts or saline material present in the mortar which attract moisture from the air. Condensation usually occurs in new structures and is dried up as the rooms are heated. Condensation can be detected by leaving a piece of flat marble in the affected room for about 24 hours when drops of moisture will also be found on the marble piece; this is condensation of moisture suspended in a warm atmosphere on cold non-absorptive surface.

Efflorescence. Where soluble salts are present in excessive quantities in the bricks or the mortar they absorb moisture either from the air or during construction and are brought to the surface in solution and deposited in concentrated patches either as a white powder or as translucent crystals, as the moisture dries out. This crystalline growth either flakes off or is reduced to a powder which can be brushed off. Attempts to seal back efflorescence are not usually successful and it is advisable to allow the efflorescence to expand itself as the wall dries before attempting any treatment at rendering or white-washing the walls.

Soluble salts can be removed by repeated washings with water and brushing the face of the masonry. Salts from small patches can be extracted by trowelling on the surface a layer of slaked lime about 6 mm thick which is made up as a stiff paste. This is left in place until dry, and then removed, and wall brushed down.

Salts from brickwork can also be removed with a solution of zinc sulphate and water. The surface is brushed off when dry. A solution of 1 part hydrochloric or sulphuric acid and 5 parts water is applied vigorously with scrubbing brushes, water being constantly sprayed on the work with a hose to prevent the penetration of the acid. This will remove white or yellow blotches from floors or walls due to efflorescence.

Cleaning of external brick walls after completion of the building can be done with a 5 per cent solution of muriatic acid. The walls must be thoroughly washed with copious flow of clean water both before and after the application of the solution.

The soil used for the preparation of mud mortar, and the water used in the construction should be free from harmful salts. Concrete made of cement, *surkhi*, sand and brick ballast and mortar of cement, *surkhi* and sand for laying bricks is suggested to be used for all foundations in effected areas. The *surkhi* to be made from slightly under-burnt bricks and finely powered, mixed with about 15 to 20 per cent of cement.

A mortar can be made as follows which is waterproof and will also be useful in preventing efflorescence :—

1 part cement : 2 parts sand to which is added 12 kg pulverized alum for each cubic metre of sand. Mix all the three dry and then add the proper quantity of water in which has been dissolved 75 grams soft soap per litre of water, and thoroughly mixed.

Roofs. The presence of moss, vegetation or other growth on roofs is a direct evidence of a porous roofing material in which water will collect and will not be drained off. Overhanging trees will keep the roof wet and their fallen leaves will block the downpipes. Cracked roofing tiles and broken pointing are common causes of leakage. Cement grout poured into the joints and cracks is very helpful. Insufficient lap of tiles or roofing sheets usually cause penetration of rain. Insufficient roof slopes or flat pitches which are too slow to drain off the rain-water quickly are also one of the main causes of leakage. (Water-proofing of Roofs has been described in detail in the Section on "Roofs".)

Rainwater Down Pipes. It is important to provide sufficient number of downpipes and of adequate size as recommended in the section on "Roofs", and it is more important to see quite often that they are not choked up. All vertical pipes should be fixed to stand well clear of the walls so that if any cracks develop in the pipes or there is leakage in the joints, the walls will suffer little damage. Tops of the downpipes should be very carefully and properly fixed with the roof outlets and which should be of sufficient size so that there is no overflowing of the rain water or leakage through the walls. The bottom end (shoe) of the pipes should be so arranged that the water is not thrown back on the walls.

Chimney Stacks. Defective or poorly executed junctions of chimney stacks and roofs are a very common cause of leakage in sloping roofs. A sufficient "tuck" of lead flashings into the chimney brickwork should be provided with cement fillets where necessary. A damp-proof course should be provided across a chimney stack at an eaves and this will check downward penetration of the dampness through the stack.

Copings to Parapets. The top of every wall not protected from the weather by a roof or over-hanging eaves should be so built as to prevent the penetration of rain through the wall. Drop courses should be provided as explained under "Lintels and Sills". The top can be finished with one course of hard, well-burnt bricks set on edge in cement mortar over two courses of slates or dense tiles projecting over the wall. (See under 'Roofs')

Lintels and Sills. All soffits or undersides of lintels and sills should be throated. The mere drafting of a line does not constitute a throating ; there should be a deep and wide chase cut in the soffit which should be returned at the ends of the sill. The top of a window sill should be sloped outwards and weather-bar or water-bar (of metal) should be fitted between the stone sill and the wood sill (or window frame) which will stop the passage of water passing between the sill and the wood frame.

Windows. Shrinkage of unseasoned wood and importance of properly designed window frames should not be ignored. Frames should be so rebated, and which should be deep enough, as to exclude the weather and afford good protection. Double rebated frames are better in severe weather

conditions. Windows opening outside are preferable. A "hood" of simple form with groove to serve as throating can be fixed on the head of the window frame. Where the windows open inside, the inner sill should be made to slope outwards and a small hole kept in the centre passing under the window frame through which any water that has penetrated inside the window can flow out.

In districts liable to heavy storms it would be advisable to provide *hoods* over all window and door openings instead of simple sun-shades.

Causes, Prevention and Remedial Measures of Dampness in Old Structures

Walls. Before applying any remedial measures to a damp wall a very important fact should be borne in mind that there should be a free escape for any water that has already entered the wall. A water-proofing treatment can be applied externally or internally. There are many water-proofing commercial products in the market such as cement paints, bitumen and tar paints, oils. Silica solution is transparent and very effective in resisting dampness. Internal treatment of affected walls would consist of removing the old plaster, applying a slurry coat of neat cement with a water-proofing compound and then cement rendering with a dense mortar of 1 : 2 with an integral water-proofer added. Another internal treatment for damp walls is the application of an impervious coating of some material or a coating of bitumen or tar followed by blinding with sand and plastering. If the body of the wall and any external covering is in porous material the internal treatment will be effective. Where evaporation from the outer surface is likely to be difficult, with internal treatment the wall still remains wet and dampness may spread to the other parts or rise to a greater height as more water is absorbed by the wall, and little benefit can be expected from internal treatment.

The following methods are also used for preventing dampness in walls :—

(*a*) Two parts by weight of coal tar and one part by weight of pitch are put in a vessel and heated and stirred until the mixture is sufficiently liquid, and which is then applied on walls. This has been found to keep out damp very well. (Bombay P.W.D. Specifications).

(*b*) The damp plaster may be varnished over with a solution of 120 grams shellac dissolved in 1 litre of nephtha. This almost immediately hardens. It is preferable to remove the damp plaster and let the walls dry.

(*c*) Spray or paint the walls with a solution of sodium silicate (water glass), followed by a solution of calcium chloride, which forms an insoluble silicate.

Another way of preventing internal dampness is by lining the walls with wooden boards or lathing which are battened out of direct contact with the walls.

If dampness is confined to one position near ground floor level above the damp-proof course, it may be due to a hole or crack in the damp-proof course through which moisture can pass into the wall above. Dampness below ground level may be due to lack of sub-soil drainage, absent of or poor vertical damp-proof course, or leaking drains.

In the case of floors, remove the top concrete and damp filling for a depth of about 30 cm under the floor and refill it with hard-core or some dry material. A water-proof cement colour or a simple cement wash with some water-proofer added may also prove beneficial.

Cement Paints have been described in detail in Section 12. Cement paints should not in general be applied to non-porous surfaces because adhesion is frequently poor. On suitable backgrounds, cement paints provide a hard matt water-proof surface of high durability. Normal oil paint should not be applied unless the wall is thoroughly dry.

3. DESIGN OF WALLS, PIERS AND COLUMNS

Explanation of Terms :—

Load bearing wall. A wall designed to carry a super-imposed load. The thickness of a load-bearing wall should be sufficient at all points to keep the stresses due to dead, live and other loads, for which the structure is designed, within the prescribed limits.

Column. An isolated vertical load bearing member, one of whose horizontal surface dimensions, whilst not less than the other horizontal surface dimension, is not more than four times as great.

Pier. Piers are usually in the form of thickened sections of a wall bonded into load bearing walls at the sides and extends to full height of the wall placed at intervals along the wall, to take concentrated vertical loads or to stiffen the wall so that it can carry additional load or resist lateral pressure without buckling. The thickness of a pier is the overall thickness including the thickness of the wall.

Pillar. A pillar is a detached masonry support. This can be rectangular, circular or of any such shape. In case of rectangular pillars, the breadth shall not exceed three times the thickness and the thickness itself shall not exceed more than three bricks.

(The terms—Column, Pier and Pillar, are very loosely used.)

Buttress. A member similar to a pier except that it is intended to provide lateral support only. It need not extend to the full height of the wall. (It is in fact a projection of masonry built into the front of the wall to strengthen it for lateral stability against thrust from an arch, roof or wind pressure.)

Lateral support. Means support which will restrict movement in the direction of the thickness of the wall or thickness or width of a pier or column. For design purposes, lateral support is considered as from floors, beams or roofs for the height, and from intersecting walls, piers or buttresses for the length. Concrete slabs bearing on walls are considered as sufficient anchorage for the supporting walls. For small houses the stiffening effects of partitions are such that the special anchoring of floors to walls is unnecessary. *Unrestrained* is without lateral support.

Slenderness Ratio for walls : Is the ratio of the effective height (or the effective length if this be less) to the effective thickness. For columns with lateral support at the top, it is the ratio of the effective height to the horizontal dimension of the column lying in the direction of the lateral support. For columns without lateral support at the top, it is the corresponding effective height divided by the least horizontal surface dimensions.

Reduction Factors for Slenderness Ratio

The permissible compressive stress for load bearing tall columns or walls should be multiplied by the following factors : (British Standard Code of Practice – 111)

Slenderness ratio	Factor	Slenderness ratio	Factor	Slenderness ratio	Factor
1	1.0	8	0.70	16	0.35
2	0.96	10	0.60	18	0.30
4	0.88	12	0.50	21	0.25
6	0.80	14	0.40	24	0.20

Effective Height. Is the height considered for designing thickness of a wall or column, and for the determination of its slenderness ratio. The effective heights are taken as follows :

(*i*) For walls without lateral support at top—1.5 actual height.

(*ii*) For walls with lateral support at top—3/4 storey height.

(*iii*) For columns without lateral support at top—2 actual height.

(*iv*) For columns with lateral support at top—actual height between lateral supports.

Height of Walls. The height of a wall is measured from top of the plinth (base of the wall) to the highest part of the wall (excluding any parapet), in the case of gable to half the height of the gable. "Storey height" is the height between lateral supports, *i.e.*, from the underside of a floor structure to the underside of a floor structure of the storey next above.

Effective length of a wall is the length measured between the centre lines of two adjacent piers, buttresses or intersecting walls. The intersecting or return walls must be of at least two-third thickness of the wall under measurement, and well bonded into it.

The load bearing capacity of a wall is dependent upon the crushing strength of the individual units (bricks, stones or blocks), the grade of the mortar used and the bond, and the slenderness ratio of the wall or column between effective lateral restraints. Crushing strength of brick masonry is only about 1/3rd to 1/5th (or even less) of the crushing strength of a single brick.

In walls there is less possibility of buckling than in an isolated column.

In circular walls resistance to buckling is greatly increased, but where the diameter is more than 20 times the thickness of the wall, it should be taken as a straight wall. For circular walls, see under "Steening of Wells" in Section 15.

Thickness of Walls. Thickness of brick walls should be determined for all but small buildings according to well-established rules on the basis of the strength of the bricks and mortar and the ratio of the thickness to the height and length of the wall, which will effect economy. For small buildings of one or two storeys, the thickness is often decided on the basis of its effective protection from the weather which gives a wall of strength many times greater than that required to carry the loads.

High isolated walls should be tested for wind pressure.

The illustration shows thickness of a load-bearing wall for different storey heights of a dwelling in lime mortar or 1 : 6 cement mortar. (Cement mortar, even 1 : 8, is stronger than lime mortar). Top storey wall should be built in cement if the height exceeds 10.5 m or made 13.5 ins. (1.5 bricks) thick.

For walls built in stone, the same thickness may be taken for ashlar masonry but it should be one-third greater for rubble and one-fifth greater for coursed rubble masonry.

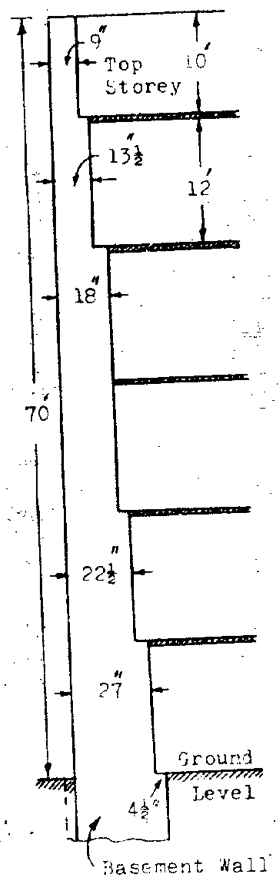
Walls of ware-houses or public buildings should be thicker by 4.5 ins. (1/2 brick) and no wall should be less than 13.5 ins. (1.5 bricks), and work built in cement mortar (at least 1 : 6) except when the height does not exceed 3 storeys when it may be in lime.

Basement and foundation walls should be 4.5 ins. (1/2 brick) thicker than the wall thickness at ground level. Check basement walls for earth pressure.

The thickness of walls determined by the above methods should be checked for the maximum permissible loads on the masonry under the super-imposed loads on the wall.

Too many openings in a wall weaken its stability.

For recesses in walls (for racks or almirahs, etc.) the wall at the back of the recess should be not less than 1 brick thick. The aggregate width of all recesses and openings formed at any one level shall not kxceed two-thirds of the length of the wall at that level.



WALL THICKNESS FOR DWELLINGS

The British Standard Code of Practice—111 specifies that the slenderness ratio should not usually exceed 18 for walls built in cement mortar. For dwellings of not more than two storeys or for reinforced walls, this value may be increased to 24. When a lime mortar is used the ratio should never exceed 12.

Thickness of walls may be taken as follows :—

Walls with roofs, built in			Walls without roofs, built in		
Cement 1 : 3	Lime or cement 1 :6	Mud	Cement 1 : 3	Lime or cement 1 : 6	Mud
H/24	H/18	H/16	H/12	H/9	H/8

H is height of wall.

For dwellings of not more than two storeys, the thickness of walls may be taken H/32 works built in cement mortar 1 : 3.

The above values may be taken for unbraced lengths up to 14 m. For walls of greater lengths, the thickness should be increased by 1/2 brick for cement walls, 3/4 brick for lime walls, and by 1 brick for mud walls. It is, however, more economical to make up the minimum stipulated thickness by the addition of piers or buttresses as explained under "Panelled Walls."

Eccentric Loadings on walls and wall plates. If a wall carries a floor, deflection of the floor tends to concentrate the load on the inner edge of the wall ; this effect will be greater for a relatively flexible flooring system such as the ordinary timber floor than for a stiffer floor of reinforced concrete. Such eccentricity of loading has a very marked effect on the strength of the brickwork and should be avoided as far as possible. To transmit load from a sloping or battened roof, the wall plate should be fixed centrally on the top of the wall. It is not generally desirable that wall plates for supporting floor joists should be built into a wall.

Cross walls. The thickness of cross-walls should be at least two-thirds of the thickness of external walls, subject to a minimum of 9 ins. (1 brick). Where, however, the external wall is not more than 6 m high and 10.5 m in length, the cross-wall may be built 4.5 ins. (1/2 brick) in cement mortar where it does not support any load.

Bonding of Cross Walls and Floors. When the courses of two load-bearing cross walls are built up together, the intersections should be bonded by laying in a true bond at least 50 per cent of the units at the intersections. Where the cross walls are built separately, the perpendicular joint should be regularly toothed and in all important buildings, joints provided with metal anchors having a minimum section of 40 mm × 6 mm with ends bent up at least 50 mm, or with crosspins to form anchorage. Such anchors should be at least 60 mm long and the maximum spacing should be 120 cm. These arrangements of bonding should be carried up to the ground so that the cross walls or piers develop the full strength for lateral support to the main wall.

For walls carrying timber floors in multi-storey flats, metal anchor straps should be used at 1.2 to 1.8 m intervals for tying the wall to the floor to have greater stiffening effect. This is not necessary for small houses.

THICKNESS OF WALLS FOR RESIDENTIAL BUILDINGS

(See note at page 7/22 for "brick thickness")

Height of wall above plinth	Length of wall	Thickness of wall in cement mortar 1 : 3 or 1 : 4	Thickness of wall in lime mortar 1 : 2 or cement mortar 1 : 6	Thickness of wall in mud mortar
Up to 3 m	Any	1 brick for the whole of its height.	1 brick for the whole of its height.	1.5 bricks for the whole of its height.
3 m to 4.5 m	—Ditto.—	—Ditto.—	1.5 bricks for the bottom 2.5 m and 1 brick for the remaining height.	—Ditto.—
4.5 m to 6 m	Up to 9 m	—Ditto.—	1.5 bricks for the bottom 3 m and 1 brick for the remaining height.	2 bricks for the bottom 3 m and 1.5 bricks for the remaining height.
4.5 m to 7.5 m	Up to 9 m	1 brick for the whole of its height if double storey, 1.5 bricks for the bottom 3 m and 1 brick for the remaining height if single storey.	1.5 bricks for the height of the 1st storey and 1 brick for the remaining height; 2 bricks for bottom 3.5 m and 1.5 bricks for the remaining height if single storey.	2 bricks for the height of the 1st storey and 1.5 bricks for the remaining height.
	Above 9 m	1.5 bricks for the height of the Ist storey and 1 brick for the remaining height; ditto.	1.5 bricks for the full height if double storey; 2 bricks for a height of 3.5 m and 1.5 bricks for the remaining height if single storey.	—Ditto.—
7.5 m to 9 m	Up to 7.5 m	1 brick for the whole of its height if double storey; 1.5 bricks at base if single storey.	1.5 bricks for the height of 1st and 2nd storeys and 1 brick for the remaining height.	2 bricks for the height of 1st and 2nd storeys and 1.5 bricks for the remaining height.
	7.5 m to 10.5 m	1.5 bricks for the height of Ist storey and 1 brick for the remaining height.	2 bricks for the height of 1st storey and 1.5 bricks for the remaining height.	—Ditto.—
	Above 10.5 m	1.5 bricks for the height of 1st and 2nd storeys and 1 brick for the remaining height.	—Ditto.—	—Ditto.—
9 m to 12 m	Up to 10.5 m	—Ditto.—	2 bricks for the height of 1st storey, 1.5 bricks for the height of 2nd and 3rd storeys and 1 brick for the remaining height.	2.5 bricks for the height of 1st storey, 2 bricks for the height of 2nd storey and 1.5 bricks for the remaining height.
	Above 10.5 m	2 bricks for the height of 1st storey, 1.5 bricks for the height of 2nd and 3rd storeys and 1 brick for the remaining height.	2.5 brick for the height of 1st storey, 2 bricks for the height of 2nd storey and 1.5 bricks for the remaining height.	—Ditto.—
12 m to 15 m	Up to 7.5 m	2 bricks for the height of the 1st storey, 1.5 bricks for the height of the 2nd and 3rd storeys and 1 brick for the remaining height.	2 bricks for the height of 1st storey, 1.5 bricks for the height of 2nd and 3rd storeys and 1 brick for the remaining height.	

	7.5 m to 10.5 m	—Ditto.—	2 bricks for the height of 1st and 2nd storeys, 1.5 bricks for the remaining height.
	10.5 m to 14 m	2 bricks for the height of the 1st and 2nd storeys, 1.5 bricks for the rest of its height.	2.5 bricks for the height of 1st storey, 2 bricks for the height of 2nd storey and 1.5 bricks for the remaining height.
	Above 14 m	2.5 bricks for the height of 1st storey, 2 bricks for the height of the 2nd and 3rd storeys and 1.5 bricks for the remaining height.	2.5 bricks for the height of 1st and 2nd storeys, 2 bricks for the height of 3rd storey and 1.5 bricks for the remaining height.
15 m to 18 m	Up to 10.5 m	2 bricks for the height of the 1st and 2nd storeys, 1.5 bricks for the height of the next storeys below top storey and 1 brick for the height of the top storey.	2.5 bricks for the height of the 1st storey, 2 bricks for the height of the 2nd and 3rd storeys and 1.5 bricks for the remaining height.
	Above 10.5 m	2.5 bricks for the height of the 1st storey, 2 bricks for the height of the 2nd and 3rd storeys, and 1.5 bricks for the remaining height.	2.5 bricks for the height of the 1st and 2nd storeys, 2 bricks for the height of the next storeys below top storey, 1.5 bricks for the top storey.
18 m to 21 m	Up to 12 m	—Ditto.—	—Ditto.—
	Above 12 m	—Ditto.—	3 bricks for the height of the 1st storey, 2.5 bricks for the height of the 2nd storey, 2 bricks for the height of the next storeys below top storey, 1.5 bricks for the top storey.

Buildings with five to eight floors are categorised as multi-storeyed and those above eight floors are considered high-rise structures.

Thickness of Walls for Public Buildings, Ware Houses and Industrial Buildings

Height of wall	Length of wall	Thickness of wall at base	
		Cement mortar	Lime mortar
Up to 4.5 m	Any	1.5 bricks	1.5 bricks
4.5 m to 7.5 m	Any	1.5 bricks	2 bricks
7.5 m to 9 m	Up to 14 m	1.5 bricks	2 bricks
	Above 14 m	2 bricks	2.5 bricks
9 m to 12 m	Up to 9 m	1.5 bricks	2 bricks
	9 m to 14 m	2 bricks	2.5 bricks
	14 m to 18 m	2.5 bricks	2.5 bricks
	Above 18 m	2.5 bricks	3 bricks
12 m to 15 m	Up to 9 m	2 bricks	2.5 bricks
	9 m to 18 m	2.5 bricks	3 bricks
	Above 18 m	3 bricks	3.5 bricks
15 m to 18 m	Up to 9 m	2.5 bricks	—
	9 m 14 m	3 bricks	—

(i) Where the construction is proposed to be in mud mortar, the thickness of the wall will be 2 bricks (min:) up to a height of 7.5 m and 2.5 bricks up to a height of 12 m.

(ii) Additional thickness of 1/2 bricks at the base will usually be sufficient where vibrating machinery is used.

(iii) The base thickness is reduced towards the top ; for 5 m from the top of the wall, the thickness should be 1.5 bricks but where the wall does not exceed 9 m in height, the wall for 3.5 m from the top may be 1 brick thick if in cement.

Cavity Walls, Two-skin Walls or Hollow Walls. A cavity wall is a wall built of bricks or blocks so arranged as to provide an air space within the wall the two leaves being tied together at intervals by metal or other ties. Such a wall can be made (*i*) of hollow blocks and laid as ordinary brickwork ; (*ii*) with common bricks laid on edge as stretchers and headers in each course breaking joint The cavaties are not kept continuous. The most common method is of making two leaves of 4.5 ins. (1/2 brick) thickness each with a cavity or space. If the two leaves are of different thickness, the thicker leaf shall be made on the inside. The width of the cavity shall not be less than 50 mm and not more than 80 mm (or 3 ins.) with 150 mm maximum. Each leaf of a cavity wall shall be not less than 75 mm thick.

The two leaves are securely tied together by galvanized iron, mild steel or wrought iron wall ties made of flats 10 mm × 5 mm cranked or twisted at their mid-point with ends split and fish-tailed, or with 5 mm dia. wires, spaced not more than four brick lengths apart horizontally and five brick heights vertically and staggered. There shall, at least, be five ties per sq. metre surface area of the wall. Additional ties shall be used near openings. The ties shall be sloped towards the exterior side to prevent water from flowing along it from outer to inner leaf. Ties shall be given a bituminous coating before insertion to protect them from corrosion. The ties are built into horizontal bed joints during erection.

Cavity walls shall not be built more than 7.5 metres in height and 9 metres in length. Where longer lengths or heights are desired, the walls shall be divided into panels.

A 1/2 brick + 1/2 brick with 50 mm cavity wall is better than a one brick solid wall in heat and sound insulation and in its resistance to rain penetration, but from structural point of view the construction is not so sound as a solid wall and is much more expensive too. Walls may be filled in with light weight or porous concrete instead of keeping a cavity. For the purpose of calculating "slenderness ratio" of a cavity wall, the effective thickness is taken equal to 2/3 of the aggregate thickness of both leaves. Roof loads should be distributed on both the leaves.

Honeycomb Brickwork. All bricks should have a bearing of not less than 25 mm in the case of half-brick thick work and 12 mm in the case of one-brick thick work. One-brick thick work should be of full bricks throughout laid as headers, and half-brick thick work should be laid as stretchers.

Panelled Walls—Combination of Walls and Piers : When piers are used to stiffen a wall, the increased strength of the wall to carry vertical loading is not accurately known. The effective thickness of a wall stiffened by piers properly bonded thereto at regular intervals is estimated on the basis of the following table :- (based on the recommendations in the British Standard Code of Practice—111.)

Effective Thickness of a Wall Stiffened by Piers

Ratio : Thickness of pier / Thickness of wall	Ratio : Pier spacing c/c / Pier width: 6 or less	8	10	15	20 or more
1.0	1.0	1.0	1.0	1.0	1.0
1.5	1.2	1.15	1.1	1.05	1.0
2.0	1.4	1.3	1.2	1.1	1.0
2.5	1.7	1.5	1.3	1.15	1.0
3.0	2.0	1.7	1.4	1.2	1.0

"Thickness of pier" means the horizontal dimension measured at right angles to the wall so as to include the thickness of that wall.

The effective thickness of a wall is the thickness of the wall between the piers, multiplied by the appropriate factor specified in the table for the respective ratios.

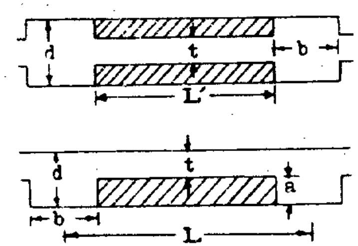
In the case of a wall stiffened by intersecting walls, the effective thickness may be determined from the table on the assumption that the intersecting walls are equivalent to piers of width equal to the thickness of the intersecting walls, and of thickness equal to three times the thickness of the stiffened wall.

Where a wall is required to be more than 9 ins. (1 brick) thick according to the rules prescribed before, the additional thickness may be confined to piers uniformly distributed throughout its length provided the following rules are satisfied, and the height of the wall does not exceed 7.5 m, and the height and length does not exceed 18 times the thickness :—

(*a*) The thickness of the wall between the piers is not less than one-half of the required thickness and (*b*) the collective width of the piers amounts to one-quarter of the length of the wall. The maximum distance apart of piers may be six times their width. "Width of pier" means the horizontal dimension measured parallel to the length of the wall.

Where a load-bearing pier is bonded into a wall, whose thickness is at least two-thirds of the horizontal dimension of that pier, measured at right angles to the length of the wall and so as to include the thickness of that wall, that pier and the portion of the wall to which it is bonded, may together be deemed to be a wall. See also under "Counterforts and Buttresses".

A chimney may be reckoned as a pier if the area of the extra solid materials satisfies the conditions of a pier, and the thickness of the back is not less than 9 ins. (1 brick).



Non bearing panelled walls are usually made of the following dimensions :

(*a*) bd^2 should be not less than $L' \times (d-t)^2$ or $(L-b) \times a^2$.

(*b*) L should not be more than 15 t for work in lime mortar and not more than 20 t for work in cement mortar 1 : 3.

(*c*) d not to exceed b.

(*d*) b not to be less than L/6.

Stresses at the bottom of panelled walls or retaining walls with counterforts :—

Considering one bay of length L :

(*a*) With equal projections on both sides of the wall : Work out moment of inertia of the bay (wall L' plus one projection of length b) about its neutral axis which will be in the centre. Section modulus will be

$$\frac{(b \times d^3) + (L' \times t^3)}{6d}$$

(*b*) When the projection is only on one side of the wall (it should be on the leeward side for boundary walls and on the pressure side for retaining walls), neutral axis of the wall is :

$$\frac{b \times d \times 1/2d + L' \times t \times (d - 1/2t)}{b \times d + L' \times t} = \text{say "a" from the projection edge.}$$

Moment of inertia is worked out about the neutral axis of the bay of bd portion plus L't portion which is = I. There will be two section modulii Z, one for tension and the other for compression.

$Zc=\frac{I}{(d-a)}$ —for compression on the leeward side,

$Zt=\frac{I}{a}$ —for tension on the windward side.

This is for walls where the projection is on the leeward side. From these values the stresses can be worked out as usual, *viz.*, max. compression:

$$=\left(\frac{W}{A}+\frac{M}{Zt}\right);\ \text{max. tension}=\left(\frac{M}{Zc}-\frac{W}{A}\right)$$

All isolated walls should be checked for wind pressure.

Masonry Columns

The height of a column in brickwork above any horizontal section should not exceed 10 to 12 times the least dimension of that section for works in lime and 20 times for works in cement 1 : 4. (This does not apply to bridges.) Reduction factors for slenderness ratio for load bearing columns should be considered as explained earlier.

Relationship between the strength of brickwork and the strength of the individual bricks and of the mortar. Strength of bricks has been given in Section 12 and it has been stated that the strength of brickwork is only about 1/3rd to 1/5th of the individual bricks. Studies made at the Building Research Station, Watford (England) have revealed a very important feature of the mortar and brickwork strength relationship. There is an optimum brickwork strength with a certain strength of the mortar used and there is no advantage in using a stronger mortar with more of cement. With a greater or lesser amount of cementitious material the brickwork is weaker. Use of rich cement mortars for jointing makes the structure unnecessarily rigid and tend to develop cracks. Stronger mortars with stronger bricks and weaker mortars with weaker bricks develop the maximum strength ; for any particular strength of brick, a corresponding mortar strength gives the maximum strength to the brickwork. Cracking of brickwork in practice is rarely due to directly applied loads ; usually it is a result of differential movements between the various parts of structure caused by foundation settlement, or by thermal or shrinkage movements. With a strong and brittle mortar, cracks develop between the mortar and the bricks and may pass also through the bricks themselves. With a weaker mortar, however, the mortar can "give" a little to take up differential movements, and so cracking is often avoided ; should movements be so great that cracking still occurs, it will tend to be distributed throughout the brickwork in the joints rather than through the bricks. A 1 : 3 cement mortar is often specified for brickwork which is needlessly strong, expensive and undesirable for most of the works. A mortar richer than 1 : 3 reduces the strength of the brickwork.

Cement/Lime Mortars. A small quantity of hydrated (non-hydraulic) lime in a cement mortar is almost always beneficial even for high strength construction as it improves the working qualities of the mortar and at the same time reduces shrinkage cracks, as the lime holds the mortar in the mix for a longer period than cement. For high-strength works where rich

cement mortar of 1 : 3 is to be used, lime up to one-quarter of the volume of cement will improve workability without impairing the strength. It will be observed from the table below that there is very little effect on the strength of the brickwork if 50 per cent of cement is replaced by lime (1 cement : 1 lime : 6 sand mix nstead of 1 cement : 3 sand), although mortar strength is reduced by about 40 per cent. A small quantity of cement, say 10 to 15 per cent, in lime mortar makes the mortar stronger and earlier setting.

In gauging mortars, Portland (common) cement or lime should not be mixed with High Alumina cement.

Effect of Mortar Proportions on Strength of Brickwork

Proportion of cement+lime : sand (by volume)	Strength of brickwork expressed as percentage of strength of brickwork built in 1 : 3 cement : sand mortar, for the following ratios of cement : lime (by valume) :—								
	All cement	1 : 1	2 : 3	1 : 2	3 : 7	1 : 3	1 : 4	1 : 9	All lime
1 : 2	96	94	90	87	84	80	74	60	—
1 : 3	100	96	92	89	87	83	79	65	45
1 : 4	—	92	87	84	81	77	71	59	—

Most of the above information has been based on Building Research Station, Watford, England, Digest No. 75.

The lime is slaked, sieved and made into a stiff paste and thoroughly mixed with sand and cement. The quick-lime should not contain over-burnt ingredients which will not readily slake in contact with water. Lime mortar should be prepared in advance of the work and gauged with cement in small quantities as required so as to be used within 2 or 3 hours before the cement has set.

High Alumina cement may be used where high early strength is necessary, and also where resistance to chemical attack is desired. Eminently hydraulic limes should not be added to cement mortars.

The following proportions may not be exceeded :—

(*a*) one volume of cement to not less than one nor more than five volumes of lime ; and

(*b*) one volume of cement-lime mixture to not less than two nor more than four volumes of sand : provided that the ratio of volume of cement to the volume of sand shall not exceed 1 : 16.

A mortar consisting of 1 cement : 1 lime : 9 sand has been used for cheap housing projects, both for brickwork and plaster.

Lime mortar shall be composed of lime mixed with sand in the proportions of one volume of lime to not less than two nor more than three volumes of sand.

Mortar proportions are given in Sections 12 and 20.

Maximum Permissible Compressive Stresses on Brick. Block or Stone Masonry (with slenderness ratio of unity).

Mortar Mix by Volume			Max: Compressive Stress in kg/sq. cm corresponding to units whose crushing strength in kg/sq. cm is :							
Cement	Lime	Sand	35	70	105	210	280	350	530	700
1	0—1/4	3	3.5	7.0	10.5	15.0	17.5	25.5	36.0	46.5
1	0—1/4	4	3.5	7.0	10.0	13.5	16.0	18.5	24.5	24.5
1	1	6	3.5	7.0	10.0	13.5	16.0	18.5	24.5	24.5
1	2	9	3.5	5.5	8.5	12.0	15.0	17.5	24.5	24.5
1	—	6	3.5	5.5	8.5	12.0	15.0	17.5	24.5	24.5
1	3	12	2.5	5.0	7.0	9.0	12.0	14.0	14.0	14.0
—	1	2	2.5	5.0	7.0	9.0	12.0	14.0	14.0	14.0
—	1	3	2.5	4.0	5.5	7.0	7.0	7.0	7.0	7.0

(Based on British Standard Code of Practice—111)

These loads apply on fully hardened masonry.

For randam rubble masonry, the permissible stresses shall be reduced by one-third.

Minimum Allowable Thicknesses of Load Bearing Brick Masonry Walls (cm)

Storeys	Floors 1	2	3	4
1	20	—	—	—
2	20	20	—	—
3	20	20	20	—
4	30	20	20	20

(do not include mud mortar)

Buildings of more than four storeys are not economical with walls built to support roofs and floors. Increased thickness of walls make the structure costlier than framed one.

Crushing strengths of Indian bricks are very variable, values are given in Section 12. The following proportions of mortars are recommended to be used for the particular strengths of bricks :

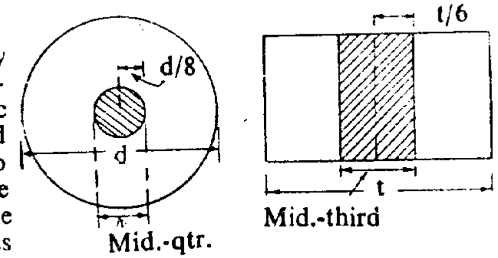
(i) for low-strength bricks—105 kg/sq. cm
1 cement : 1 to 2 lime : 9 sand

(ii) for medium-strength bricks—210 to 350 kg/sq. cm
1 cement : 1 lime : 6 sand

(iii) for high-strength bricks - 500 kg/sq. cm or above
1 cement : 3 sand, to which lime, up to 1/4 of the volume of cement may be added to improve workability.

LATERAL STABILITY OF WALLS

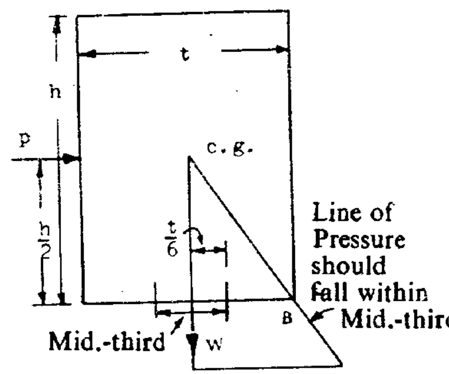
Centre of Pressure :



For the stability of masonry structures there should be no tension produced due to eccentric loads, wind pressure or inclined thrusts (from arches, etc.). To avoid tension the resultant force or the line of pressure must lie within middle-third of the cross section at the base of rectangular sections and middle-quarter of circular sections. When the resultant falls at the edge of the middle-third, the factor of safety against overturning is between 2 and 3, and when it falls within middle-quarter, the factor of safety is 4.

Design of a Wall Against Wind Pressure (or any Horizontal Force)

Wind pressure is the principal force tending to disturb the stability of isolated walls. Greatest stress produced is at the bottom of a wall where the wind exerts the greatest turning moment ; if the thickness in variable, the lowest point at each change of section should be considered. Locate the point where the resultant of all the forces acting on the wall cuts the base. This will fix the thickness of the wall. Example :—



Equating the resisting moment of the wall to the moment due to wind pressure ;

p = wind pressure per sq. m.

w = weight of the wall per cu.m plus any load on it.

Resisting moment of the wall = weight of the wall per unit length × distance from the point where the centre of gravity (c.g.) of the wall will cut the base, to the edge. (Unsymmetrical walls can be divided into separate blocks and each block tested for stability as an independent unit. Taking moments about B : (considering one metre length of wall)

$$ph \times \frac{h}{2} = wth \times \frac{t}{2} \quad \text{or} \quad t = \sqrt{\frac{ph}{w}}$$

This gives the expression for the thickness of the wall where the wall will overturn. The line of resultant pressure will pass through B.

The resultant pressure to pass at the middle-third point, the expression will be :

$$ph \times \frac{h}{2} = wth \times \frac{t}{6} \quad \text{or} \quad t = \sqrt{\frac{3ph}{w}}$$

This determines the thickness of the wall.

The whole of the base is under compression.

Factor of Safety = Resisting moment/Overturning moment.

When the resultant cuts the base at the 1/3rd point, the pressure on the heel of the wall is zero and on the toe it is twice the average, *viz.*, 2w/A.

For compound walls (not of much importance) the factor of safety against overturning may be taken as low as one, in which case the wall will have small tension but will stand. In concrete walls leverage may be taken up to 5/12 t instead of 1/6 t. It will have a tension of about 56 tonnes/sq. m due to normal wind pressure.

The position of the resultant—"line of pressure" can also be determined graphically (as shown in the sketch).

Stresses at the base can be calculated from the usual equation :

$$\left(\frac{W}{A} \pm \frac{M}{Z}\right) = \frac{W}{A}\left(1 \pm \frac{6 \times e}{t}\right)$$

$$\frac{ph^2}{2} = \frac{fI}{y} = fZ = \frac{fbt^2}{6}, \quad \text{or} \quad f = \frac{3ph^2}{bt^2}$$

Whence, tension $= \left(f - \frac{W}{A}\right)$ and compression $= \left(f + \frac{W}{A}\right)$

This gives tension and compression at the edges/sq. m

M = Bending Moment = W × e

Z = Section Modulus $bt^2/6$

W = total vertical load, (including weight of wall)

A = bt = area of the base. b is taken 1 m strip and t is the width of the base.

e = deviation from the centre of the base t of the resultant = M/W

For stability against sliding see under "Retaining Walls".

Inclined Thrusts. Inclined thrusts should be resolved into their horizontal and vertical components. The vertical component is added with the weight of the wall :

$H = T\cos\phi$ H = horizontal component,

$V = T\sin\phi$ V = vertical component,

T = inclined thrust,

ϕ = angle of the inclined thrust with the horizontal.

To find out the thickness of the wall in order that the base may just be wholly under compression, the moment due to weight of the wall plus the moment due to the vertical component of the thrust are equated to the moment due to the horizontal component of the thrust.

Moment due to the wall = weight of the wall × $\frac{t}{6}$

Moment due to V = V × 2/3t

Moment due to H = H × height of the thrust point above the ground level.

"Wind Pressure" has been dealt with in detail in Section 11. For surfaces which are not plane multiply the normal wind pressure for a flat surface by the co-efficients given in the following tables :

Plan section of surface	Ratio of height to base—width		
	0 to 4	4 to 8	8 to 16
Circular	0.6	0.65	0.7
Hexagonal	0.7	0.8	0.9
Octagonal	0.8	0.9	1.0
Square	1.0	1.15	1.3
Cup-shaped	1.3	1.4	1.5

Corbels and Cornices shall not ordinarily project more than 15 to 20 cm, and this projection shall be obtained by projecting each brick (or stone) course by not more than 1/4th of the brick (or stone) length measured in the same direction as the projection and which should be only 1/8 where strength is required. For cornices projecting more than 20 cm and requiring more than quarter bricks projection, metal cramps shall be used. For larger projections stone or concrete slabs should be used well bonded into the wall with sufficient superincumbent weight of masonry to hold it back in place. (See under "Sun-shades".)

4. MASONRY ARCH DESIGN

(See also under "Bridges")

Choice of Type of Arch. Where abutments are of amble size, the segmental arch is the strongest, but where the abutments are made small, semi-circular or pointed arch should be used. Semi-circular arches are the strongest and exert no thrust on abutments or piers.

Fixing Rise and Radius of Arches. A good rule for the radius of segmental brick arches over doors and windows or other small openings is to make the radius equal to the width of the opening. For segmental arches the rise of the arch at the crown is 1/10 to 1/12 of the span. Rise of an arch between 1/4 to 1/2 of the span is considered most econonomical for buildings.

Fixing Thickness of Arch Rings. There are many empirical formulae for fixing the thickness of an arch ring. For brick arches : (English formulae)

$T = \sqrt{0.20 \times R}$—for single semi-circular or oval arches.

$T = \sqrt{0.25 \times R}$—for arches in series.

Trautwine's formula : for cut stone work

$$T = \frac{\sqrt{R + 1/2S}}{4} + 0.2 \text{ ft.}; \quad R = \frac{(1/2S)^2 + r^2}{2r}$$

Rankine's formula : $T = 0.4\sqrt{S}$

R is radius at crown ; S is span ; T is thickness of arch ring ; r is rise, —all in feet.

A good common rule for light load brick arches is to make 4.5 ins. (1/2 brick) thickness rings for each 1.5 m (5 ft.) of span. For heavy loads the thickness of arch rings may be taken for brickwork in cement mortar 1 : 4 as follows :

Up to 1.5 m (5 ft.)—9 ins. (1 brick)
1.8 to 4.3 m (6 to 14 ft.)—13.5 ins. (1.5 bricks)
4.6 to 7.6 m (15 to 25 ft.)—18 ins. (2 bricks)

The thickness of arch ring at springing may be taken the same as at crown for small spans. In case of large spans of over 6 m, the thickness at the springing should be increased by about 20 per cent. In case of well-dressed stone arches, the thickness of arch ring may be reduced to about 3/4th.

Construction of Arches

In building arches for all ordinary works, concentric rings with non-continuous radial joints (of half brick) may be used, but this method should not be adopted for heavy loads or for spans above 7.5 m ; or alternatively, a number of keys should be provided at intervals. The other method is to lay bricks alternatively as headers and stretchers in section with continuous radial joints as shown in the sketch for 'Flat Arch'. Where arches are built in 4.5 ins. (1/2 brick) concentric rings, each ring should be fully completed before the one above it is commenced.

Plain brick arches (where bricks have not been cut or rubbed) or rough brick arches, should be turned in half brick rings. The mortar in the joints that are parallel should never exceed 3 mm in thickness.

It is preferable to provide all arches of 1.8 m (6 ft.) span and above with keys. For spans 1.8 to 3.7 m (6 to 12 ft.) there should be one key at the crown, and for spans above 3.7 m, additional keys should be provided so that the distance between keys is not more than 2.7 m (9 ft.) measured along the intrados.

Keys should extend over the full thickness of the arch. Headers and stretchers in keys should be so arranged as to break effectively the bond in the adjacent arch rings.

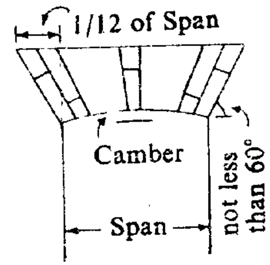
Arch-work should be carried up evenly from either abutment and as soon as the arch is complete the masonry should be built up to the height of the crown so as to load the haunches.

In any structure of more than four spans, not less than four spans should be completely centred at one time ; where there are three spans or less, all the spans should be complctely centred before the building of any of the arches is commenced and the building of the several arches should be simultaneously executed.

Reinforcing Arches. Hoop-iron if laid around an arch between half-brick rings and also longitudinally and radially, will strengthen a brick arch considerably. The bands of hoop-iron which traverse the arch radially should preferably be bent and prolonged into the bed joints of the backing and spandrels.

A Relieving arch is built over a lintel or a flat arch to relieve the latter of the superincumbent weight.

A Flat arch has a camber of about 1/100 of span. It should have angle of not less than 60 deg. at the skewback which gives a projection of about 1/12 of span at the top (extrados) beyond the span.



FLAT ARCH

Jack Arches are usually made for 1 to 1.5 m spans, 4.5 ins. (1/2 brick) thick. Rise is about 1/6 to 1/8 of the span. Where bigger arches are required, the thickness of the arch ring should be increased to 9 ins. (1 brick) and rise 1/4th of span.

Tie-Rods are provided for the end spans or under concentrated loads. In series of arches it is safer to provide rods every 4th or 5th span. Tie rods are generally 12 to 16 mm dia. for small spans and 20 to 25 mm dia. for big spans, and 1 to 1.5 m apart. Spacings of the tie rods should not exceed 20 times the width of the supporting beams.

The thrust of the arch which is to be resisted by the tie rods can be found from the formula :

$$T = \frac{12.5 \, WL^2}{R}$$

Where : T = horizontal thrust of the arch in kg per running metre ;

W = live and dead load on the arch in kg/sq. m on end span, and live load only for intermediate spans ;

L = span of arch in metres ; R = rise in cm.

From this the size of the tie rods and the spacing can be calculated.

Small size of rods and as near as possible should be preferred.

Tie rods should be designed on the net area under the threads.

The tie rods should be placed generally not less than 7.5 cm above the bottom of the joist, and should be properly anchored.

If the load is concentrated at the centre of the arch the thrust will be twice that given by the above formula.

Jack arches are supported either on the lower flanges of I-beams or on top of the upper flange. Old rails are also used where available cheaply. RCC pre-cast beams can be used instead of rolled beams ; such beams can be made with splayed sides for resting the ends of the arches. Steel beams may be encased in cement concrete. Beams are designed as simply supported with distributed roof load. For design of beams, see Table at page 4M/18.

To prevent lateral displacement of the beams under the thrust of the arch, at least three complete lines of centerings should always be in use. The first two courses adjoining the joists should be laid in 1 : 3 cement mortar and the remaining may be with lime or 1 : 5 cement. The bed face of the brick (9 ins. × 4.5 ins. or 20 cm × 10 cm) should be made normal to the arch thrust. The portion of the beam exposed above the arch should be cased with 1 : 2 : 4 cement concrete with small aggregate. Haunches when filled should give a thickness of at least 40 mm over the crown of the arch, which may be with fine lime concrete.

Centerings for Jack Arches. There are various methods of making centerings either of wood or iron. Centerings are generally hung from the roof beams by iron clips or wires. Centerings may not be moved till after consolidation of the concrete terracing. Some engineers prefer to remove centerings twelve hours after the completion of the segment of jack-arch over it, provided the adjacent jack-arches have been completed.

For construction of brick Jack arches centering for the full length is not required. Centering is made in small convenient lengths which are shifted over.

For anchorage of beams tie rods may be fixed with washer plates and nuts outside the end walls, of size about 15 × 15 cm, 6 mm thick. In case the thickness of the wall is not sufficient to afford full enchorage, angle iron of size about 75 × 75 × 6 mm or I-beam may be used, embedded in the wall.

Striking Centres of Arches. Centres should be struck as noted below :--

(*i*) Single segmental arch : Immediately after the arch is finished, or within 24 hours.

(*ii*) Series of segmental arches : Centres of each arch should be struck as soon as the arch succeeding it is completed.

(*iii*) Semi-circular, eliptical or pointed arches : As soon as the adjacent brick-work has reached two-third of the height of such arches and the mortar has had time to set and harden.

For striking of the centres of large brick arches, some engineers are of the opinion that it should be delayed for 2 or 3 months and during that period the arches should be eased a little from time to time.

5. RETAINING WALLS & BREAST WALLS

A Retaining Wall is a wall built to resist the pressure of earth filling or backing, deposited behind it *after it is built.*

A Breast Wall (or face wall) is a similar structure to retaining wall built to protect the freshly cut surface of a natural ground, whether with vertical or inclined face, to prevent it from fall due to the action of weather.

Angle of Repose :

If a mass of earth (clay, sand, gravel or any such material) is left exposed to weather for sometime, its sides will slip and will gradually attain a stable slope without tending to slide. The angle between the horizontal and this slope is termed the natural angle of repose for that particular material.

Angle of Internal Friction :

The angle of internal friction has been explained in detail under "Soil Mechanics". The angle of repose of a soil is not the same as its angle of internal friction since the angle of repose is determined by the unstable layers at the surface of the slope, loosely tipped, whereas internal friction includes not only intergranular friction but also interlocking. The angle of repose, however, is approximately equal to the angle of internal friction of sand in a loose state. The angle of internal friction for dense sands varies between 35 deg. and 46 deg. and for loose sands between 28 deg. and 34 deg. That for dense well graded gravel may be as much as 50 deg. Some engineers consider that in all formulae of soil pressure the angle of internal friction should be used and not the angle of repose.

Angle of Friction. The limiting angle of a plane (surface of any material) to the horizontal on which plane rests a block which is just on the point of sliding. This angle depends upon the value of the co-efficient of friction between the two surfaces, *i e*, the plane and the block. The tangent of the angle of friction is known as the "co-efficient of friction" or the "co efficient of sliding friction".

Back filling. That portion of the material retained by the wall (including special filter material), which has been placed behind it after construction to fill in the space between the wall and the natural ground.

Backing. All the material retained by the wall.

Earth Pressure. Any pressure exerted by or through the retained soil at the back of the wall, usually an active pressure or thrust.

Active pressure or active earth pressure. The lateral pressure exerted by the soil on the back of a wall.

Passive pressure or passive resistance. The lateral resistance of the soil on the front of a wall.

Surcharge or surcharge load. The part of the material or load supported by a retaining wall, at a level above the top level of the wall, which may, by virtue of its nature or position increase the active earth pressure on the wall.

Toe wall is a small retaining wall built at the foot of an earth slope.

Stability of Bank Slopes. The safe slope of a granular bank does not decrease as the height increases because its shearing strength increases as the bank becomes higher (due to additional weight). The safe slope of a clay bank becomes flatter as the height of the bank increases because its shearing strength does not increase to resist the corresponding increase in the height. The steepness of the safe slope of an embankment depends on the shearing strength of the soil. A slope is in itself stable provided that the angle of slope is less than the angle of repose of the material. This applies to dry materials. Most slips occur in cohesive soils due to increase in moisture content. For granular soils, the resistance in sliding is dependent upon the angle of internal friction. For sands a very small movement only is required to change the soil from at rest state to the active state.

The natural, strongest and ultimate form of earth slopes is a concave curve in which the flattest portion is at the bottom. In constructing slopes the reverse of this form is most often made, which invites slips. Straight or convex slopes continue to slip until the natural form is attained. Therefore, in cuttings, concave slopes should be formed to avoid slips.

Slopes in Cuttings depend on the strength of material, depth of excavation and the bedding plane. (i) In clays and silts : 2 to 1 when well drained, and 3 or 4 to 1 when wet. (ii) In gravels and sands : 1 or 1.5 to 1 when not more than 6 m high, and 1.5 or 2 to 1 when in greater height. (iii) In fine sands : 2.5 to 1. (iv) Undisturbed earths : 1 to 1. Slopes of embankments is given in Sections 17 and 18.

Any material likely to slip must be removed and the slope flattened. If a slope passes through two different soils, the top being more likely to slip; the top should be trimmed to a flatter slope and the bottom kept at a steeper slope. Where the slope is very high, it must be reduced by cutting and inserting one or more berms. Retaining walls (toe walls) or piles are provided at the toe of the slopes to prevent slipping. The possibility of occurrence of slips is greater with cuttings than with embankments. After the construction of a cutting it should be seeded with grasses. The roots strengthen the soils and they also draw moisture from it which adds to stability. Alternatively, pitching may be done.

Design of Retaining Walls. A vertical wall backed with earthy materials is subjected to a thrust which tends to overturn it as well as cause it to slide. In the design of a retainiag wall the thrust on the back of the wall due to earth pressure is calculated and the wall made of such a cross-section (thickness) the weight of which will resist the movement due to the thrust and the resultant force on the base lies within the middle-third to ensure that there is no tension at the base In cohesive soils the resultant should fall at or near the middle of base, because in such soils unequal settlement of the toe may start a vicious circle. It is also necessary to check that the stress at the toe does not exceed the allowable bearing capacity of the soil and that the wall does not fail by sliding bodily forward on the base. The pressure at the back of the wall is greater the heavier the material and the less the angle of repose.

Two theories are commonly used for the design of retaining walls : (*i*) Rankine's theory, and (*ii*) Coulomb's Wedge theory. The Rankine and Coulomb solutions give the same result for a vertical wall and a horizontal backfill if no wall friction is allowed for. For sloping backfills and walls off the vertical the Wedge theory is uneffected in its application but in Rankine's theory the resultant acts parallel to the slope of the fill. Rankine's formulae have been the most generally used for common works. There is another theory, called Dr. Scheffler's theory but the overturning moment obtained with this theory is less than that obtained by the above two theories.

When slopes are expressed as 1.5 to 1 ; 2 to 1, the first figure represents the horizontal measurement, the second vertical measurement. Similarly is 1.5 : 1 ; 2 : 1 or 1 : 4.

Rankine's theory assumes the earth a homogeneous, incompressible granular mass with no cohesion and is thus more appropriate for cohesionless soils, *e.g.*, sand, gravel, broken stone. It makes no allowance for adhesion or for friction between the earth and the back of the wall. Although these assumptions are not always correct but give results erring on the side of safety.

In the case of walls backed by hard clay, the Wedge theory may give results in excess of the actual, whereas walls backed by soft clay may be subjected to greater pressure than those given by the Wedge theory.

Soil	Angle of repose	
Wet clay ; wet sand and clay or wet gravel and clay. (Soils not properly drained)	20°	Average angle of repose for common designs may be taken as per table appended
Dry clay ; wet sand ; gravel	27°	
Dry sand ; loose earth, dry or wet ; damp clay ; gravel, sand and clay ; common soil, (properly drained)	33°	
Sand and clay ; gravel and sand	37°	

For retaining walls of considerable height supporting soft clay (cohesive soils), Bell's formula is recommended. Intensity of pressure at any depth

$$=wh\left(\frac{1-\sin\phi}{1+\sin\phi}\right)-2c\tan\left(45-\frac{\phi}{2}\right)$$

Where : c=cohesion in kg/sq. m, which may be taken for very soft clays—1000 to 1700 ; soft clays—1700 to 3500 ; firm clays 3500 to 5000 ; fairly, stiff clays - 5000 ; stiff clays - 7500 to 10000 ; very stiff clays—10000 to 15000 ; sands predominating with some clay - 2000 ; cemented sand and gravel—2500, sand-gravel mixture cemented with clay - 5000.

Coulomb's Wedge Theory. In theory it is assumed that there is a wedge having the wall as one side and a plane called the "plain of rupture" as the other side, and this wedge-shaped mass of earth tends breaking away and slide down and forward, thus exerting thrust on the wall, and the wall has to support this wedge. For a vertical wall without surcharge the plane of rupture bisects the angle between the plane of repose and the back of the wall. This theory, however, does not determine the direction of the thrust, and which is taken to act horizontally at 1/3 the height of the supported backing above the base of the wall, for walls with or without surcharge. For a wall with a vertical back and level backing the overturning moment due to pressure is the same as with Rankine's formula.

Simplified "Wedge theory" formula :—

$$P=\frac{wh^2}{2}\times\frac{\cos^2}{\left[1+\sqrt{\frac{\sin\phi\,.\,\sin\phi\,(\phi-\delta)}{\cos\delta}}\right]^2}=\frac{wh^2}{2}\times K_1$$

For a wall with a vertical back and a level backing, $\delta=0$.

Hence $P = \frac{wh^2}{2} \times \frac{\cos^2\phi}{(1+\sin\phi)^2} = \frac{wh^2}{2}\left(\frac{1-\sin\phi}{1+\sin\phi}\right)$

Rankine's formula for walls without surcharge :

$$P = \frac{wh^2}{2} \times \frac{1-\sin\phi}{1+\sin\phi}$$

The intensity of pressure at any depth is $wh \times \frac{1-\sin\phi}{1+\sin\phi}$

The earth pressure acts horizontally at 1/3 the height* of the supported backing above the base of the wall. The overturning moment due to earth pressure $= \frac{wh^3}{6} \times \frac{1-\sin\phi}{1+\sin\phi}$. This is equated with the moment due to the weight of the wall acting through its centre of gravity, as explained earlier under "Design of Wall Against Wind Pressure".

P = total lateral earth pressure per m length of wall due to the backing.

w = weight of backing per cu. m,
h = height of backing,
ϕ = angle of repose of backing,
δ = angle of surcharge.

Rankine's theory assumes earth to be perfectly dry without any adhesion. In practice, the filling behind a wall is exposed to varying atmospheric conditions and seldom remains perfectly dry. Therefore, in dry locations or even where the backfill is properly drained, the partial hydrostatic pressure exerted by a wet filling should be taken into account. No structure should be designed for less than the equivalent fluid pressure of 500 kg/cu. m. (Dry earth pressure is generally equal to 1/3 of water pressure.) (See under Saturated Back-fills.)

Lateral Pressure on Walls due to Superimposed Load on Backing or Fill—Earth Pressure on Abutments : Load on the backing will occur due to traffic on a bridge approach or a road supported by a retaining wall. Any load at the back of the wall is reduced to an equivalent height of backfill (surcharge) corresponding to its weight.

*Some authorities consider the centre of pressure exerted by the backfill, when considered dry, located at 0.42 h for a level fill instead of h/3.

$P = 1/2w\,(h+h')^2 \times \frac{1-\sin\phi}{1+\sin\phi}$

$y = 1/3\,(h+h')$

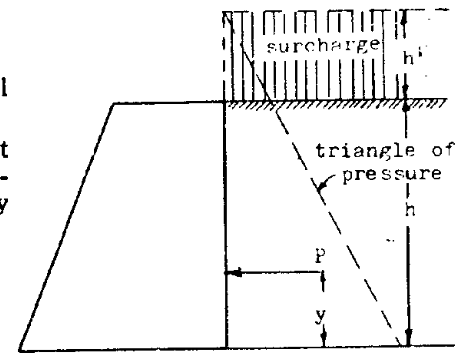
P = resultant pressure per m of wall length,

h′ = height of earth-fill equivalent to surcharge in m (superimposed load per sq. m divided by weight per cu. m of fill.)

Another method is :

$P = 1/2wh(h+2h') \times \frac{1-\sin\phi}{1+\sin\phi}$;

$y = 1/3h \times \frac{h+3h'}{h+2h'}$



ABUTMENT

Return Walls. A return wall is a retaining wall built parallel to the centre line of a road to retain the embankment.

Road traffic exerts a load on the backfill of a retaining wall. This may be taken equivalent to a surcharge of one metre for a medium density traffic roads. (See Design of Abutments under "Bridges".)

Live load due to road traffic or any isolated load is assumed to be dispersed through the backfill at an angle of 45°. In case the 45° line falls away from the wall there will be no surcharge load. No surcharge load need be considered when it does not come within a distance of h/2 from the top of the wall.

$P_1 = 1/2wh^2 \times \frac{1-\sin\phi}{1+\sin\phi}$

$y_1 = 1/3h$

$P_2 = w_1 h' \times \frac{1-\sin\phi}{1+\sin\phi}$

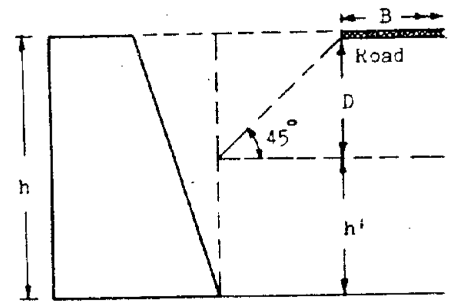
$y_2 = 1/2h'$

Total pressure moment is

$$P_1 y_1 + P_2 y_2$$

P_1 = resultant pressure due to fill only,

P_2 = resultant pressure due to surcharge,



RETURN WALL

$w_1 = \frac{W}{(B+2D)}$; W is surcharge load due to live load on road width B, or any isolated load on width B.

The portion of the backfill which is over the heel of the wall should be taken with the weight of the wall.

Wing Walls : (Surcharged walls)

$$P = 1/2\ wh\ (h+2h) \times \frac{1-\sin\phi}{1+\sin\phi}$$

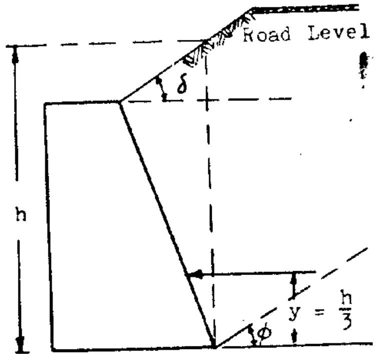
$y = 1/3h$

h'=equivalent level surcharge due to the sloping fill.

$= 1/3h \cot\phi \tan\delta$

ϕ=angle of fill,

δ=angle of surcharge.



Surcharged Retaining wall with Rankine's Theory

For angle of surcharge δ between o° and ϕ°

$$P = \frac{wh^2}{2} \cos\delta \frac{\cos\delta - \sqrt{\cos^2\delta - \cos^2\phi}}{\cos\delta + \sqrt{\cos^2\delta - \cos^2\phi}} = \frac{wh^2}{2} \times K_2$$

where $\delta = \phi$ $\qquad P = \frac{wh^2}{2} \cos\phi$

Here the resultant P acts parallel to the surcharge slope of the fill and although it is greater than for a level fill it has a smaller leverage and the moment is not much greater than for a level fill.

The vertical component of the inclined thrust P is added with the weight of the wall where back is inclined.

The value of K_1 in Coulomb's formula and K_2 in Rankine's formula may be taken from the following table :

Value of K_1, or K_2

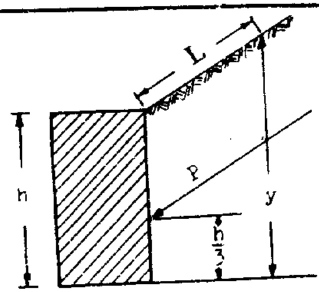
Angle of Surcharge	Angle of Repose 15°	20°	25°	30°	35°	40°	45°
10°	0.70	0.57	0.46	0.37	0.29	0.23	0.17
20°	—	—	0.54	0.42	0.32	0.25	0.19
25°			—	0.49	0.37	0.27	0.21
30°				—	0.44	0.32	0.23
35°					—	0.39	0.27
40°						—	0.34

Another simple method for surcharged fills

Take L=h,

Take y as the height instead of h and design the wall as for level backing.

P acts parallel to the surcharge slope at h/3.



Slope	1 : 1	1.5 : 1	2 : 1	3 : 1	4 : 1
Value of y	1.71h	1.55h	1.45h	1.31h	1.23h

Passive resistance of earth in front of toe of wall

Total passive resistance$=\frac{wd^2}{2}\left(\frac{1+\sin\phi}{1-\sin\phi}\right)$ per m, length of wall, acting horizontally at 1/3d above the base of the wall. d is depth below earth surface. Passive resistance is taken not more than 50 per cent of that given by the above formula, and for small depths of earth, is generally neglected.

Saturated Back-Fills. Backfills when saturated due to lack of drainage (or in water logged areas) exert much heavier pressure, especially clayey soils. The lateral pressure will be the total of the force due to the water plus the force of the submerged backing. Pressure due to water will be $\frac{wh^2}{2} \times \frac{h}{3}$, where w is the weight of water and h is the height to which water will occur. The pressure due to the submerged backing can be obtained as described earlier for dry materials with the angle of repose and weight of the soil under water as given in the table following, or, the ordinary dry soil pressure is reduced in the ratio—weight of soil under water/weight of dry soil in air. The angle of repose of a saturated soil is about 5 to 10 deg. less than the angle of repose of the dry soil. The weight of a soil under water=(weight of the soil in air+percentage of voids in the soil × weight of water)−weight of water.

Material	Average angle of repose under water	Weight under water, kg/cu. m
Sand	27°	960
Sand and clay	18°	1040
Clay	16°	1280
Gravel, clean	27°	960
Gravel and clay	18°	1040
Gravel, sand and clay	18°	1040
Soil	16°	1120

Retaining walls with backfills of earth charged with water may be designed for a total pressure of 1200 kg/cu. m and other walls where precautions are necessary against cracking, such as water reservoirs or dams, may be designed for a total pressure of 1600 kg/cu. m.

Sliding: Frictional resistance between wall and ground :—

The force that tends to slide a retaining wall on the foundation soil is the horizontal component of the earth pressure (including any super-imposed load on the backing). In order to prevent sliding, total horizontal force÷total vertical force, should not be more than the safe co efficient of friction between the wall and the ground. Where practicable, a factor of safety of 2 should be obtained against sliding, and it should in no case be less than 1.5. All walls above 3 m height should be checked for safety against sliding forward. Passive resistance due to any solid earth in front of the toe of the wall is not usually allowed for unless of great depth and reliable.

Total vertical force is taken to include the weight of the backfill if on the wall (where the wall is inclined or stepped), vertical component of the backfill pressure if inclined, load due to a bridge or any such structure as in the case of an abutment.

If the resistance to sliding is too little, base width of the wall will have to be increased or preferably, the base should be sloped down towards the backfill. Foundations will be normal to the face in the case of walls with battered fronts, and in the case of vertical fronts the foundations can be sloped down about 1 in 8.

Surfaces	Co efficient of friction	Surfaces	Co-efficient of friction
Masonry on moist clay	0.33	Limestone on same	0.75
,, dry clay	0.50	Cement blocks on same	0.65
,, sand	0.40	Cement concrete on clay	0.20
,, gravel	0.60	,, ,, sand	0.40
,, same or brick	0.70	,, ,, gravel	0.40
,, rock	0.75	Bricks on same	0.65
Fine cut granite on same	0.60	Wood on same	0.48

The co-efficient of friction is the tangent of the angle of internal friction; values are only approximate.

Foundations should be taken deep enough to safeguard against weather, and should be at least $h/10+30$ cm below ground level. The projection of any footing course should not exceed half the depth of the course. 7.5 cm steps may be given at the heel. The maximum pressure on the toe should not exceed 0.7 of the permissible ground bearing pressure. Where, however, the pressure exceeds the allowable limits, the toe T can be projected in the following proportions to decrease the toe pressure.

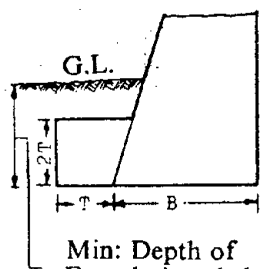
For T=o pressure p becomes = p

For T=B/6 ,, ,, ,, = .62p

For T=B/5 ,, ,, ,, = .56p

For T=B/4 ,, ,, ,, = .48p

For T=B/3 ,, ,, ,, = .37p



Min: Depth of Foundations below G.L.=2T+30 cm

Example :

$P=1/2wh\,(h+2h')\,\frac{1-\sin\phi}{1+\sin\phi}$ (formula)

Effective height for earth pressure=H'

Equivalent level surcharge due to the sloping fill
$=1/3H\cot\phi\tan\delta$

H' for 1.5 : 1 surcharge and 1 : 6 batter of wall

$=H-\left(\frac{H}{6}\times\frac{2}{3}\right)=\frac{8}{9}H$

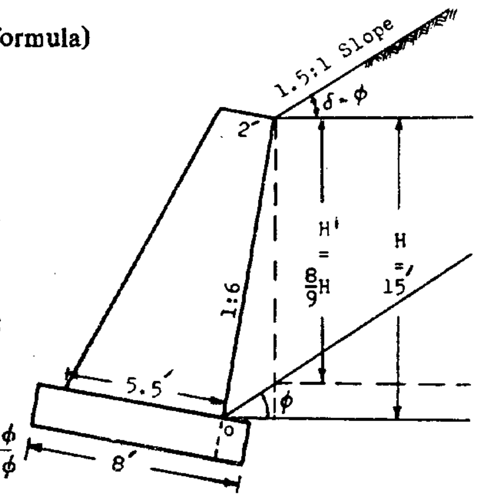
If $\phi=\delta$, then $1/3H\cot\phi\tan\delta$ will be 1/3H'
=level surcharge=4.44'

Then

$P=1/2wH'(H'+2/3H')\frac{1-\sin\phi}{1+\sin\phi}$

$y=1/3H'+(H-H')=6.1$ ft.

Earth Pressure=Py

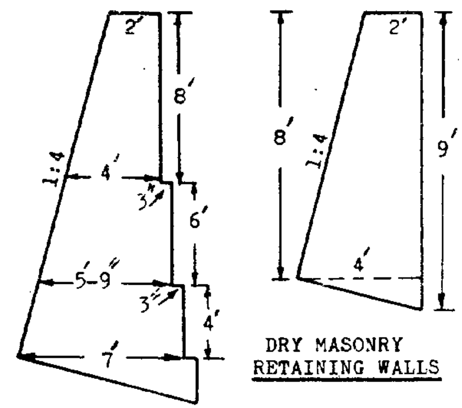


Work already Built

Take moments about O for test at ground level.

Dry Retaining Walls

Take top width about 2 ft. (60 cm), front batter 1 : 4 to 1 : 3 (hor. : ver.), and back vertical. All courses normal to the face; foundations at right angles to the face batter, which should be in lime concrete if the soil is impermeable. If the height exceeds 12 ft. (3.6 m) provide 12 in. (30 cm) courses in lime at every 2 to 6 ft. (0.6 to 1.8 m) height; through bond stones at intervals of 5 ft. (1.5 m).

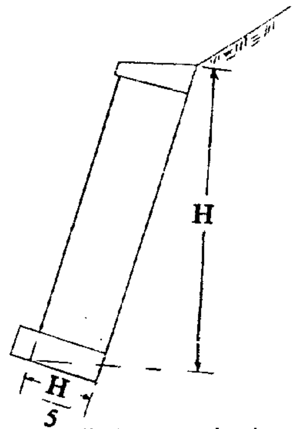


DRY MASONRY RETAINING WALLS

Long lengths of dry rubble retaining walls should be divided into panels separated from one another by short lengths of walls 5 to 7 ft. (1.5 to 2.1 m) long built in mortar at intervals of say, 20 to 30 ft. (6 to 9 m), in order to confine subsequent damage, if any, only to the panels affected.

Special care is required for the stability of the walls in the selection of stones as regards their shape and size since there is no mortar. All material should preferably be collected at site for selection before commencing the work. Occasionally, the interstices of dry stones are filled with shingle or small pieces of stones and the face cement pointed.

Breast Walls



Most of the soils generally can stand a steep slope immediately they are cut but after some weathering they start falling down to their natural angle of repose. Breast walls are built soon after the cutting. The section of a wall may be as shown in the figure, built with front and back batters of 1 in 2 to 1 in 4 (hor. to ver.) with top 60 cm wide, according to the slope of the soil. Where cut or fill slopes intersect the original ground surface, slopes are to be rounded to blend with the natural ground surface.

Revetment Walls have the same function as retaining walls, *viz.*, to keep in safe equilibrium masses of earth and to prevent sliding action. They usually have projections like butterssses at intervals.

Counterforts and Buttresses. Retaining walls are often built with counterforts and, boundary walls with buttresses at short intervals which allow of the average section of the wall being made less than would otherwise be required.

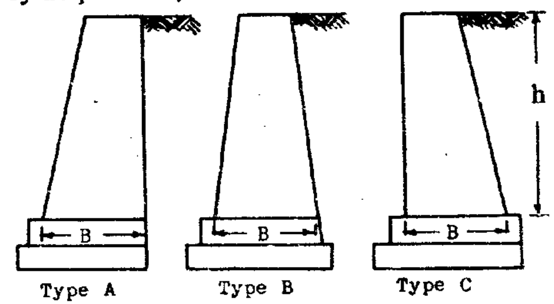
Design of counterforts is based on the same principles as for panelled walls. The distance apart of counterforts generally varies from 3 m for low walls to 6 m for high walls. About one-eighth of the mass of masonry (wall plus counterfort) is taken for the counterforts (calculation is made per bay) and height is kept below the top of the wall by about 1/4th to 1/5th of the height of the wall. The thickness of counterforts at top is equal to the thickness of the wall at the top and length about one-fifth of their distance apart. Counterforts and buttresses are generally made stepped or sloping, giving greater thickness at the bottom. Thin counterforts at frequent intervals are more satisfactory than thicker ones at longer intervals.

Rules for Retaining Walls. Top width of retaining walls should be not less than 60 cm for stone masonry and 55 cm for brick walls, and which should be one m for railway works. A bottom thickness of one-third will do for most of the works. The front face should have a batter of at least 1 in 24, with 1 in 6 maximum ; most common is 1 in 12. A batter economically adds to the stability of the wall for any outward displacement. Front batter if more than one-sixth, will let in moisture and encourage vegetable growth in the joints. For brick walls, after every five courses from top increase thickness by half a brick. Long walls should have counterforts at the back.

The back should be left rough or built in steps to increase friction between the wall and the backing. Backfill should be deposited in 10 cm to 15 cm layers with moderate compaction sloping downward from the wall to reduce lateral pressure, after the wall has attained sufficient strength. Backfilling of the walls should be done with granular materials

or small stones hand packed in the form of active wedge pressure (to the angle of repose of the soil) in clayey soils. This will decrease the back pressure on the wall and help drainage.

Design Data for Retaining Walls for Level Fills. (For surcharged fills increase the base width by 20 per cent.)



K : is the ratio of the weight of wall masonry and weight of the soil material per cu. m; p : is the max. toe pressure in kg/sq. m at the base.

The resultant falls at the edge of the middle-third and the walls will have a factor of safety of between 2 and 3 against overturning.

Average Angle of Repose Deg.	Base width B—given as ratio of height h: Type A	Type B	Type C	Rectangular	Min: width for a rectangular wall that will just stand the earth pressure
			K = 1.00		
20°	.68	.65	.73	.73	.40
27°	.57	.55	.62	.62	.35
33°	.48	.47	.54	.53	.31
37°	.46	.45	.50	.50	.28
p=	600 h	800 h	1000 h	1000 h	
			K = 1.25		
20°	.61	.62	.73	.65	.36
27°	.51	.52	.62	.55	.31
33°	.43	.44	.53	.47	.28 } common
37°	.41	.42	.50	.45	.25 } common
p=	750 h	950 h	1100 h	1200 h	
			K=1.50		
20°	.54	.60	.73	.59	.33
27°	.46	.50	.62	.50	.29
33°	.39	.42	.53	.43	.26
37°	.37	.40	.50	.41	.24
p=	950 h	1100 h	1300 h	1500 h	

Reports show that walls built with the above dimensions have stood well for years. Walls do not generally fail for less thickness but fail due other reasons.

Material		Angle of Repose Deg.	Wt. in kg per cu. m of soil
Ashes	...	40	800
Clay, wet	...	15—20	2200—2550
„ damp, well drained	...	30—45	2000—2550
„ dry	...	25—30	1800—2080
Sand, wet	...	15—30	1800—2000
„ moist	...	30—45	1600—1800
„ dry	...	25—35	1450—1600
Earth, loose	...	30—45	1600
„ rammed	...	50—65	1900
„ dry	...	30—40	1800
„ moist	...	45—49	1600
Sand and clay, wet	...	18—20	2000
Gravel	...	40—45	1450—1600
„ wet	...	27	2000
Gravel and clay, wet	...	18	2000
Gravel, sand and clay wet		19	2080
Gravel and sand	...	25—30	1600—1750
Shingle	...	38—40	1450
Shingle and earth, moist	...	40	2200
Vegetable soil, dry	...	30	1450—1600
„ „ moist	...	45—50	1600—1750
„ „ wet	...	15—17	1750—1900
Peat	...	14—45	500
Silt, wet	...	10—20	1750
„ dry	...	20	1750
Rubble stone	...	45	1750

The above figures are approximate and are variable.

Drainage of retaining walls and weepholes. To prevent water pressure behind the wall, drainage should be provided by the use of large material against the back of the wall and by weepholes. Walls retaining soils through which water freely passes, such as clean gravel and sand, should have a drain of loosely packed rubble running along the back footings, from which good-sized weepholes, from 1.8 to 3 m apart, should lead through the base. With more retentive soils, a drain at least 250 to 300 mm wide should run nearly the whole way up the back of the wall. The mouths of the weepholes should always be carefully protected by loose packing. In some cases extra weepholes at higher levels may be advisable, which may be 50 to 75 mm square, or 75 mm pipes may be used at about 2 m intervals (in staggered positions) vertically and horizontally, the lowest being 30 cm from the ground level. Weepholes should be given a fall of 1 in 8 from the back of the masonry to the face. Weepholes should be provided in all abutments and wing walls.

Functions of Soil Angles

ϕ (Degree)	$\cos\phi$	$\sin\phi$	$\frac{1-\sin\phi}{1+\sin\phi}$	$\left(\frac{1-\sin\phi}{1+\sin\phi}\right)^2$	$\cos^2\phi$
10	.985	.174	.704	.496	.970
15	.966	.259	.589	.348	.933
17	.956	.292	.548	.300	.914
20	.940	.342	.490	.240	.833
22	.927	.375	.455	.207	.859
25	.906	.423	.408	.166	.821
27	.891	.454	.376	.141	.794
30	.860	.500	.333	.111	.740
31	.857	.515	.322	.104	.735
32	.848	.530	.308	.094	.720
33	.839	.545	.295	.087	.704
34	.829	.559	.283	.080	.688
35	.819	.574	.271	.073	.671
36	.809	.588	.260	.068	.655
38	.788	.616	.238	.057	.621
40	.766	.643	.218	.048	.588
42	.743	.670	.199	.040	.552
44	.719	.695	.180	.032	.509
45	.707	.707	.174	.030	.500
50	.643	.766	.133	.018	.414

Functions of Slopes

Slope (hor. to ver.)	Angle with horizon	Sine of angle	Length of slope (height taken as 1.0)	$\frac{1-\sin\phi}{1+\sin\phi}$	$\left(\frac{1-\sin\phi}{1+\sin\phi}\right)^2$
1/4 to 1	75°—58′	.970	1.013	.002	—
1/2 to 1	53°—26′	.895	1.118	.055	—
3/4 to 1	53°—8′	.800	1.250	.111	—
1 to 1	45°—0′	.707	1.414	.171	.030
1.25 to 1	38°—40′	.624	1.600	.232	.054
1.33 to 1	36°—53′	.600	1.608	.250	.062
1.5 to 1	33°—42′	.554	1.802	.285	.080
1.75 to 1	29°—44′	.496	2.016	.337	.113
2 to 1	26°—34′	.447	2.236	.382	.144
2.5 to 1	21°—50′	.372	2.689	.485	.211
3 to 1	18°—26′	.316	3.162	.520	.289
3.5 to 1	16°—0′	.275	3.628	.569	.324
4 to 1	14°—2′	.242	4.124	.610	.371

Angle of Inclination Deg.	Slope (ver. to hor.)	Angle of Inclination Deg.	Slope (ver. to hor.)
14	1 in 4	30	1 in 1.7
15	1 in 3.36	32	1 in 1.6
17	1 in 3.27	35	1 in 1.43
20	1 in 2.7	37	1 in 1.33
21	1 in 2.63	40	1 in 1.2
25	1 in 2.15	45	1 in 1.0
27	1 in 1.96	48	1 in 0.9

Failure of retaining walls is generally due to unequal settlement of the foundations, excess of toe pressure, and lack of drainage. Most of the failures are in clayey soils which exert much heavier pressure when saturated as clay swells through absorption of water. Therefore, when deciding upon the angle of repose for a design, due consideration must be given to the effect of rains upon the soil which make the angle of repose flatter. The tendency of the backing to slip is very much increased when the material is saturated with water, therefore, every precaution should be taken to ensure good drainage.

6. PARTITION WALLS

Brick Walls. Where it is desired to keep the thickness of a partition wall 3 ins. or 4.5 ins. (5 cm or 10 cm in case of modular bricks) it should be reinforced with hoop iron 25 mm wide and 18 guage, in courses not more than 30 cm apart and continued for 23 cm into the main wall on which the partition wall abuts, and folded over. If the partition wall exceeds 6 m in length or 4.5 m in height, the hoop iron should be introduced at courses not more than 15 cm apart. The wall should be built in cement mortar and there should be a beam underneath to support its weight. (The hoop iron may be of 12 to 20 gauge (Gauge measures are given at page 4M/17) and can be in two strips.

If hoop iron is exposed to the weather for sometime to remove the bluish smooth surface, its adhesion to the mortar is greatly increased. The bond of the hoop iron with the brickwork is greatly improved if it is punched at intervals of about 15 cm so as to form burrs on both sides of the strips. The punched hole may be 6 mm dia. If a hoop iron strip is not available for the full length, it should be rivet-jointed with an overlap of not less than 75 mm. The joints in brickwork, where the hoop iron is to be laid, should be 25 mm thick to ensure a cover of at least 12 mm of the cement mortar between the reinforcement and the bricks.

When a partition wall has to be built in a location where there is no beam, wall, or other proper support underneath, and it is expected to act as a beam by itself, and to be held up by the side walls, it is necessary to lay most of the reinforcement in the bottom courses and reduce it proportionally towards the top. In such cases the hoop iron should be placed at every course for the first ten courses from the bottom and thereafter at every alternate course.

Half-brick Thick Walls. All courses shall be laid with stretchers. Long lengths of half-brick thick walls shall be provided with reinforcement at every third course with 2 nos. 6 mm dia. bars (prefer 25 × 2 mm wrought iron flats—hoop iron). Half the mortar for the joint shall first be laid and the other half laid after the reinforcement is laid, so that it is fully embedded in the mortar.

Joining Old Brickwork with New Brickwork. The old brickwork shall be toothed to the full width of the new wall and to a depth of quarter of a brick in alternate courses. Thickness of each course of new work is made equal to the thickness of the corresponding course of the old work by adjusting thickness of the horizontal mortar joints. This applies where the bricks used for the old and new work are of the same size. Where, however, the old work consists of 9 ins. size bricks and the new work of modular bricks of 20 × 10 cm size, the old brickwork shall be toothed to the full width of the new wall and to a depth of 5 cm in alternate stages of four courses. Four courses of old brickwork will be inserted to three courses of new brickwork. Old brickwork shall be cleaned of loose mortar, etc., and thoroughly wetted before starting new work.

Lathing and Plaster Partitions

Expanded-Metal Lathing. Expanded metal may be of 20 mm shortway of mesh and of 2.447 kg/sq. m weight available. Lathing is stiffened by 10 mm diameter round iron rods spaced 30 cm apart vertically, which are tied to lathing at intervals of not more than 10 cm by means of 14 to 18 gauge galvanized iron wire. If the lathing is loose the plaster will crack. All metal works should be cleaned of rust before plastering. The undercoat mix recommended for plain XPM lathing is 1:2:9 (by vol.)—cement : lime : sand. A stronger mix can be of 1:1:6. If lime plaster is to be used, it may be of 1.5 parts of clean sharp coarse sand, 1 part of slaked lime and 1/2 part of cement. The mortar should be thoroughly mixed with chopped hemp in the proportion of not less than 5 kg of hemp to 1 cu. m of mortar, or hair should be incorporated in the mortar.

Light gauge metal lathing normally fixed does not constitute a rigid background and for this reason the application of lime mixes more heavily gauge with cement, or use of pure cement/sand mortar is undesirable. Rich mixes develop high early strength and shrink to an appreciable extent during drying. The lime used should be non-hydraulic or semi-hydraulic, quick-lime or hydrated lime. If quick-lime is used it should be run to putty and matured for at least 15 days before use. If dry hydrated lime is used it is preferable to soak it to a putty at least 16 hours before use. (More details are given under "Plastering", and "Mortars and Concretes" in Section 12.)

Brick-nogged partitions consist of frame-work of wooden posts and planking, the interspaces being filled in with brickwork or stone masonry. Posts 150 mm by 130 mm are fixed at central distances of 150 cm. Horizontal pieces 150 mm by 50 mm are fixed 90 cm vertical distances apart. Cross-braces may be fixed between the ribs. Plastering is usually done on both sides.

Hollow blocks of concrete or terra-cotta are used for partition walls. They are sound-proof, light and cheap.

Woodan partitions are given under "Timber Structures."

7. SOUND INSULATION IN BUILDINGS

Siting. Where the windows of bed-rooms and living rooms face a main traffic route or a railway, they should be not less than 30 m to 45 m from its near edge and should be more where possible. Where the windows face at right angles to the direction of the noise, or away from it, the distance may be reduced to about 23 m to 30 m. In the case of local roads the above distances may be reduced to 15 m and 11 m respectively.

Trees also reduce sound to some extent.

Walls. Sound transmission through partitions can be reduced by the following method :—

(*a*) A massive and rigid construction that does not have openings for pipes or ventilators. Sound is transmitted through holes and cracks, and space left due to badly fitting doors and windows.

(*b*) A hard reflecting surface on the outside of the wall.

(*c*) An air gap to prevent continuity of structure (hollow walls).

(*d*) A layer of insulating material. An air space is generally better than a filling material.

(*e*) A non-homogeneous structure containing inert cells.

(*f*) A sound absorbent surface facing the other room such as, fibre boards, hair felt, mineral wool or slag wool. Fibre boards and porous surfaces should not be painted or varnished.

Floors. Sound transmission through floors can be reduced by :—

(*a*) A "floating floor" which is isolated from the walls by inserting a thin strip of felt or some other similar insulation between the skirting and the floor boards. A layer consisting of not less than 5 cm of concrete is poured in-situ upon a rasilient quilt overlying the main supporting structure (bottom concrete under the floors).

(*b*) Provision of sound insulating materials between the floor covering and the floor proper. Coarse cinder fills on floor slabs make an effective deadening pad. Wood, cork, rubber or asphaltic combinations effectively deaden sound.

(*c*) A massive and rigid construction.

(*d*) A hollow floor construction.

(*e*) Insulated and suspended ceiling.

It is possible to reduce the harshness of floor noise by resilient converings such as carpets, cork or linoleum if the materials are reasonably thick.

To prevent echoes the back wall behind the audience should be highly obsorbent of sound, and there should be an absorbent area on the side walls near the platform or stage end to prevent lateral echoes.

Insulating Sanitary Fittings

Water-closets should not be fixed above a living-room or next to a bed-room unless the latter is well insulated, as for instance by cupboards.

The WC pan and cistern should be insulated. The pan should rest upon a thin pad of felt, linoleum, cork, rubber or other suitable resilient material. Cisterns should not be fixed direct to a bed-room wall and should be fixed upon insulators fixed to the brackets. The pipes should be wrapped where they pass through walls or floors and be held in insulated clips.

Sound Insulating Materials

Compressed straw or reeds slabs ; cork slabs ; slag wool ; sponge rubber ; wood shavings ; felt ; bitumen ; asbestos ; breeze bricks. A layer 12 to 25 mm thickness is usually sufficient.

Increased spacing of glass in double windows is particularly useful in improving the insulation at low frequencies, which are important in traffic noise.

8. STAIRCASES

(Wooden and RC stairs have been described in their respective Sections.)

The following terms are generally used :—

Tread — The horizontal upper surface of a step upon which the foot is placed.

Riser—The vertical portion of step.

Nosing—The exposed edge of the tread, usually projecting and rounded.

Rise—The vertical height between the upper surfaces of two successive steps.

Going—The horizontal distance between two riser faces.

Fliers—Steps rectangular in plan.

Winders—Steps tapering (triangular) in plan, used where the direction of the stairs changes. That fitting into a wall angle and which is the central winder of a series, is termed *kite* winder on account of its so formed shape.

Flight — A series of steps between landings.

Landing — A level platform at the top of a flight between floors.

Newels —Posts used at the junction of flights of stairs with landlings or with other flights, or at the foot of a stair.

Curtail Step—The lowest step of a flight usually employed with geometrical stairs.

Balusters —The vertical members between the handrail and strings to stiffen the handrail and prevent persons falling through.

Balustrade — The framed fence formed by strings, handrails and balusters.

Strings or stringers—Term used in wooden stairs. The sloping wooden members like beams on which the ends of steps rest. Two strings are usually provided, one on the outside and another adjacent to the wall. A third is provided in between the two where the steps are wide. The steps are either housed and wedged into the strings or the strings are cut on which the treads are fixed. A string is also called *carriage piece.*

Forms of Staircases

Dog-legged—The succeeding flights run in opposite directions and in plan there is no space between them.

Open-newel—There is a space or opening called "well" between the forward and backward flights, with a half space landing and a newel post at each angle.

Geometrical—In this type of stairs the well is curved between the forward and backward flights.

For the design of stairs average human stride is taken as 60 cm.

$$\text{Pitch}=\frac{\text{Rise}}{\text{Tread}}-\text{Max: 45 deg., Min: 25 deg.}$$

Standard sizes:

Rise = 18 to 20 cm Tread = 23 to 25 cm	for ordinary residential buildings.
Rise = 15 to 18 cm Tread = 28 cm	for public buildings with wide stairs.

Pitch should not be more than 42 deg.; 30 deg. pitch is common.

No staircase in a residential building should have a rise of more than 23 cm and a tread of less than 23 cm. In the case of public buildings (including warehouse and industrial) a rise of not more than 18 cm and a tread of not less than 27 cm is desirable. The wider the tread the less should be the riser and the greater the rise the less should be the tread. Width of the tread in a winding staircase is measured as 45 cm from the inside or the small end of the tread.

Treads usually adopted are 23 cm to 30 cm with risers of 15 cm to 19 cm. For concrete steps, normally adopted tread width is 27 cm with a riser height of 18 cm. The following proportions may be taken :—

Tread—cm	23	25	28	30	33	35	38
Riser —cm	19	17	16	15	14	12	11

Winders should not be used if they can be avoided but if they are necessary they should be at least 23 cm in width at about 41 cm away from the handrail, and located near the bottom and not the top of the stairs. Only three winders should be used in a quarter space landing.

Height of flights for public buildings should be limited to 2.4 m or 12 steps (max. 15) without landings or turns. Greater heights should only be reached by flat pitches such as 28 cm × 15 cm ; 30 cm × 14 cm.

The min. clear head-room should be 2.1 m measured from the top of the riser to the ceiling.

The treads should be level throughout and made with rough surface to reduce slipping.

Lobbies, Corridors, Landings and Passages

Minimum width for a residential building ... 80 cm

" " public building ... 140 cm

In a public building if the number of users is more than 20 and less than 100, the min: width should be 1.83 m and, for number of users above 100, it should be 2.28 m. The width should not, however, be less than the width of the stairs. See also under "Miscellaneous Technical information."

The imposed loads to be allowed on stairs and landings are given in the Section on "Reinforced Concrete".

Passage for Staircases. When serving more than one staircases its minimum width should be equal to the width of such staircases plus half of the total width of the remaining staircases.

Width of Staircases

Minimum clear width of a staircase for a single family residential house is 75 cm which should be 95 cm if the number of users is 10. A building intended to be used by two families, or a commercial building, shall have a staircase of the following minimum width :—

Number of users up to 10	...	...	110 cm
" " from	11 to 20	...	125 cm
" " from	21 to 100	...	140 cm

Stairs should be at least 110 cm wide to allow two persons to pass easily.

For a public, warehouse, or an industrial building :—

Number of users up to 200	...	...	155 cm
" " from	200 to 350	...	185 cm

Increase by 25 mm for every additional 15 persons until a maximum of 2.75 m is reached.

A single staircase of the width mentioned above may be replaced by two staircases each of a width at least equal to 2/3rd the width prescribed for a single staircase provided neither of the two substituted staircases be less than the min. width prescribed. Where the number of users exceeds 300 it is preferable to provide two or more staircases.

Stone Steps. Each stone should rest at least 40 mm on that below it. For steps which have one end free cantilevered, the length fixed in the wall should be half the wall thickness with a minimum of 23 cm. Steps supported at both ends should rest at least 15 cm on the walls at either end, which may be 115 mm min. for 90 cm wide stairs and should be increased up to 23 cm for wider stairs. The bottom step is bedded on the floor and the lower front edge of each other step rests on the upper back edge of the step below. Each step may simply rest upon the one below it but it is better for the upper step to be rebated over the back of the one below to prevent sliding.

A Ladder should have a min. width of 45 cm for access to a terrace.

Handrails, Parapets and Balustrades

The height of railings is usually 75 cm to 90 cm. If the railing is composed of balustrades, the spacing between them should not be more than 75 cm.

Parapets and balustrades of staircases in places of assembly, where danger would result in the event of panic, should be designed for horizontal static load of 300 kg/metre run acting at handrail level. (See also at page 6/50.)

Dowels, Cramps, Joggles, etc.

Cramps may be of copper or lead, 50 mm to 150 mm long, 16 mm to 25 mm thick and 25 mm to 50 mm wide, having each end turned at right angle. Copper cramps are forged and set with neat cement, lead cramps are formed by running molten lead into the dovetail channels. Joggles and dowels should be of double wedge form and made from copper or from slate or similar stone and set in neat cement. No iron cramps, joggles, dowels, whether galvanized or otherwise, should be used. (Large stone landings which cannot be obtained out of one piece of stone are joggled at their ends.)

9. CEILINGS

Plaster on wire netting or metal lathing. The netting to be of galvanised wire of 12 mm mesh No. 20 gauge (0.91 mm) fixed to wooden battens or rafters at not more than 45 cm centres by means of 40 mm wire nails to be spaced at intervals not exceeding 15 cm. Where netting passes over timber or iron framing, a space of 12 mm should be left by blocking out to permit room for plaster key. After stretching, the whole surface of the netting should be brushed over with a thin mixture of cement slurry.

Expanded metal lathing may be of 10×40 mm (SWM×LWM) and of light weight.

Cloth Ceiling is fixed on frames not exceeding 1.5×1.5 m in size with 10 mm×40 mm wooden fillets. The cloth should be first damped, stretched over frames and fastened with tacks on the outside. A cost of whitening consisting of chalk and glue is given over the cloth. On no account shall whitewash be used as it rots the cloth.

10. CHIMNEYS & FIRE-PLACES

Brick Chimneys for Factories

Brick chimney shafts should be constructed throughout with bricks and mortar of the best quality ; should taper uniformly from base to top at the rate of not less than 1 in 40. Circular form is considered to be the best and most stable. The flue must be circular. A circular chimney should not exceed 25 times its internal diameter in height. The thickness of the enclosing brickwork at the top of a chimney shaft and for 7.5 m below the top should be at least 25 cm and should be increased to one-half brick for every additional 6 m or part thereof measured downwards. If the inside diameter at the top exceeds 1.4 m the top length should be 1.5 bricks thick instead of 1 brick thick. The width of the shaft at its

base should be at least one-tenth of the height for square shafts, and one-twelfth of the height for circular shafts. Circular steel reinforcing hoops may be provided not less than 10 mm× 65 mm size, built into the brick-work at each change of wall thickness, and just above and below the flue openings.

Caps tie head of chimney together. The footings should spread all round the base by regular offsets to a projection equal to the thickness of the enclosing brickwork at the base and the space enclosed by the footings should be filled in solid as the work proceeds. Scaffolding used for building a chimney should be so arranged that it does not prevent the chimney from setting.

A chimney shaft should be provided with independent lining of fire-bricks 10 cm thick and separated from the masonry enclosing the shaft by a cavity at least 50 mm (prefer 80 mm) and the cavity should be covered at the top with corbelled brickwork.

Chemical works are usually required to have their chimneys at least 76 m high to ensure that fumes are discharged well above the inhabited area. Power station chimneys should have a height not less than 2.5 times that of the highest point of the station roof or adjacent buildings.

Chimney stacks or smoke flues should be carried up at least 1 m above flat roofs and not less than 0.6 m above any ridge of a pent roof.

Steel Chimneys are usually cylindrical in shape with a wide curved flare at the bottom. A heavy base plate is provided to which the chimney is riveted and the plate is secured to the foundation by holding down bolts. A steel chimne· 1 m dia. and 20 m high requires a 5 mm thick steel.

Wind pressure for calculating stresses in chimneys and tall structures has been given in Section 11.

11. BUILDING BYE-LAWS

(Based on IS Code of Building Bye-laws: 1256)

Size of Rooms

Minimum size of Habitable (or Living) Rooms:

Area –9.5 sq. metres with a minimum width of 2.4 metres.

Height: Not less than 2.75 metres, with minimum head-room at any point in the room of 2.4 m. Maximum useful height considered is 3 m to 3.6 m, but a height of less than 3.6 m does not make a good residential room. Beyond a certain point increasing the height of a room in preference to floor area would not be of much use as regards ventilation. Lofty rooms are cooler.

Kitchen : Minimum floor area to be 5.6 sq. m which shall not be less than 1.8 m in width at any part. Where there is a separate store, the floor area of the kitchen may be reduced to 4.8 sq. m. A kitchen which is intended for use as a dining room also, shall have a floor area of not less than 9.5 sq. m with minimum width of 2.4 m.

Height not less than 2.75 m.

Both-room and WC : The size of a both-room shall not be less than 1.5×1.2 m or 1.8 sq. m. If it is combined bath and WC, the minimum floor area shall be 2.8 sq. m. The minimum floor area of an independent WC shall be 1.1 sq. m. Height 2.4 m minimum. The floor level of a toilet should be kept 15 cm lower than the rest.

Height of a cellar or basement, mezzanine floor, store, gallery or verandah shall in no part be less than 2.3 m. A passage under a landing shall have a minimnm head-room of 2.1 m.

Window Area for Dwellings

Every habitable room shall have one or more apertures such as windows, fan lights, opening directly to the external air or into an open verandah of an aggregate area, inclusive of frames, of not less than 1/10th of the floor area excluding doors for dry hot climates, and 1/6th of floor area for wet hot climates. Ventilators may be provided at the rate of 1400 sq. cm every 14 cu. m capacity of such rooms. The minimum area of opening for ventilation should be 2700 sq. cm. The ventilators should be fixed as high as possible under the ceilings. Window sills are 77 cm above the room floor level.

Every kitchen shall be ventilated according to the standards prescribed for habitable rooms near the ceilings as far as possible.

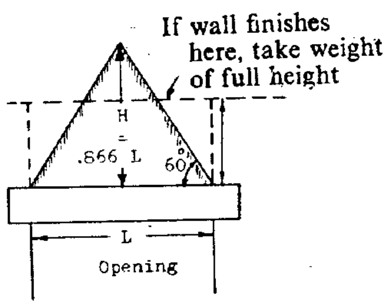
Bath-rooms and WCs shall have windows of an area of not less than 10 per cent of the floor area and located in an exterior wall.

Basements and cellars shall have windows opening in exterior walls of area of not less than 2.5 per cent of the floor area.

Factories and Warehouse Buildings. Every room should have doors with clear opening not less than 1/15th of the floor area, abutting on open air space, of width not less than 1/3rd the height of the part of the building abutting such open space.

12. LINTELS

The loads coming on a lintel are uncertain and are considered to depend on the "arching" action of the masonry above. If the height of the wall above the lintel is greater than 0.866 L, the weight on the lintel is usually assumed as that of an equilateral triangle on base L. (Some engineers take L as effective span instead of clear span). This total weight is considered as uniformly distributed load over the span, and lintel designed with BM = WL/6.



$W = 0.433\ L^2 \times$ thickness of wall $\times$ weight of one cu. m of masonry.

This method applies when the masonry above the lintel is well bonded and the walls on both sides of the opening are not less than L/2. When, however, one end of lintel is close to the end of wall (less than L/2), weight of the wall above the lintel to be taken for design should be L^2 instead of $0.433\ L^2$. When walls on both sides of the opening are less than L/2, full height of the wall above the effective span should be taken. In case of uncoursed rubble walls or walls in mud, it is safer to take the span as uniformly loaded with height of wall equal to 1.5 times the span.

There is no "arching" action when the mortar is still green. The bricks in fact form a composite beam of a much greater depth than would a supporting beam. The lintel should be kept propped up during bricklaying till the mortar has fully hardened (for at least 7 days).

Typical cases :

(*a*) If the height of the wall above the lintel is less than H (0.866 L), take full height of the wall plus any other load on the wall.

(*b*) Should another opening occur in the triangle thus formed on the lintel top, the load on the lower lintel will be that portion of the triangle not covered by the opening plus the triangular load on the upper opening.

(*c*) With a concentrated load falling inside the Δ, take weight of the Δ as explained before plus the load concentrated over the span.

$$BM = \frac{WL}{6} + \frac{W_1 L}{4} \qquad W_1 \text{ is the concentrated load.}$$

Some engineers assume the load distribution at 60° from a concentrated load occuring inside or near the load triangle.

(*d*) With a concentrated load above the Δ, add the weight from the concentrated load to the weight of the Δ and consider the whole as uniformly distributed as before unless the concentrated load is more than L above the apex of the Δ in which case it may be neglected.

Uniformly distributed floor loads above the equilateral triangle are distributed, but such loads falling within the triangle are considered by taking into account only that stretch lying inside the triangle.

Deflection of a lintel should not exceed 1/480 of the span and overall depth should not be less than 1/20 of span.

Stone Lintels for small openings

For 150 cm opening it will be 15 cm.

Thickness = 25 mm for each 30 cm span + 25 mm.

The increase in weight of a stone slab after 24 hours soaking in water shall not be more than 10 per cent of the dry weight.

RC Lintels

For small spans and ordinary loads, take 150 mm overall depth for spans up to 120 cm and add 25 mm for each 30 cm span.

Reinforcement

For thickness of wall 10 cm (1/2 brick) take 2 bars ; for 20 cm (1 brick) take 3 bars, and for 30 cm (1.5 bricks) take 4 bars ;

6 mm for spans under 1 metre

8 mm for spans 1 to 1.5 metres

10 mm for spans 1.5 to 2.0 metres

12 mm for spans 2.0 to 3.0 metres

All bars should be hooked and alternate (central) bars bent up. It is advisable to provide two bars of 6 mm or 10 mm diameter at the top to assist in assembling the reinforcement cage and to bind all with 6 mm dia. stirrups at 150 to 300 mm centres. Pre-cast lintels may be used up to 1.22 m span.

For stone walls the depth of the lintel should be increased by about 10 per cent and rods by about 25 per cent.

Rods should be bent at a distance of between L/5 to L/4 from the edges of the opening to an angle between 30 to 45 degrees. (See under "Reinforced Concrete.")

Bearings of lintels over walls should not be less than the overall depth of the lintel with minimum of 150 mm for brick or coursed rubble walls and 23 cm for random rubble walls.

Addition of some light reinforcement in the brick panels will be of value in strengthening the wall against the effects of unequal settlements and shrinkage.

13. STONE CHHAJAS (SUN-SHADES)

RCC Sun-shades are given in Section 8

For a sloping chajja, a minimum bearing of 20 cm in the wall should be provided. Each slab should be anchored down by means of a holding down bolt 12 mm in diameter and 450 mm long, the lower end being bent at right angles for fixing into the masonry joint. The bolt should pass through the hole drilled in the centre of the bearing of the stone slab. The holding down bolt should be secured at top by suitable washer and nut. In the case of an horizontal chajja, the stone slab should be fixed with a slight outer slope of 1 : 20. A minimum bearing of 20 cm in the wall with a holding down bolts should be provided.

14. SITING OF BUILDINGS, ORIENTATION AND VENTILATION

Orientation of Buildings

Heat and humidity are the two controlling factors in the design of a dwelling. Indian climate is classified for design purposes either hot-arid or hot-humid. Hot-arid climate is characterized by the high summer day-time temperatures, low relative humidity, wide diurnal temperature variation. Characteristics of the hot humid climate are low summer day-time temperatures, high relative humidity and low diurnal temperature variation. Orientation of a house has a great influence on the comfort conditions indoors.

The following orientations are suggested according to the prevailing monsoon winds :—

Hot dry areas :

Northern India — Orient along E and W, facing N.
Central India — Orient along E-SE and W-NW, facing N-NE.

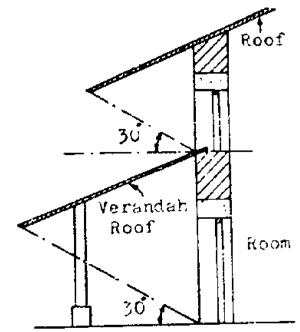
For orientation in Delhi the best position of a building is considered when its longer side makes an angle of 22.5 deg. on the East-West line towards East-South.

Hot humid areas :

West coast regions — Orient along SE and NW, facing SW.
East coast regions — Orient along SE and NW, facing NW.
Bengal — Orient along E and W, facing S.

(Based on the results of the experiments conducted at the Central Building Research Institute, Roorkee.)

In hot climates living rooms on the south and west sides should be protected by verandahs, baths, and stores, etc. But in hill stations living rooms are generally open on the south and west sides for the sun. In long buildings such as hospitals, schools, one of the long sides should face north and south and west protected by verandahs. Drawing offices and dark rooms should be located on the north side. Sunshades need be provided only on the south and west sides. Eastern or north-eastern corner is the best for kitchens. Kitchens should have cross ventilation.



GUARDING AGAINST SUN RAYS

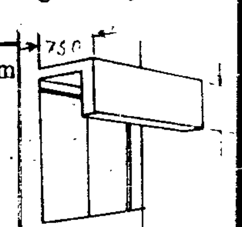
For bed rooms at least one wall must be on the outside for good ventilation and the room should be placed in the direction of the prevailing wind. Latrines or WCs should be so located that the wind passing through them should blow in a direction away from the house.

For proper ventilation (air and light) the height of a house (from plinth to top) should not be more than twice the width of the street. This is called 63.5 deg. rule,

Temperature Variations in Structures. In India the maximum daily range of shade temperature is about 17 deg. C., and the yearly range about 45 deg. C., and in the sun, about 44 deg. C., and 72 deg. C. respectively. The normal average temperatures may be taken as 21 deg. C. in the plains and 15 deg. C. in hill stations.

Expansion and contraction due to temperature changes may be provided for in various structures as follows : + −

Steel exposed to the sun	... ...	12 mm	in 30 m
Steel shaded from the sun	... ...	8	,, ,,
Masonry and concrete exposed to the sun		8	,, ,,
Masonry and concrete shaded from the sun		5	,, ,,
Structures in contact with water	...	5	,, ,,



Sun-shade

In hill and coast stations the variations in the temperature are about 3/4th of the above.

15. PROTECTION AGAINST WHITE-ANTS or ANTI-TERMITE TREATMENT

Damp proof courses of rich and dense cement concrete (1:2:4 or richer) and cement concrete floors give good protection against white-ants. The following methods also offer protection :

(*i*) A solution of 1/2 kg of copper sulphate in 20 litres of water in the mortar. The solution also protects wood, takes two days of steeping per 25 mm of wood thickness.

(*ii*) Yellow arsenic in mortars in the proportions of : Concrete—6.5 kg, masonry and plaster—0.8 kg, per 10 cu. m.

(*iii*) A layer of about 75 mm graded washed sand or sand and cinders over which a sodium arsenate solution is spread.

(Arsenic being a poison should not be used on surface works).

C.P.W.D. and IS : 6313 give detailed specifications for treating the soil beneath the building and around the foundations with certain insecticides which are sprayed in solutions with hand operated pressure pumps.

Extermination of Termites in Buildings

For a site infested with white-ants, the following method is recommended in the Madras P.W.D. Specifications :—

"The whole area proposed to be occupied by the building together with an extra width of 3 m all round shall be excavated to a depth of 15 cm and soaked with water. Spreading wet straw over the area brings the ants to the surface. If a white-ant's nest exists on site, its presence will become evident in a few days, whereupon the nest should be completely dug out, the queen ant destroyed, and the nest flooded with boiling hot water containing a solution of arsenic."

Recourse should be taken to inject oil/kerosene base solvents into termite channels in woodwork and masonry followed by sealing openings and spraying the liquid.

Termites generally do not penetrate masonry or concrete of first class construction in which there are no voids. Masonry with lime mortar of mix leaner than 1 : 3 shall not be used to be in contact with soils where a concrete floor has not been laid. If the floor construction has vertical joints between the floor and the plinth masonry, these joints should be filled with heavy grade coal tar pitch to minimise the infilterate of termites through these joint.

16. PLASTERING & POINTING

Surface Preparation. Surfaces to be rendered must be clean and free from all dust, loose material, grease, etc., and be well wetted for a few hours (the walls should not be soaked but only damped evenly) ; but the walls should not be too wet as plastering on wet walls is seldom satisfactory. A good "key" is essential for a successful rendering and for avoiding cracking and crazing. All joints in the masonry should be raked out to a depth of at least 12 mm with a hooked tool made for the purpose whilst the mortar is still green, and not later than 48 hours of the time of

laying. Joints should not be raked out with a trowel or a hammer as the edges of the bricks get chipped. The brickwork should be brushed down with a stiff wire brush so as to remove all loose dust from the joints, and thoroughly washed with water. On old walls it may sometimes be advisable, to ensure a good key for the new rendering, to destroy the smooth surface of the brickwork with some tool. If the walls are washed over with a solution of one part hydrochloric acid to ten parts water, it has the effect of bringing the grains in the brickwork to the surface. The acid solution is left on for about a quarter of an hour and afterwards washed off very thoroughly with water.

Plaster may be applied in one, two or three coats ; two coats are usually sufficient. Three coats would be used only on wood or metal lathing or on a very rough uneven background. The thickness of the first coat should be just sufficient to fill up all unevennesses in the surface. No single coat should exceed 12 mm in thickness ; lower coats should be thicker than upper coats. Thick coats shrink more and crack. Under coats of coarse stuff should be allowed to dry for 3 to 5 days and shrink properly before subsequent coats are applied. The surface should not be allowed to dry during this period. Following up with finishing coats too soon is a common cause of cracking and crazing. A good key for all stages of plastering is essential. When applying another coat of plaster, the previous plastered surface should be scratched or roughened before it is fully hardened to form a mechanical key. The method of application of the mix influences the adhesion ; if thrown on, the mix will stick better than if applied by trowel.

All plasters shrink on drying. Much fine material makes for high shrinkage ; it may also interfere with the setting of cement plaster. Fine sand is often recommended for plastering but it should not be so fine as to pass more than 5 per cent through a 150 micron sieve or more than 20 per cent through a 300 micron sieve. Rich mixes tend to develop a few large cracks ; weaker mixes, finer and distributed cracks. A strong coat should not be applied over a weaker one which would be unable to restrain its movements.

Plaster work on new construction should be deffered as much as possible so as to let shrikage in RC and masonry take place before plastering.

Lime plaster. The lime used for plastering ranges from fat lime to strong hydraulic lime. Fat lime is most commonly used. Hydrated lime is generally preferred to the lime which has to be slaked as the hydrated lime can be used atonce whereas the ordinary slaked lime must be kept in the form of a putty until slaking is quite complete ; but fresh slaked lime is better as regards quality. Coarse sharp sand should be used with lime. Lime and sand plaster is weak and soft and takes a long time to harden. Fine stuff for finishing coats is made by mixing water with a thoroughly slaked lime to bring it to the consistency of cream ; it is then left to settle, the superfluous water poured off and the water evaporated. An equal volume of fine sand is then added.

Lime mortars should preferably be gauged with cement. Cement-lime plaster should be used within two hours after the addition of water to cement provided it is kept agitated or turned over at intervals of at least 20 minutes. Surkhi is also added with a lime mortar. A lime plaster should not be finished very smooth on walls which have subsequently to be white-washed as white-wash will not stick to a smooth surface. Proportions of mortars have been given earlier under "Expanded Metal Lathing and Plaster Partitions" and under "Mortars and Concretes" in Section 12. The plastered surface must be kept wet for several days to prevent cracking.

Cement mortars should preferably be gauged with fat lime. Cement/lime/sand mortar hardens slowly and reduces cracks ; adding of lime also gives easier working. Sand used should not be very fine ; if sands of uniform particle size are used in a cement/lime mix, the mix needs an excess of water and this may result in low strength and high shrinkage. (See under "Cement/Lime Mortars" described at page 7/21) and "Mortars and Concretes" in Section 12.) The plastered surface must be watered for several days to prevent cracking. All plaster-work should be dried slowly avoiding draughts and exposure to excessive heat and sunlight. Over-rapid drying produces cracks.

To ensure even thickness and true surface, patches of plaster about 15 cm × 15 cm should be first applied, horizontally and vertically, not more than 2-metre intervals over the entire surface or, wooden screeds 7.5 cm wide and of the thickness of the plaster may be fixed vertically about 1.8 to 3 m apart to act as gauges.

Trowels for plastering have face measuring about 25 × 10 cm. Wooden trowels produce a sandy grannular surface.

Floating Coat with Neat Cement Slurry. The quantity of cement used for floating coat is 1 kg per sq. m.

Defects in Plasterwork. (1) *Cracks* are chiefly due to : (*i*) Structural defects in building and discontinuity of surface ; (*ii*) Plstering on very wet back-ground ; (*iii*) Old surface not being properly prepared ; (*iv*) Over-rapid drying ; (*v*) Excessive shrinkage of the plaster due to thick coats. (2) *Pitting and Blowing* are due to faulty slaking and hydration of the lime particles in the plaster. (3) *Falling out* of plaster is chiefly due to : (*i*) Lack of adhesion for not having formed a proper "key" in the back-ground ; (*ii*) Excessive moisture in back-ground ; (*iii*) Excessive thermal changes either in back-ground or plaster ; (*iv*) Rapid drying ; (*v*) Insufficient drying between each coat of plaster.

Repairing Cracks in Plaster. Hair cracks in plasters will generally disappear with white-washing. Wider cracks can be filled in by forcing down a mortar consisting of 1:2:7 by weight of plaster of paris, cement and sand. Only that much quantity should be mixed with water which can be used up in half-an-hour's time.

Architectural Plaster Finishes

(*a*) *Rough cast.* A wet plastic mix of 3 parts cement, 1 part lime, 6 parts sand, and 4 parts of 6 mm to 12 mm shingle or crushed stone, which is thrown on to the wall by means of a scoop or plasterer's trowel.

(*b*) *Pebble-dash.* A 10 mm coat of 1 part cement, 1 part lime and 5 parts sand upon which, while it is still soft, is thrown 6 mm to 12 mm shingle.

(*c*) *Ornamental finishes.* A mix of approximately 1 part cement, 1.5 parts lime and 6 parts sand which after application is figured by the use of combs, trowels or special tools.

Plastering on Lathing. Lathing forms a convenient base in some forms of construction for plastering on walls and ceilings. Metal lathing is most commonly used which is fixed to timber supports by galvanized nails or staples at short distances. It is also often used to bridge the junction of two dissimilar backgrounds, or to provide a suitable key for plastering over a wooden beam. Wire netting for metal lathing should be galvanised wire 0.90 mm dia. and of 12.5 mesh. Expanded metal can also be used weighing not less than 1.5 kg per sq m. Lathing must be stretched tight with the help of some tension devices such as mild steel rods as plaster will crack on a loose lathing. Cement slurry should be brushed over the lathing after rust has been cleaned or given a protective coat of bitumen oil paint. Most common defects found in plaster on metal lathing are extensive cracking, particularly along the line of fixing of the lathing to its supports, or of unevenness of the finished plaster surface. Plastering on expanded metal lathing has been described earlier under "Partition Walls."

Wooden laths are sometimes used for architectural works such as pattern staining. Laths must be of well seasoned wood. Such laths consist of strips of wood of size 90 to 120 cm long, and 25 mm × 10 mm to 20 mm in section. Wooden laths should be thoroughly wetted before plaster is applied.

Lime punning or Neeru finish. Neeru is obtained by mixing lime putty and sand in equal proportions and chopped jute @ 4 kg per cu. m of mortar. The mixture is ground to a fine paste. It is used as a thin coat of 3 mm thickness. Before actual use, putty should be matured for 2 to 3 days.

Water-proof Mud Plaster for Walls. Just a couple of hours before mud plaster is desired to be done, 60 kg of bitumen cut back per cu. m of soil should be added. Mud plaster is applied 12 mm thick over a clean damp surface and rubbed smooth with a trowel of a wooden float. Occasionally sprinkling of water in hot weather is necessary, to avoid cracks, while the plaster is drying. After partial drying of the plaster the surface is finished with a gobri leap (cowdung) to fill up the cracks which might have appeared on drying. Gobri leap is prepared by mixing equal quantity of soil (clay), free from coarse sand, to cowdung 100 kg of bitumen cut back per cu. m of soil is added before applying the leap.

A method of water-proofing existing mud houses is—a water-proofing material based on asphalt is sprayed over the walls with the help of an ordinary hand sprayer, which gets absorbed in the mud walls. Two or three sprayings are usually sufficient. This makes the walls water proof and weather resistance. The dark brown water-proofed mud walls can be white-washed by adding a little adhesive in the lime wash. The walls so water-proofed have withstood a number of heavy showers.

Chopped straw (bhoosa) for mixing with mud mortar for plastering should be clean, thin fibres not longer than 20 mm.

Soil for making mud mortar should be of tenacious nature, free from vegetable roots, stone gravel, kankar, and efflorescent salts. Soil shall not be collected from localities afflicted by white ants. (See also under Mud Plaster in Section 20 "Estimating"). After mixing water, and after a lapse of 12 hours the mortar shall be trodden with men's feet and spades to make it a homogenous mass.

POINTING

Pointing should be done whilst the mortar in the joints is still green. The surface of the work should be prepared as explained under "Plastering." When commencing masonry work each day, the first thing to be done if the surface is to be subsequently pointed, is to rake out the face joints of all masonry which was finished on the previous day.

Hor. Joint

Ver. Joint

The joints must be well wetted in old work before pointing as the mortar will not stick on to a dry surface. The work pointed should be kept wet for at least three days.

There are about half a dozen types of pointing but most common are the flush, weathered and grooved or ruled. Weathered pointing is used for horizontal joints and grooved for vertical joints of walls ; flush pointing is used for all vertical and horizontal joints in walls which are subsequently to be white-washed, and also for floors.

Mortars for pointing have been given under "Mortars and Concretes" in Section 12. See also under "Cement/Lime Mortars" described earlier in this Section. Mixes of the composition of 1 cement, 3 lime, 10 sand by volume or even 1:4:16 mixes have been used successfully for bedding and pointing but mixes of 1:2:9 or 1:1:6 should be preferred. Mortar must be well pressed into the joints.

17. WHITE-WASHING & COLOUR-WASHING

(See also Section 12)

The white-wash shall be made from pure fat lime or shell lime. The lime shall be brought to the site in an unslaked condition and thoroughly slaked on the spot, mixed and stirred with sufficient water to make a thin cream and allowed to stand for 24 hours. If 4 grams of gum (or shellac) and 50 grams of common salt dissolved in hot water, are added to one kg of limestone for the last coat, the white-wash will not easily rub off. Indigo (neel) up to 3 grams per kg of lime dissolved in water, is added and the wash stirred well. Before any lime-wash or colour-wash is applied to a surface, it is essential that all loose material and dirt shall be removed with a brush or by rubbing with an old gunny bag. Lime putty is used to make

good all holes and irregularities of surface or minor repairs, which should be let to dry for 24 hours before white-washing. All greasy spots should be rubbed and given a coat of rice water and sand ; and surfaces discoloured by smoke given a wash of a mixture of wood ashes and water or yellow earth, before the application of the white-wash. *Lime putty* is obtained by slaking lime with water and sifting it.

The lime-wash should be strained through a coarse cloth or sieved through a fine wire gauze before applying. The coats are given alternatively vertically and horizontally. One stroke is given from the top downwards and the other from the bottom upwards over the first stroke and similarly, one stroke from the right and another from the left over the first brush before it dries. Each coat should be let to dry before applying the next coat.

White-washing Cement Concrete. Wash the concrete surface with soap suds, scrape off all grease with a wire brush, give one coat of sodium silicate and water 1:5. Allow to dry, apply white-wash. There will be no scaling or flaking off after this treatment. Cement paints have been described elsewhere.

An excellent white-wash that will adhere to stone, iron or glass may be made by scattering 1/2 to 1 part by weight of tallow in small lumps over 16 parts of quicklime, slaking it with only just sufficient water to form a thick paste, stirring occasionally to assist in dispersing the tallow, and allowing it to stand until cool. The resultant paste should then be let down to a thin wash, which is strained and applied in the usual manner. If tallow cannot be obtained some other oils or fats, *e.g.* linseed oil, or castor oil can be used ; prefer some common vegetable oil and mix about 10 per cent by weight of dry lime. If the oil does not sponify and incorporate with the lime, it should be boiled a little until the oil disappears. The oil forms with the lime an insoluble soap, which when once dry, will not wash off with heavy rain.

If white-washing is to be done on cement plastered walls, add some soap in the wash

White-washing with chalk whiting. 2.5 kg of whiting and one litre of double size prepared from 0.25 kg glue should be mixed together and placed in a covered vessel with enough water to cover the mixture which should be left to cool until it becomes a jelly.

Cement wash with boiled linseed oil on iron work. (See also at page 12/40). Cement and boiled linseed oil are mixed together to the consistency of cream in the proportions : Cement 6 kg, double boiled linseed oil—5 litres.

Colour-Washing

Mineral colours, not affected by lime, shall be added to white-wash. Indigo (neel) shall, however, not be added. For new work, the priming coat shall be of white wash with lime or with whiting. For old work, after the surface has been prepared, a coat of colour wash is applied over the patches and repairs. Then a single coat, or two or more coats of colour wash are applied. The finished dry surface shall not be powdery and shall not readily come off on the hand when rubbed.

In replacing one colour with another a coat of white-wash should be given or the old paint scraped off, before the new colour is given. Gum or rice water should be added as for white-washing.

Distempering

Distempers form a cheap, durable and easily applied decoration for internal use on plastered, cement concrete and various wall-board surfaces.

New plaster should be allowed to dry for at least two months before any treatment is attempted. New lime plastered surfaces should be washed with a solution of 1 vinegar to 12 of water, or 1 to 50 sulphuric acid solution and left for 24 hours, after which the walls should be thoroughly washed with clean water. If the plaster contains cement the surface should be washed over with a solution of 1 kg of zinc sulphate in 10 litres of water and then allowed to dry ; before distempering it should be wiped with clean cloth to remove any efflorescence. Pitting in plaster due to removal of nails or other fixtures can be made good with plaster of paris mixed with dry distemper of the colour to be used. Distempers give poor results in wet locations ; should be applied in dry weather.

Troubles with distemper are most often due not to the material itself but to its use on surfaces that have been insufficiently or incorrectly prepared. All loose and flaking material should be removed from old walls by scraping or wire brushing. Any uneveness should be made good by applying putty made of plaster of paris mixed with water and then sand papering the same after it is dry. To get good results it is necessary to apply a priming coat (as recommended by the manufacturers). It often happens that fresh coats of distemper pull off the old coats, absorb water from the new distemper, the adhesion becomes reduced, and as the new coats contact in drying out, they tend to pull off the old distemper. On new lime plastered walls distempers should be applied in two coats over one coat of priming. On old lime-plastered walls covered with one or two coats of hard dry white-wash, one coat of distemper without priming can be used, but a coating of warm glue is useful. Distempers grow dark with age. Slight stains on distempered walls can sometimes be removed by a soft wet cloth.

Distempers are : oil-bound washable paints, washable oil-free distempers, non-washable distempers or emulsion paints. Distempers should be fast to rubbing with fingers.

For applying *oil bound* distempers on already dry distempered surfaces, the distemper, whether in good or bad condition, shall be removed completely by washing and allowed to dry fully and then sand papered smooth before applying the oil bound distemper. Oil bound distemper is not recommended to be applied within six months of the completion of wall plaster.

Distemper should not be mixed in a larger quantity than is actually required for a day's work and should be kept well stirred and applied with

proper distemper brushes (and not with white-wash brushes). In applying the distemper the brush should first be applied horizontally and then immediately crossed off perpendicularly. Brushing should not be continued too long as the distemper becomes sticky and brush marks results. After each day's work the brushes should be washed in hot water. Hot water should be used in preference to cold water in preparing a distemper. (See also under "Painting".)

Colour Scheme for Walls

There are only three primary colours—red, yellow and blue. All other colours are mixtures of these three. There are warm colours and cool colours. Red, orange and yellow are warm ; violet, blue and green are cool. Excess of red or orange is upsetting, most uncomfortable to live with in a hot climate. The coolness of blues and greens recall water, ice, snow and trees. White, pure greys and black are neutral.

Colour terms : A tint is a colour plus white ; a shade is a colour plus black ; a hue is a colour plus a little of another colour that is close to it on the colour wheel. Tone means the intensity of colour, value refers to its lightness or darkness. Dark values are called shades, light values are tints.

It is rare to find exactly the right colour on a colour card and it will nearly always be too strong. Another difficulty is that as the colour dries, it alters and it is therefore impossible to be sure that the colour is right at the moment it is put up on the wall as a sample. It also changes in electric light. So the safest is to put up a patch (actually two or three is best), let it dry and then observe it in artificial light as well.

White is popular in contemporary homes. Broad expanses of white make a room look larger. Used for ceiling and woodwork, white gives a crisp finish to a scheme and looks light and fresh. The most difficult thing to achieve is a dead white that remains white. Nearly all whites become cream—a disadvantage. If you look behind the pictures in almost any white room some months after it has been painted, you will see that the original colour was far whiter. To get a successful lasting white, you must use a paint pigmented with titanium oxide.

Cool Colours :

(1) Yellow Green	(2) Green	(3) Blue Green
(4) Primary Blue	(5) Blue Violet	(6) Violet

Warm Colours :

(1) Red Violet	(2) Primary Red	(3) Red Orange
(4) Orange	(5) Yellow Orange	(6) Primary Yellow

Blue is usually a difficult colour. In a blue room, the walls reflect strongly upon each other. If you want a pale blue room, the colour pattern should look almost colourless. Pink becomes paler with time. In a few months' time it will become too pale.

If you want your ceiling and walls to be the same colour, the ceiling should be painted several tones paler than the walls. If they are painted the same colour, the ceiling being in shadow will look darker than the walls.

Grey is essentially a cool colour. It is an excellent background colour. It makes almost every other colour more vibrant when used in combination ; white and black become starkly brilliant and bright tones glow. Green is a very good background colour too. A green that has quite a bit of blue in it will cool off a too bright room and, conversely, a green that leans towards the yellow side of the colour wheel will warm a cold looking room. Pastels and pale tints make a room look larger.

But no colour is absolute. Colour changes as soon as another colour is next to it. This has to be taken into account when planning the walls. The right colour can make a room look larger. Pale colour reflects light instead of absorbing it, these tones take full advantage of natural light and make artificial light more effective For instance, a room with small windows and little light will look much larger and brighter if planned around white with touches of sunny yellow.

A long narrow room will take on much better proportions if the two narrow walls are painted darker than the wider ones. Very often an attractive patterned wall-paper is used for this purpose.

A square boxy room can take on a better shape by painting one wall in a bold tone, the other three in pastel of the same colour.

18. STABILIZED SOIL FOR BUILDING CONSTRUCTION

(See also "Soil Mechanics")

Soil-cement

The addition of cement to a soil improves its compression strength and also makes the soil highly resistant to the softening action of water, which is thus made stronger and more durable than the untreated soil. Percentage of cement required depends upon the soil characteristics. Sandy soils can be stabilized by the addition of 5 to 15 per cent of cement ; the higher the clay content the more cement is need. While mixing cement with soil, adequate water must be provided for the hydration of cement. As local soil has usually to be used, and since the properties of soils are very variable, small scale experiments must be made before attempting a big project. The treated soil must have a specified compression strength.

A preliminary estimate of the suitability of a soil for stabilization can be obtained from its particle size analysis, liquid and plastic limits and the results of compression tests on cubes or cylinders of un-treated and treated soils. Weathering test can be made by subjecting the specimens to cycles of alternate wetting and drying. In stabilization using resinous or bituminous admixtures, the effectiveness of the admixtures is determined by measuring the weights of water, either absorbed in a specified time by completely immersed disc shaped specimens, or picked up by capillary action by small cylindrical specimen which have only their lower faces in contact with water. Satisfactory mixes absorb very little water.

Soil can also be stabilized by the addition of the following materials of which the walls can either be made directly, or bricks moulded in the normal way and sun-dried, and walls built as with ordinary bricks.

(*i*) 4 per cent lime ;

(*ii*) 1.5 per cent cement+2.5 per cent lime ;

(*iii*) 0.5 per cent (about 8 litres to 1 cu. m of soil) of liquid Asphalt. Mortar for jointing bricks is prepared with 1 per cent of bitumen.

Soil, which is normally considered suitable for manufacturing *kacha* sun-dried bricks, is processed to the consistency as required for manufacturing such bricks and the bitumen is intermixed thoroughly by kneading the puddle.

The addition of lime to the soil lowers the high plasticity index of clay and retards its tendency to develop shrinkage cracks (along with the sand added to it). The addition of cement and liquid asphalt helps to increase resistance to water erosion.

Soil-cement has been used for house construction in the Punjab and elsewhere which has stood the weather very well. Local soil which is normally considered suitable for manufacturing bricks was generally used, and the method led to very speedy construction. Detailed specifications of the houses built in the Punjab have been published by Indian National Society of Soil Mechanics and Foundation Engineering, from which the following brief notes are given :—

Soil grading :

Sand content	Not less than 35 per cent
Liquid limit	Not more than 25 per cent
Plasticity Index	Not less than 8.5 per cent and not more than 10.5 per cent
Total solids	Not more than 0.5 per cent
Sodium sulphate	Not more than 0.10 per cent.

Tests. The crushing strength of blocks 65 mm in diameter, made out of the cement-soil mixture actually used, shall not be less than 28 kg/sq. cm. The dry bulk density of the compacted cement soil mixture shall not be less than 1.80. "Optimum moisture" shall be added to the cement soil mixture before compaction.

Weather resistance test shall consist of 12 cycles of the following process made on soil blocks : (*a*) Immersion of blocks under water for 5 hours at room temperature ; (*b*) Heating it in an oven for 42 hours at 70 deg. C ; (*c*) Cooling it for an hour at room temperature and brushing off loose material. The specimen at the end of 12 cycles shall not have lost more than 3 per cent of its weight, to pass the test.

The outside plaster shall have at least an adhesive strength of 0.84 kg/sq. cm when two blocks, as for compression strength, are joined together with the plaster.

Plasticity Index of the soil is determined roughly by the "syring test," which consists in extruding the wet soil from 3 mm diameter holes and judging the plasticity index from the state of surface on the filaments of the soil extruded.

Freshly dug soil is so pulverised that not less than 85 per cent of it passes through 8 mm screen. The soil is tested, mixture corrected. required quantity of water added and left overnight to soak. Next morning cement is added and mixed up thoroughly. Cement shall not remain in contact with moist soil for over 3/4 hour before compaction.

Foundations. Cement-soil blocks with 7.5 per cent have been successfully used, but subsequent research shows that 5 per cent soil rammed *in situ* can serve the purpose.

Walls in superstructure. 2.5 per cent cement-soil mixture at optimum moisture, rammed *in situ* between shutterings, and 30 cm in thickness.

Roofing. The conventional type of roofing used in the area. Cement concrete bed plates placed over grooves made on the wall.

Plaster. On the outside walls with 1:5, cement: sand, 12 mm thick over a coat of cement wash.

The cement-soil mixture shall be poured into the shutterings in layers of 75 mm at a time. Compaction shall be done by means of iron rammers 7 kg in weight. Vertical joints shall be provided not more than 1.8 m apart and horizontal joints 90 to 140 cm apart. (Joints are made with the same principles as for RCC works). Vertical joints are made by using end plates. Horizontal joints shall be formed by finishing smooth the rammed surface at the end of the day's work and sprinkling dry sand over it before starting work next morning. Individual layers shall not be finished smooth, though they shall be horizontal.

Form-work. The form-work shall be of the sliding type and shall consist of wooden boarding 23 cm wide and at least 50 mm thick. The boarding shall be supported by vertical stiffeners at least 100 × 100 mm if wooden, or 40 × 40 × 6 mm, if of angle iron, spaced not more than 100 cm apart. These stiffeners shall be held in position by means of spacer bolts 16 mm dia, passing through the stiffeners and the boarding, and adjusted to the exact thickness of the walls. The shuttering shall be made in lengths not more than 3.3 m each and in case of angles and tees of wall corners, each leg shall be about 2.4 m. The end plates shall be plain or tongued depending on whether it is an opening or a vertical joint.

Curing. The drying of the compacted walls shall be controlled by sprinkling water from time to time for about 10 to 14 days. For another 2 to 3 weeks further natural drying out of the walls shall be allowed to let maximum shrinkage take place.

Finishing and Plastering. Immediately after compaction while the wall is still green and soft, all unevennesses shall be scraped off to make the wall faces reasonably even to receive plaster. On the exposed wall faces, a coat of cement wash consisting of 1 part of cement to 3 parts of water by volume shall be thrown evenly on the wall and allowed to set overnight. Before plaster is applied, it must be ensured that the cement

wash is sticking rigidly to the wall face and does not rub off with the fingers. If it does, it should be rubbed off with a soft brush and new wash application repeated. Only freshly prepared cement wash shall be used.

Soil-cement Floors. The following specifications will make a good floor :

(*a*) Soil cement mixture containing 5 per cent cement plus 30 per cent brick ballast passing 32 mm gauge, compacted 75 mm thick. (*b*) Soil-cement mixture containing 5 per cent cement compacted at optimum moisture with hand rammers in 2 × 1.5 m slabs 75 mm thick. After the slabs are cured for a week with a frequent sprinkling of water, 1:3 cement-sand (fine) plaster is applied 12 mm thick in 60 × 60 cm slabs. Loam soil with a plasticity index of 8.5 to 10.5 and a sand content of not less than 35 per cent is used (For "loam" see under "Soil Mechanics".) (Based on the results of the experiments carried out at the P.W.D. Research Laboratory, Karnal).

Bituminised Soil

Sundried (*kuchha*) bricks made of bituminised soil are rendered water repellent preventing their disintegration by rain water ; their bearing power is also improved. The soil which is considered normally suitable for manufacturing *kuchha* sundried bricks, is processed to the consistency as required by the brick moulder. A bitumen of low viscosity is added in the proportion of 0.5 per cent by weight of the soil (approximately 5 litres of bitumen for each 0.6 cu. m of processed soil). The mixing technique is quite simple. The bitumen is spread over the surface of the wet soil puddle prepared by the brick moulder and then intermixed thoroughly by kneading the heap with feet and with spades to a stage that no streaks of bitumen are visible in the puddle. The bituminised soil resulting therefrom is then moulded into bricks in the normal way by the brick moulder and sundried. The bituminised soil mortar used for jointing the bricks is also prepared in the same way, except that the bitumen is added at the rate of one per cent instead of 0.5 per cent used for brick making and the consistency of the mortar is kept somewhat more fluid than that of the bricks.

(Extracts from the Papers published in the Institution of Engineers (India) Journal of Sept., 54 issue, and the reports published by the Hirakud Research Station).

19. FLOORING

Preparation of Base. All earth filling under the floors must be thoroughly rammed. All floors in contact with the ground shall be laid on 10 cm lime concrete over 10 cm of clean dry sand. This will keep out damp and white ants. The lime concrete shall be laid true and parallel to what is required on the finished surface. All floors shall be perfectly level, except bath room and verandah floors, which shall be given an outward slope of 1 in 64. A straight edge of a length not less than 1.83 m and with parallel sides, as well as a 250 mm spirit level, shall be kept for testing the floor levels.

Brick or Tile Flooring. The bricks or tiles shall be the best available selected for smooth face, good colour and hardness. The bricks may be laid flat or on edge, and shall be well soaked in water when to be laid in mortar. The ground surface should be thoroughly watered and well rammed. For the floor to be laid in mortar, the bricks shall be laid with bed and vertical joints quite full of mortar. Where cement pointing is specified, the joints shall not be less than 6 mm thick, and shall be flush pointed after being raked out 25 mm deep whilst the mortar is still damp. The work shall be protected from the effects of sun and rain, and shall be kept moist for 10 to 15 days after the pointing has been finished.

For dry brick paving, the bricks will be laid on edge on 12 mm thick bed of mortar and the joints shall not exceed 6 mm in thickness, which shall be filled with dry sand Top finishing of the dry brick paving may be with : (*a*) 1:2:4 cement concrete 20 mm thick ; (*b*) 12 mm cement plaster 1:3 ; (*c*) Cement pointing 1:3.

Flagged Flooring. The slabs may be of unequal sizes but shall not be less than 35 cm in width or greater than 75 cm in length, and not less than 40 mm in thickness. The flags shall be soaked in water for at least one hour before laying and be evenly and firmly bedded in mortar. Places not firmly bedded can be found by tapping with a mallet. Where no pointing is specified, no joint shall be more than 3 mm in width and must be struck off flush with a trowel whilst laying the flags. Where no pointing is specified, joints shall not be less than 6 mm wide.

Marble Flooring. All marble slabs shall have a minimum thickness of 20 mm and shall be bedded in 20 mm thick mortar. The joints shall be kept as small as possible. When properly set, the floor shall be rubbed with carborandum stone or hard sand and water and then with finer carborandum stone or with brick and emergy powder. The surface shall then be finally smoothed down with pumice stone. When the smoothing process has been completed, the surface shall be polished with putty powder rubbed with felt pads. plenty of water being used.

The flooring must have set fully before any walking over is allowed and no load shall be laid over for at least 7 days.

Soil-cement floors have been described under "Stabilized Soil for Building Construction" ; Timber floors under "Timber Structures" and Cement Concrete floors under "Reinforced Concrete".

20. LIGHTNING CONDUCTORS

Lightning protective systems must be installed on all buildings and structures vulnerable to lightning strokes owing to their height or exposed situations ; buildings of public or strategic importance (factories, magazines, oil and gas tanks, power houses) large warehouses, chimneys, towers, spires, etc.

Lightning conductor is usually a band or a rod of metal connected to a terminal (also called "finial") extending 30 cm minimum above the highest point of the structure (particularly at changes in direction). An air termination need not have more than one point, it is now considered there is no advantage in the multi-point type formerly in use, and only a single air terminal may be used provided it will give the desired zone of protection. Finials may be 12 mm diameter copper or phosper bronze solid rods (pointed at the top).

In case the height of a structure is 36 m or more and width at the top 1.5 m or more, a minimum of two air terminals should be provided connected to a band. On flat metal roofs they should be fixed at intervals of about 30 m. Salient points even if less than 15 m apart, should each be provided with an air termination. Particular attention is necessary to the case of chimneys where the hot gases emitted may act as lightning conductors far into the atmosphere.

Conductor Materials. The materials recommended are copper, copper-clad steel, galvanized steel, aluminium and alloys (phosphor-bronze). Copper is a better conductor than iron in the ratio of 100 to 17 and is also easier to manipulate about the various projections, but iron is better to resist fusion. Copper is preferable to galv. iron where corrosive gases or industrial pollution or salt laden atmospheric conditions are encountered. Aluminium has a conductivity almost double that of copper, weight for weight and is increasingly finding favour.

Minimum Sizes of Conductors (IS : Code)

Material	Above ground	Below ground
Round galv. iron wire	No. 0 SWG (8 mm dia.)	No. 4/0 SWG (10 mm dia.)
Galv. iron strip	20×3 mm	32×6 mm
Round copper wire or copper clad steel wire	No. 4 SWG (6 mm dia.)	No. 0 SWG (8 mm dia.)
Stranded copper wire	0.5 sq. cm or 7/3.4 mm dia.	Not allowed
Copper strip	20×3 mm	20×3 mm
Round aluminium wire	No. 4/0 SWG (10 mm dia.)	Not allowed
Aluminium strip	25×3 mm	Not allowed

All iron roofs of a building, all metallic finials, chimney, ducts, vent pipes, railings, gutters, down pipes, ridges, hips and the like, should be bonded to, and form part of the air termination network. Any metal coming within 1.2 m of the course of a conductor should be connected with it ; any other heavy metals even beyond 1.2 m should also be connected. Where a long line of the metal ridge exists, each end of the ridge should be directly connected to earth by a rod.

Gas pipes should be connected as far away as possible from the position occupied by lightning conductors and as an additional protection the service main to a gas meter should be metallically connected with house services leading from the meter.

The conductors of the lightning protective system should not, as far as possible, be laid parallel to any electric conductor. If this is absolutely unavoidable, the distance between the two must exceed 2.1 m. Metallic parts of the electric installations should not be connected to the lightning protective system. In large buildings where more than one conductor has been fixed, all conductors should be connected by separate horizontal conductors both at the top and at the bottom.

If the roof and other parts to be protected consist of copper, the lightning conductors must be of copper, and if of zinc or galvanized iron, the lightning conductors must be of tinned copper, galvanized iron or bare aluminium. A conductor should be made of the same material throughout including the points and the earth plate, except as given below. The upper end of the conductor under all circumstances should be a solid rod 20 mm dia.

Down Conductors. Down conductors should preferably be run along corners and other projections. The runs should be as straight as possible and should follow the most direct path possible without sharp bends, upturns and kinks, and bends in any direction should not be less than 30 cm radius, otherwise there is the danger of the discharge leaving the conductor and entering the building. No change of direction should be more than 30 deg. Conductors should not be passed round projecting cornices but should be taken through them if possible. Joints should, as far as possible, be avoided. Where these are necessary, they should be soldered and double riveted; joints for rod may be of the clamped or screwed type. Keep all joints at accessible places. Conductors should not be insulated from the walls but should be secured by clamps of the same metal as the conductor, built into the wall. The supports or hold-fasts should be such as to allow the conductor to expand or contract with changes of temperature; slight loops are provided in the tape between the hold-fasts.

Earth Connections. The lower extremity of the conductor ("earth") should be buried in permanently damp soil. Ground plates should not be laid directly in waters of wells, trenches, lakes, etc. The ground where the earth electrodes are buried should be made slightly sloping towards the burial spot so as to permit rain water to soak in. A water-drain from the house can also be led to the place of the "earth". Where dampness is not assured the earth plate or rod should be buried at least 3 m into the ground with charcoal or powdered coke filling extending 15 cm around it on all sides. Coke should not be used with copper plates. No "earth" should be fixed within 3 m of a wall or of foundations.

Where bed rock is found near the surface, ground connections may be made by digging trenches radially from the building and burying in them the conductors or their equivalents in the form of metal strips or wires. The depth of the trench should be 60 to 90 cm.

Each down conductor should, preferably, have an independent earth termination and may be connected to water pipe systems in the vicinity in addition. The conductor in case of important buildings should bifurcate close below the surface of the ground, and one lead should be led to the nearest water main and soldered to it; where water mains do not exist a 30 mm galv. pipe at least 50 cm in length should be driven into the ground under sub-soil water level and the lead connected thereto. The second lead should be "earthed" as described under.

Earth Electrodes. Earth electrodes should consist preferably of copper rods and/or strips driven down to sub-soil water level.

(*i*) *Rods*: These should be of 12 mm diameter and driven into the ground to a depth of at least 2.4 m.

(*ii*) *Pipes*: 50 mm galv. iron pipe driven down to at least 2.4 m below ground level. The pipe should preferably go down into the sub-soil water for about 1.8 m. The lower 1.8 m of the pipe should be perforated with 3 mm drilled holes and the pipe filled with finely powdered charcoal through which the conductor cable is passed to the bottom of the pipe and is also riveted and soldered with the pipe at its top end. When the sub-soil is gravel or where the ground is made up, pipe should not be used.

Strips: When strips are used these should be buried in trenches at least 45 cm deep, in straight lines or parallel lines, or in radial formations. The distance between any pair of parallel strips should not be less than 1.8 m. Strips have to be used where rock is encountered at a depth less than 2.4 m below ground level or 2.1 m below the lowest level of the excavation, thus prohibiting the use of vertically driven pipes.

Plates: The "earth" plates may be of size 90 cm × 90 cm × 2.5 mm thick if of copper, and copper plates must be used when the conductor is of copper. The plate should be 6 mm thick if of iron. Copper plates to be tinned on both sides and iron plates heavily galvanised. In a dry ground a larger plate is necessary. The plates are buried to a depth where they will be continuously wet, and iron plates should be surrounded by charcoal. The plates may be buried either horizontally or vertically but the vertical method is better. The conductor is soldered, riveted or welded to the earth plate.

The lightning protective system should be thoroughly tested when finally completed, and once a year of important structures. The overall resistance of the complete conductor system from the top finial to the earth in wet weather should be about 1—2 ohms and should never exceed 10 ohms. The test should preferably be conducted in the driest season of the year. The test can be carried out with the Megger Earth Tester.

Zone of Protection. For all ordinary buildings the zone of protection for a single vertical conductor is considered to be a cone with the apex at the highest point of the conductor and a base of radius equal to the height. Structures of base area up to 90 sq. m may have one conductor only. One additional conductor is required for every 300 sq. m or apart thereof in excess of 90 sq. m or at least one down conductor per 30 m of perimeter, whichever is less. The angle of protection may be considered to be 45 deg.

Protection of Special Structures

Buildings with inflammable roofs: All parts of a lightning protective system installed on structures with such roofs should be separated by non-metallic supports at least 30 cm from the roofing material.

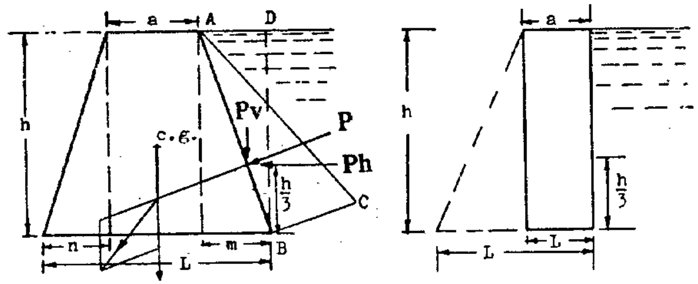
Structures with Explosive or Inflammable Contents. Such structures should not have spires, or flagstaffs. Radio aerials should not be provided within 15 m of the structure. One or more tall supports each equipped with a lightning protective system should be provided in such a way that the structure falls within the zone of protection.

The overhead line of electric supply to the structure should be terminated at a point about 15 m away from the building, whence the supply may be taken by underground cables. Electric cables entering the structure should be metal sheathed, the sheathing being earthed at the point of entry outside the structure. Other metallic objects like water pipes, wire ropes, rails, etc., entering the structure should be earthed at the point of entry and also at a few more points to a distance of about 150 m. All metal forming parts of the structure should be bonded or welded together and connected to the ring conductor at least at two points.

(See also Section on "Electric Services")

21. WATER STORAGE TANKS AND DAMS

Design of Walls Against Water Pressure



Pressure Due to Fluids :

Pressure at B—is $w \times h$ (w is unit weight of water)

Total pressure on the wall $AB = w \times (h/2) \times AB$

The overturning moment is found in the same manner as explained under "Design of Walls Against Wind Pressure" or "Retaining Walls".

When L (thickness of a rectangular wall) = 5/12 h this will exactly balance a masonry wall of weight 1900 kg/cu. m against pressure of water on its full height. But the resultant must fall within the middle-third to avoid cracks and tension and the wall (or dam) must also be safe against sliding.

For the resultant pressure to be within middle-third of the base the width L of the base can be found from the following formulae :—

Symmetrical (*Trapezoidal*) Section :

$$L = \sqrt{h^2 - (a+n)^2(g-1) + n^2g + \frac{n^2g}{4}} - \frac{ng}{2}$$

Front face vertical (n=0) : $L = \sqrt{h^2 - a^2(g-1)}$

Back vertical (m=0) ; $L = \sqrt{\frac{5}{4}a^2 + \frac{h^2}{g}} - \frac{a}{2}$

or $L^2 + aL - a^2 - \frac{1}{g}h^2 = 0$,

Rectangular section : $L = h\sqrt{\frac{1}{g}}$

$g = \frac{\text{weight of masonry}}{\text{weight of water}}$; w = unit weight of water.

Back face vertical is more economical.

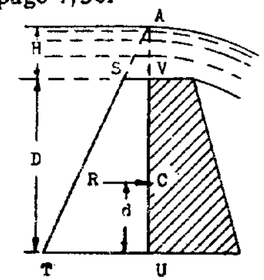
Top width is generally kept about 1/3 to 1/5 of the height. Stress at the edge (toe) =

$$\frac{2V}{3\left(\frac{L}{2} - e\right)} = \frac{V}{L}\left(\frac{4}{3 - 6\frac{e}{L}}\right)$$ V – is the total vertical load

Sliding—To be safe against sliding the weight of dam × co-efficient of friction should be greater than the total water pressure Ph (or any lateral force tending to slide the dam or wall).

Co-efficient of friction for sliding are given at page 7/36.

Total pressure per unit length is equal to area of the trapezoid S T U V



$R = \frac{D}{2}(2H + D)\,w$; $d = \left(\frac{3H + D}{2H + D}\right)$

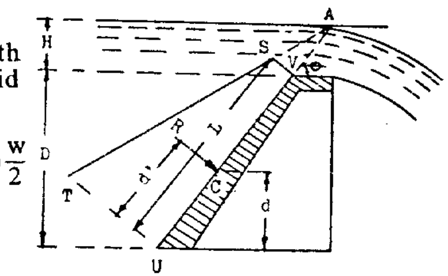
$SV = wH \times TU = (D + H)w$

Pressure at V is SV

" " U is TU

All pressures act normal to the surface.

Total pressure P per unit length of dam is given by the area of trapezoid of pressure S T U V or



$R = L(2H + D)\frac{w}{2} = \frac{D}{\sin\phi}(2H + D)\frac{w}{2}$

$d' = \frac{d}{\sin\phi} = \frac{L}{3}\left[\frac{3H + D}{2H + D}\right]$

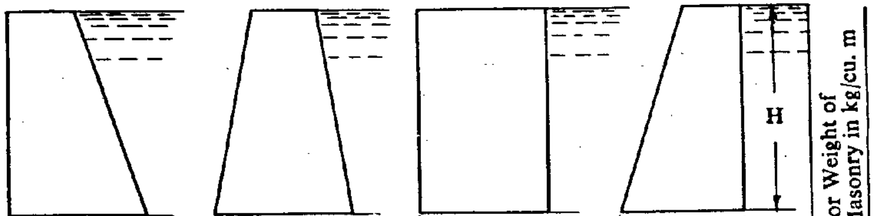
The horizontal component of the pressure against the face of the dam is the same as for a vertical surface. The vertical component of this pressure is given by the total weight of water vertically above the face of the dam.

If H = 0, $R = \frac{D^2W}{2\sin\phi}$ and $d' = \frac{L}{3}$

When there is possibility of upward water pressure (due to infilteration) at the base, this force should be subtracted from the weight (or mass) of the wall. (See also under "Retaining Walls").

To safeguard against uplift (giving 50 per cent allowance) the ratio of the base width to height should be 0.75.

SECTIONS OF WALLS FOR RETAINING WATER



Base	P	Base	P	Base	P	Base	P	For Weight of Masonry in kg/cu. m
.96H	980H	.78H	850H	.74H	1170H	.65H	780H	1920
		.74H	930H	.70H	1270H	.62H	850H	2080
		.71H	1000H	.67H	1360H	.60H	930H	2240
.95H	1120H	.68H	1049H	.64H	1460H	.58H	980H	2400

P is the max. toe pressure in kg/sq. m at the base level.
Foundation projections & concrete will be below the base line.
Extention of foundations will reduce the toe pressure.
Top width may be about 0.2 to 0.3 H, except for rectangular section.

Where the earth pressure against masonry wall of a reservoir is of permanent nature and well rammed, or the reservoir is below ground level, there will be excess pressure on the water side equivalent to that form a fluid weighing 560 kg/cu. m. Therefore, the wall/walls of such a reservoir need be designed only for this pressure.

22. STONE MASONRY—General Specifications

(For Characteristics of Stones, see Section 12)

Normally stones used should be small enough to be lifted and placed by hand. The lengths of the stones should not exceed three times the height and, the breadth on base should not be greater than three-fourth of the thickness of the wall, not less than 15 cm. The height of stones may be up to 30 cm. Stones with round surface should not be used. All stones should be wetted before use. Percentage of water absorption shall not be more than 5 per cent. Minimum crushing strength of building stones shall be 200 kg/sq. cm.

(*a*) Random Rubble Masonry :

The stones are hammer-dressed on the face, sides and beds to such an extent that the stones will come into close proximity with the neighbouring stone.

Beds and joints—Not to exceed 25 mm thick.

Height of course - Uncoursed.

Bonds or through stones—Bond stones running right through the wall shall be given at least one bond stone for every 0.5 sq. m of wall surface, and staggered.

Quoins or corner stones—Face beds to be squared back carefully at least 100 mm and joints 65 mm.

Hearting—Stones to be not less than 15 cm in any direction, carefully laid, hammered down with wooden mallet into place and solidly bedded with mortar, chips and spalls being wedged in to avoid thick beds of joints and mortar.

(*b*) Random Rubble brought up to course :

The stones are hammer-dressed on bed and top surface unless the natural cleavage of the stones give parallel faces. No face stone to be narrower or shorter than its height, and no such stone shall tail into the wall less than its height and at least 1/3 of the face stones shall tail into the wall twice their height.

Beds and joints—Not to exceed 12 mm thick. The stones shall break joint on the face for at least half the height with those of courses above and below.

Height of course—Not less than 150 mm in height and brought up to level beds, and shall be laid at right angles to the batter.

Bonds or through stones—In the interior thickness of the wall bond stones at least 45 cm long shall be given so as to approximately provide through-bond of long stones every 150 cm.

Quoins—As for (a) above.

Hearting—As for (a) above, but vertical plums to be placed wherever possible, projecting not less than 150 mm to form bond between successive courses.

(*c*) Square Rubble, uncoursed :

Beds and joints—Not to exceed 6 mm thick. Face beds to be squared back at least 100 mm and joints 65 mm. No spalls or pinnings to show on face.

Height of course—Uncoursed, but no stone to exceed 200 mm in height to avoid long vertical joints.

Bonds or through stones—As for (a) above.

Quoins—As for (b) above, but corner of each quoin to have chisel drafted margin of 25 mm on each side to facilitate plumbing.

Hearting—As for (b) above.

(*d*) Square Rubble, brought up to courses :

Beds and joints—To be one-line dressed.* No face joint shall be

* "One-line dressed" means sparrow picked or chisel dressed so that no portion of the face dressed is more than 6 mm from edge of a straight-edge laid along face of stone.

thicker than 10 mm. The face stone shall be laid alternate headers and stretchers.

Height of course 15 to 23 cm. No course to be of greater height than any course below.

Bonds or through stones—150 cm apart in the clear in every course and to be staggered.

(e) Block in Course :

The stone shall be hammer or chisel-dressed on all beds and joints so as to make rectangular shapes (two-line dressed).* Joints shall be dressed at right angles to the face for a distance of 75 mm.

Beds and joints Not to exceed 3 mm thick. The face stones shall be laid alternate headers and stretchers.

Height of course Each course shall consist of stones of even thickness not less than 125 mm. No stones in face shall have less breadth than height ; and no stone shall tail into the wall less than its height and at least 1/3 of the face stones shall tail into the wall twice their height.

Bond or through stones—Through stones going right through the wall for walls up to 75 cm thick, shall be inserted in each course at 150 cm intervals breaking joints with similar stones in courses above and below at least 60 cm.

Quoins Short bed to be at least equal to height and long bed at least equal to twice height. Beds and joints to be squared as for walling.

(f) Ashlar :

Every stone shall be chisel-dressed on all beds and joints, to be true and square giving perfectly vertical and horizontal joints with the adjoining stones or brickwork. (Three line dressed).*

Beds and joints No joint shall be thicker than 3 mm. The face stones shall be laid alternate headers and stretchers ; the headers shall be arranged to come as nearly as possible in the middle of the stretchers above and below so that the stones break joint on the face for at least half the height of the course.

Height of course Not less than 30 cm. No stone to be less in breadth than in height, or less in length than twice its height.

Bond or through stones—Not exceeding 180 cm apart in the clear, and to be staggered. In walls 75 cm thick and under, the headers run right through the wall, if more, overlap at least 15 cm.

Stone Arching. In arches up to 38 cm thick all stones shall be through stones extending from the intradoes to the extrados. In arches over 38 cm thick it may be convenient to build two rings in which case the stones shall be laid alternatively headers and stretchers, the headers shall be through stones from intrados to extrados and the stretchers through stones for one ring. In the case of three rings alternate headers shall break joint to the amount of the full depth of one ring.

In the case of rubble arching, stones shall be not less than 75 mm thick on their least dimension and shall break joint for not less than 150 mm and all stones in one course shall be of approximately the same thickness. The thickness of the joint at the intrados shall not exceed 12 mm and the open extrados joints shall be solidly wedged with chips and spalls set in mortar.

For ashlar fine tooled work, no stones shall be less than 30 cm long and 50 per cent of the stones shall be 45 cm long or over. The thickness of the joints shall not exceed 5 mm and all joints shall radiate properly from the centre of the curve.

In the case of block-in-course, stones shall be not less than 15 cm thick on their least dimensions and, thickness of joints shall not exceed 6 mm.

Plumb Concrete and Plumb Masonry

The proportion of plumbs (big stones) should not generally exceed 50 per cent of the total volume of the plumb concrete. For this it will be sufficient to make up the total volume with plumbs and fill in all the interstics which should not be less than 15 cm, with cement concrete. The cement concrete may be 1:3:7 (cement, bajri, coarse aggregate). The plumbs are laid in layers using the cement concrete as mortar.

23. GENERAL SPECIFICATIONS FOR BRICKWORK

Bricks should be soaked in water for at least one hour before use for works in cement and lime mortars. (Tests, however, show that bricks absorb no more water after 15 minutes soaking). The bricks should be sufficiently soaked before use but not excessively so. The absence of bubbling when the soaked brick is immersed in water is the test for thorough soaking. See page 12/3.

Bricks made from saline soil (known as *Kallar* soil are not suitable for building works. Such bricks show heavy efflorescence and scum (white powdery deposit on the surface of the bricks), crumble on weathering and lose strength in course of time. The surface of the walls gets disfigured and the plaster peels off quickly. When used in reinforced structures such bricks corrode the reinforcement. The new work should be kept wet during construction and for 10 days after completion.

For brickwork in mud, the bricks should only be dipped into a tub of water and not soaked before use.

Masonry work such as at top of walls, should be kept covered with 12 mm of water for about 10 days when the work is not in progress. For this purpose the top of masonry is provided with small mud mortar parapets all round the edges and crosswise so as to form small compartments. (Does not apply to brickwork in mud.)

* "Two-line dressed" means sparrow picked or chisel dressed so that no portion of the face dressed is more than 3 mm from edge of a straight-edge laid along face of stone.

* Three-line dressed" or fine chisel-dressed means that the surface of the stone is dressed until a straight-edge laid along the face is in contact at every point, this is also called "plain face".

No mortar joint should exceed 6 mm for 1st class brickwork in cement and 10 mm for 2nd class brickwork in lime, 12 mm for 3rd class brickwork in mud mortar, and no joint should be less than 5 mm in thickness.

Mortar of the proper consistency only should be delivered on the work, any subsequent thinning with water should be prohibited. Mortar that is too thick or too thin should be sent back to the mixing floor.

Bricks should normally be laid in English bond with frogs upward.

The surface of each course should be thoroughly cleaned of all dirt before another course is laid on top. If the mortar in any course has begun to set, the joints should be raked out to a depth of 12 mm before another course is laid. When the top course has been exposed to the weather for any length of time it should be removed and the surface of the second course thoroughly cleaned before any more courses are added. The work should be added on uniformly throughout so that there is equal distribution of pressure on the foundations, to avoid cracking ; no portion of the work shall be left more than 90 cm lower than another.

If a work is to be pointed or plastered the surface should be prepared as explained under "Plastering". If pointing or plaster is not provided as a separate item the joints should be struck and finished at the time of laying. Straight lines can be marked with a string which is pressed into the mortar.

When work is to be built on a soil that contains harmful salts (even in traces) only selected well burnt bricks (preferably slightly over-burnt) should be used for a height of at least 60 cm above ground level, as bricks which are not thoroughly burnt rapidly corrode away in such a situation.

Lime Concrete

Brick ballast where used must be well soaked with water before mixing. The ballast should be spread evently on a floor and the correct proportion of well mixed dry mortar spread over it. The material is then thoroughly mixed. The surface of the concrete while being rammed may be lightly sprinkled with water to compensate for loss by evaporation in the hot season.

Preventing Cracks in Building Construction

Provide a smooth bearing for RCC slabs on the wall with 6 mm cement plaster 1:3, finished with a floating coat of neat cement, covered with a thick coat of lime wash or kraft paper. The sides and top of slabs and beams in contact with walls should be pointed with thick coat of hot bitumen at 1.7 kg/sq. m.

The slab should not bear on full thickness of external wall. A gap of about 12 mm should be kept between the slab and external masonry and filled with bituminous filler or impregnated fibre board. Bituminous filler can be made of : 80 kg hot bitumen, 1 kg eement, 0.25 cu. m coarse sand. The external masonry of wall beyond the expansion joint should not be less than 10 cm.

Provide RCC or plain cement concrete 1:2:4 bed plate finished with a smooth surface as above, under beams. Minimum thickness of RCC bed plate should be 10 cm and that of plain concrete 20 cm.

Expansion joints should be provided at 5 to 6 m in case of sun shades, 12 to 13 m in case of covered verandah slabs and 12 to 15 m in case of slabs continuous over rooms in a row of quarters.

In the case of *sun shades*, the expansion joints should not extend to the portion embedded in masonry but should stop short of the face of the wall by 5 cm, and distribution reinforcement in the embedded portion and in the 5 cm portion of the chaija slab where there is no expansion joint, should be increased to 40 per cent of the main reinforcement. The gap in the expansion joint in the projected portion should not be filled with any material.

Where RCC slabs or beams are embedded in masonry, provision should be made for their expansion and contraction (sliding), especially at the joints to avoid cracks in the masonry.

23. GLOSSARY OF TERMS

Arcade : A series of arches with their supporting columns or piers.

Arris : The meeting of two surfaces producing an external angle.

Base is immediately above plinth. A building having no plinth, immediately above footings.

Basement or Basement Storey or Cellar : Part of a building (usually a storey) below ground level.

Bat : Part of a brick.

Batter : The slope away from you of a wall or timber piece, etc.

Bay : The space between two piers, columns or projections.

Bay window : A window projecting outward from a wall and reaching up to the ground.

Bevel : Any inclination of two surfaces other than 90 deg. (either greater or less).

Blocking Course : A course of stones (or only one stone) placed on the top of a course to add to its appearance and also to prevent the cornice from overturning.

Bressummer : Joist embedded in concrete ; beam over verandah posts on which purlins of sloping roofs rest. Also means a beam which carries a wall.

Brick core : Brickwork filled in between the top of a lintel and the soffit of a relieving arch.

Brick nogging : Brickwork filled in between wooden posts or studs (for making a wall).

Bull's eye : A circular or oval opening in a wall.

Buttress : A projection of masonry built into the front of the wall to strengthen it for lateral stability against thrust from an arch, roof, or wind pressure.

Flying Buttress : A detached buttress or pier of masonry at some distance from a wall, and connected therewith by an arch or portion of an arch, so as to discharge the thrust of roof or vault, on some strong point.

Chamfer : To cut off, in a small degree, the angle or arris formed by two faces, usually at an angle of 45 deg.

Chase : A recess made inside of a wall to accommodate pipes or electric wiring, etc.

Composite Building : A building of which part is masonry and part is either open or framed ; or a building of which part is open building and part is framed building.

Coping : The capping or covering placed upon the exposed top of a wall (or parapet), usually of stone, to throw off and prevent the rain-water soaking into it.

Corbel : One or more courses of brick projecting from a wall (like a cornice), generally to form a support for wall plates, etc. A brick should not project more than 1/4 beyond the lower course.

Counterfort : Is a projection of masonry built into the back of the wall.

Cowl : A hood shaped top for a chimney ; a ventilating top of a sewage pipe.

Cross Wall : An internal weight bearing wall built into another wall to the full height thereof.

Dormer Window : A small vertical window built in a sloping roof.

Dowel : A pin or peg let into two pieces of stone or wood for joining them ; a cramp iron.

Drip : Part of a cornice or projecting sill etc., which has a projection beyond other parts for throwing off rain-water.

Efflorescence : The formation of a whitish loose powder or crust, on the surface of brick walls.

Extrados : The outer surface of an arch.

Frog : Is a small recess on the top surface of a brick, made while moulding, usually embossed with the initials of the contractor. It forms a key for the mortar and also reduces the weight of the brick.

Gable : The entire end wall of a building. (The term is generally used for the triangular end wall of a sloping roof.)

Haunch : That part of an arch lying midway between the springing and the crown.

Herring-bone work : Masonry work (generally in floors) in which the bricks are laid slanting in opposite directions.

Hydroscopic : A substance that attracts water from the air.

Intrados : The inner surface of an arch.

Jambs : The two sides of doors, windows or other openings between the back of a wall and the chowkat or frame. The portions of the openings outside the frames are called *Reveals*.

Joggle : A dowel or stub tennon joint by means of which one piece of stone or timber is fitted to another.

Keystone : The uppermost or central voussoir of an arch.

King closer : A brick cut lengthwise so that one end is nearly half the width of the other. They are used in the construction of jambs.

Lobby : An open space surrounding a range of chambers, or seats in a theatre ; a small hall or waiting room.

Mantel : The facing and shelf (usually ornamental) above a fire-place.

Mastic : A preparation of bitumen used for water-proofing and damproofing, etc.

Mat finish : A term applied to surface finishing (generally painting) which is free from gloss or polish (not shining).

Mezzanine floor : An additional (low storey) floor, gallery or balcony erected between the floor and ceiling of any storey.

Mosaic : Small pieces of stones, glass, etc. (generally of different colours) laid in cement mortar to form artistic patterns for flooring and dados, etc.

Mullion : An upright (piece) in any framing ; a division piece between the sash of a frame.

Oriel Window : An upper storey window projecting outward from a wall (and which does not reach up to the ground, as distinguished from a bay-window).

Party Wall : A wall erected on a line between adjoining property owners and used in common.

Pedestal : A base or support, as for a column or statue, and generally of a bigger size.

Pilaster : A right-angled column or projection from a pier or wall ; a square pillar made generally to support a concentrated load.

Pillar : A detached vertical support to some structure ; a solid portion of a wall between window openings and other voids.

Plinth : The portion of the external wall between the level of the street and the level of the floor first above the street.

Queen closer : A brick cut lengthwise into two so that each piece is half as wide as the full brick.

Quoin brick : A brick forming a corner in brickwork ; it has one end and one side exposed to view.

Recess : A depth in the thickness of a wall.

Refractory materials : The term "refractory" is applied to various heat resisting materials such as, fire-bricks, furnace linings.

Reveal : A vertical side of a window or door opening from the face of the wall to the frame. (*See Jambs*).

Skew-back : That (inclined) part of a pier or abutment from which an arch springs.

Sleeper Walls : Low walls erected at intervals between the main walls to provide intermediate supports to the lowest floor.

Snap header : A brick header not extending the full length of a brick into a wall, usually half a brick.

Soffit : The lower horizontal face of anything ; the underface of an arch where its thickness is seen.

Spall : Bat or broken brick ; stone chips.

Spandrel or Spandril : The space between the top of a masonry arch and the roof, beam or carriageway, etc.

Spandrel Wall : A wall built upon the extrados of an arch up to the top level of the roof or beam, etc.

Splay : An oblique surface (bevel or chamfered), as of the jambs of a doorway or window ; of which one side makes an oblique angle with the other.

Springing line : A line of intersection between the intrados and the supports of an arch.

Spring points : The points from which the curve of an arch springs.

Springer : The voussoir placed next to the skew-back in an arch.

Squint Bricks : Bricks used for forming acute or obtuse corners in brick masonry.

Striking : The releasing or lowering of centrings of arches or lintels.

String Course : A horizontal (usually ornamental) course projecting along the face of a building, (usually introduced at every floor level or under windows or below parapets) for imparting architectural appearance to the structure and also keeping off the rain water.

Throating : Term used for making a channel or groove on the underside of string-courses, copings, cornices or sun-shades, etc., to prevent rain water from running inside towards the walls.

Underpining : The process of supporting the existing structure for renewing or repairing the lower walls or foundations.

Vault : An arched masonry structure (with series of arches).

Veneered Wall : In a wall in which the facing material is merely attached to. and not properly bonded into the backing.

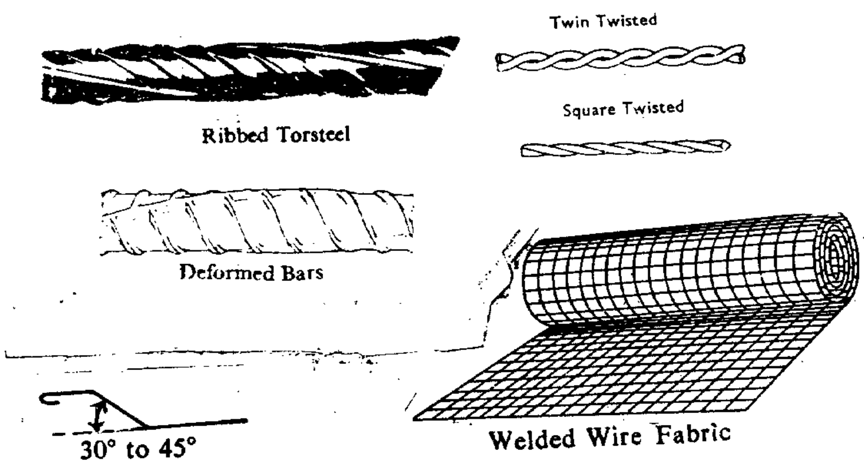
Voussoir : The wedge shaped structure component of a stone arch.

Weathering : A slight slope or fall given on the upper surface of cornices, copings, sun-shades, window sills, etc., to throw off the rain-water; action of sun and rain on structures or soils.

SECTION 8

REINFORCED CEMENT CONCRETE & REINFORCED BRICKWORK

Page


Twin Twisted
Ribbed Torsteel
Square Twisted
Deformed Bars
30° to 45°
Welded Wire Fabric

The following Notations have been used generally :

fc = working compression (in bending) stress in concrete,
fs = working tensile stress in steel,
fs' = working compressive stress in steel in compression,
Es = modulus of elasticity of steel,
Ec = modulus of elasticity of concrete,
m = modular ratio of steel and concrete,
BM = moment of resistance or bending moment,
Mc = ,, ,, relative to the concrete,
Ms = ,, ,, relative to the steel,
As = total area of tensile steel in sq. cm,
C = area of concrete in compression in sq. cm,
b = breadth of beam or width of flanges in T-beams,
b' = width of stem or web,
d = depth of beam from top of concrete to centre of tensile steel (effective depth) ; dia of bar,
h = overall depth of beam,
j = ratio of lever arm of resisting couple to depth d,
k = ratio of depth of neutral axis to depth d,
p = percentage of steel or pitch of shear resistance bars,
S = intensity of shear stress, (nominal shear stress)
Sp = permissible shear stress,
S_R = strength of shear reinforcement,
St = total shear force across the section under consideration,
s = permissible unit punching shear stress,
Aw = cross sectional area of stirrups,
t = thickness of flange of T-beam,
$Q = \frac{BM}{bd^2}$ moment of resistance factor,
I = moment of inertia,

SI units have been used in the latest IS Code : 456—1978. In SI units, stress is expressed in N/mm^2, b and d in mm and, area in mm^2. BM is expressed in N . mm units. The relationships in general are as follows :

1 kg/cm^2 is taken as 0.1 N/mm^2	10 kg/cm^2 is taken as 1 N/mm^2
1 kg-cm is taken as 100 N-mm	0.01 kg-cm is taken as 1 N-mm
1 kg (f) is taken as 10 N	0.1 kg (f) is taken as 1 N
1 t/m is taken as 10 k N/m	100 kg/m is taken as 1 k N/m

1. THEORY OF REINFORCING CONCRETE AND BASIS OF DESIGN

The use of plain concrete in structural works is limited by the fact that the tensile strength of concrete is only about 1/10th its compressive strength. Hence, a beam of plain concrete will fail in the bottom when the top portion can still take ten times the stress. By inserting steel bars in the bottom of the beam to take the tensile stress, the beam is made ten times as strong as a plain beam. Volume for volume steel costs about 60 times as much as concrete and for the same cross-section steel resists about 280 times as much in tension and 28 times as much in compression as concrete. Therefore, a combination of concrete and steel makes for economy.

Methods of Design

Two methods of design for reinforced concrete structures are used, the *working stress* or *elastic* method, and the *load factor* or *ultimate load* method. In the working stress method, the design is based on the working loads and the criterion for the strength of the structure is its capacity to sustain the loads and forces imposed on it. The load factor method of design is based on a determination of the load at which a structure fails, and a certain factor of safety is adopted. The working stress method of design, which is the older and by far the still most commonly used, is adopted here. (Both the methods give somewhat different results.)

Factor of Safety, which is the relation between the ultimate strength at failure and the permissible stress, generally adopted is 3 for concrete based on cube crushing strength, and 2 for steel based on the ratio of yield stress to permissible stress.

Modulus of Elasticity is a measure of the elastic property of a material and is the ratio between the stress caused by an applied load and the resulting strain or deformation, which disappears on removal of the load. In other words, modulus of elasticity $= \frac{\text{stress}}{\text{deformation}}$.

Modular Ratio. The relation between the modulus of elasticity of reinforcing steel and the modulus of elasticity of concrete is called the modular ratio. It is represented by notation "m" here.

As there is no relative movement between concrete and steel in a reinforced concrete unit, the elongation or contraction of both concrete and steel is equal. As such, modular ratio m is proportional to the permissible stresses in steel and concrete which work together. In other words, the value of modular ratio varies with the modulus of elasticity of steel and concrete respectively.

$$\frac{\text{Modulus of elasticity of steel}'}{\text{Modulus of elasticity of concrete}} = \frac{\text{Stress in steel}}{\text{Stress in concrete}} = m.$$

The modulus of elasticity of steel is taken at a constant value of

200000 N/mm² but the modulus of elasticity of concreté is very variable and varies with the strength of the particular concrete. The modular ratio specified in the Indian Code of Practice is $\frac{2800}{3fc}$ where fc is maximum permissible compressive stress due to bending in concrete in kg./sq. cm.

The modular ratios for various grades of concrete will be as follows :

Grades of concrete	M 100 1:3:6	M 150 1:2:4	M 200 1:1.5:3	M 250 1:1:2
Modular ratio m	31	18.7	13.3	11

(Rounded-off values)

According 1978 Code value of "m" is $\frac{280}{3fc}$, where fc is in N/mm².

When a steel bar is embedded in the bottom of a concrete beam and the beam is stressed, the concrete and the steel will extend or compress equally together provided there is no slip of the bar in the concrete, the deformation in both the materials will be equal. Since stresses are proportional to the respective elastic moduli, the stress in the steel will be "m" times the stress in the concrete. Similarly, if a steel bar is embedded in a concrete column, then under load the steel and the concrete both must shorten by an equal amount, and since the steel takes "m" times more stress than the concrete when strained equally, the steel will carry "m" times more load per unit area than the concrete.

The tensile stress (in the bottom portion of a beam under load) is in design assumed to be carried wholly by the steel, the strength of the concrete in tension being neglected as it will have failed before the steel is fully stressed under the working load ; the concrete on the tensile side will always crack (though the cracks may not be visible to the naked eye).

Due to inequalities of workmanship and materials and variable conditions during placing and other reasons, strength of the concrete will be found to differ considerably even in adjacent parts of the same structure. Many assumptions are made in reinforced concrete design, therefore, fictitious accuracies is merely a waste of time.

Grades of Concrete. Concrete is of two grades—ordinary concrete and "controlled concrete." Ordinary concrete in which the proportions of cement, sand and aggregate are arbitrarily specified (like 1:2:4, 1:3:6). Controlled concrete is in which the proportions of cement, sand, aggregate and also water are determined by tests in a laboratory, and the exact proportions depend upon the gradings and size of the aggregate. Controlled concretes give higher strengths by about 25 per cent for the same proportions of ingredients. (The ingredients in controlled concrete are slightly different from the arbitrary proportions of ordinary concretes.) Concrete consolidated by vibrations give still higher strengths by about 10 percent than when consolidated by hand.

Properties and Strength Requirements of Structural Concrete and Steel

The strengths expected to be attained by concretes of different mixes and the permissible stresses are laid down in Tables I and II.

Tests on concrete may be carried out either on site (called "field test" or "works test") or in a laboratory. For field grade concrete, tests may be limited to "workability" and the "slump tests". Workability means the correct proportioning of coarse and fine aggregates and the water. Slump tests will be described hereafter. Tests are carried out before the work starts and also while the work is in progress. To test the quality of the concrete that has been placed and has hardened, it is necessary to cut out sample pieces, called "cores".

Compressive strength of concrete is assessed by the crushing to destruction of the test cubes which require the use of a compression testing machine and is usually carried out in a laboratory. The other test carried out is the transverse strength test (beam test) to determine the modulus of rupture (or flexural strength) of the concrete.

The crushing strength of a concrete is tested either by 15 cm cubes or 15 cm diameter and 30 cm high cylinders. Cylinder tests are adopted in American practice and cube tests in British practice. Cylinder test specimens indicate lower strength figures than cube test specimens by 0 8. Cylinders fail by shear at 60 deg. and cubes fail at 45 deg. to the horizontal.

The allowable maximum working stress for flexural compression (compression in bending) in concrete is fixed from the crushing strength tests with a factor of safety of 2.4 for cylinder tests, and 3 for cube tests, at 28 days' strength. The direct compressive stress is taken 0.8 of the flexural compressive stress.

The flexural (or bending) strength is determined by means of a beam test ; a concrete beam under bending will break on the tension face. Concrete tested in direct tension gives a different result from concrete tested in bending, and to keep the distinction clear, the latter is expressed as the *modulus of rupture*, which is the extreme fibre stress at the time of the fracture. The flexural strength of concrete is about 1.3 to 1.8 times the direct tensile strength.

The cube crushing strength alone of a concrete is not final and reliable guide to its quality and durability ; it must also have an adequate cement content and a low water-cement ratio, as explained in the text following. The strength and durability of a concrete depends upon its quality and constituent materials and susceptibility of corrosion of steel is governed by the cover provided and the permeability of the concrete.

Badly mixed concrete is weaker in some parts and stronger in others. Concrete gains strength more slowly at low temperatures than at high ; if frozen, setting and hardening ceases ; if the temperature is too high, water required for the hydration of cement may evaporate and the concrete may not develop its full strength. Strength of concrete increases with age. For ordinary Portland cement concrete, strength at 3 days is about 1/3rd of the strength at 28 days ; strength at 7 days is 3/4th of the strength at 28 days, and strength after one year is 60 per cent greater of the strength at 28 days.

Strength at 28 days is taken as the standard strength for design purposes.

Stresses due to shrinkage or expansion of the concrete may be neglected in calculations for common structures.

A reinforced concrete member of sound concrete designed to the permissible stresses may be taken to have a factor of safely of about 2.5 at 28 days after placing the concrete.

Table I. Strength Requirements of Cement Concrete

Grade of Concrete	Crushing or Characteristic Compressive Strength		Modulus of Rupture by Beam Test	
	at 7 days	at 28 days	at 3 days	at 7 days
M100–1:3:6 (M10)	70 kg/cm2 7 N/mm2	100 kg/cm2 10 N/mm2	12 kg/cm2 1.2 N/mm2	17 kg/cm2 1.7 N/mm2
M150–1:2:4 (M15)	100 kg/cm2 10 N/mm2	150 kg/cm2 15 N/mm2	15 kg/cm2 1.5 N/mm2	21 kg/cm2 2.1 N/mm2
M200–1:1.5:3 (M20)	135 kg/cm2 13.5 N/mm2	200 kg/cm2 20 N/mm2	17 kg/cm2 1.7 N/mm2	24 kg/cm2 2.4 N/mm2
M250–1:1:2 (M25)	170 kg/cm2 17.0 N/mm2	250 kg/cm2 25 N/mm2	19 kg/cm2 1.9 N/mm2	27 kg/cm2 2.7 N/mm2

Based on IS:456–1964 and 1978 (1 N/mm2 is 10 kg/cm2)

Fexural strength or Modulus of Rupture $=0.7\sqrt{f_{SR}}$ N/mm2

where : f_{SR} is characteristic compressive strength of concrete in N/mm2.

Crushing strength tests are made on 15 cm cubes. The maximum nominal size of aggregate should not normally exceed 40 mm. Where the size of the coarse aggregate does not exceed 20 mm, the cubes need be only of 10 cm size. Where concrete with aggregates larger than 40 mm size is required to be tested, the size of cubes may be increased keeping in view that generally the length of side of the cube should be about four times the maximum nominal size of aggregate in the concrete constituting the cube specimen.

Where cylinders are used for testing, minimum cylinder compressive strength required (on 15-cm diameter and 30-cm high cylinders) =0.8 compressive strength specified for 15-cm cubes.

The permissible working strength for concrete is adopted as one-third its cube crushing strength at 28 days. The result of 7-day test are often used in order not to hold up the work for 28 days pending results of tests. Concrete is deemed satisfactory if the 7-day cube strength is at least 2/3rd of the required 28 days strength.

For rapid-hardening cement minimum cube strength in a "works test" is taken as strength at 7 days.

Table II. Permissible Working (or Design) Stress for Cement Concrete

Grade of Concrete	In compression		In Bond-Average for Anchorage	Bearing Pressure on Full Area of Concrete	Direct Tensile Stress
	Due to Bending	Direct Compression			
M100—1:3:6 (M10)	30 kg/cm2 3.0 N/mm2	25 kg/cm2 2.5 N/mm2	4.0 kg/cm2 0.4 N/mm2	25 kg/cm2 2.5 N/mm2	12 kg/cm2 1.2 N/mm2
M150—1:2:4 (M15)	50 kg/cm2 5.0 N/mm2	40 kg/cm2 4.0 N/mm2	6.0 kg/cm2 0.6 N/mm2	37.5 kg/cm2 3.75 N/mm2	20 kg/cm2 2.0 N/mm2
M200–1:1.5:3 (M20)	70 kg/cm2 7.0 N/mm2	50 kg/cm2 5.0 N/mm2	8.0 kg/cm2 0.8 N/mm2	50 kg/cm2 5.0 N/mm2	28 kg/cm2 2.8 N/mm2
M250–1:1:2 (M25)	85 kg/cm2 8.5 N/mm2	60 kg/cm2 6.0 N/mm2	9.0 kg/cm2 0.9 N/mm2	62.5 kg/cm2 6.25 N/mm2	32 kg/cm2 3.2 N/mm2

(*i*) Table based on IS Code : 456–1964 and 1978. In the 1964 Code Metric units were used while in the 1978 Code SI units have been used and the values expressed in N/mm2 (10 kg/cm2 has been taken as 1 N/mm2), which are at 28 days after placing when properly cured and may be adopted for common works.

(*ii*) In the designation of a concrete mix, the letter M refers to the mix and the number to the specified 28-day works cube compressive strength of that mix expressed in kg/cm2 or N/mm2.

(*iii*) Grades of concrete lower than 1:2:4 shall not normally be used for reinforced concrete works.

(*iv*) For plain concrete permissible tensile stress in bending shall be taken as equal to the shear stress without reinforcement given in the tables under "Shear."

(*v*) Tensile stress in flexure for members cast with construction joints may be taken 0.8 of the tensile stress in bending.

(*vi*) In case of high yield strength deformed bars, the permissible bond stress my be increased by 40%. For bars in compression, the values of bond stress for bars in tension shall be increased by 25%.

(*vii*) According to 1978 Code, Direct Tensile Stress may be calculated from the expression :

$$f_t=\frac{F_t}{A_c+mA_s}$$

where : F_t is total tension on the member minus pretensions in steel, if any, before concreting ; A_c is cross-sectional area of the reinforcing steel ; m is modular ratio ; and A_s is cross-sectional area of reinforcing steel in tension. For members in direct tension, when full tension is taken by the reinforcement alone, the tenstle stress (f_t) on the effective concrete section shall be not greater than the values given in the table.

Ribbed-Torsteel

(See also under "Permissible Working Stress for Concrete" and "Permissible Working Stresses for Steel Reinforcement".)

Torsteel is produced in the form of cold-twisted deformed bars for concrete reinforcement, available in almost all the standard diameters ranging from 6 mm to 50 mm.

The following diameters may be used instead of the mild steel bars for tension and compression reinforcement to give the same strength :

For Tension Reinforcement

Mild steel – mm	8	10	12	16	20	22	25	28	32	36	40
Torsteel – mm	6*	8	10	12*	16	18	20	22	25	28	32

*Original spacing should be reduced by 10%

For Compression Reinforcement

Mild steel—mm	12	16	20	22	25	28	32	36	40
Torsteel – mm	10	14	2×12	18	20	22	2×18	2×20	32

Torsteel can be used without end hooks. When the above substitution tables are used, no extra lap or anchorage length over those necessary for the equivalent sizes of plain round bar is required even when no end hooks are used.

Torsteel costs about 10% more than tested mild steel.

There is growing tendency to use high yield point mild steel bars that have been cold drawn to increase their strength, but the difficulty of placing and bending these bars may out-weigh the initial saving in the weight of the steel required.

Shear in Beams

Shear at a section of a beam is the algebraic sum of all the external loads and reactions on any side of it ; maximum shear occurs near the supports. (See Section 3 under "Bending Moment and Deflection in Beams".) Tensile and compressive stresses which are set up due to bending are greatest in the flanges or extreme edges of the beam and decrease to zero at the neutral axis, and are normal to the section. The effect of shear stress is greatest in the web of the beam and is maximum at the neutral axis and decreases to zero at the extreme edges. Shear forces tend to cause diagonal cracks radiating from the top and at 45 deg. to the plane of the beam. These are steeper where the bending moments prevail and are more inclined where the shear forces are largest. Cracks due to bending moment are wider at the bottom and narrower at the top compression side, while cracks due to shear are widest in the region of the neutral axis and become thinner towards the upper and the lower edges of the beam.

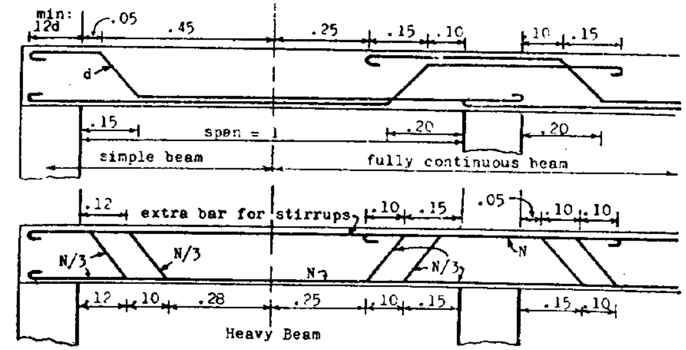
The intensity of shear stress S (nominal shear stress) at any cross-section in beams or slabs of uniform depth $= \frac{St}{bd}$

Where : St is the total shear force across the section (design loads) ; b is breadth of the beam (or web in the case of T or L beams) ; d is effective depth.

Shear reinforcement shall be provided to carry a shear equal to :

$$St-(Sp \times bd) = \text{say } S_R$$

Sp is the permissible shear stress (as given in the Table VII)—According to IS : 456—1978.



Shear reinforcement may be provided either :

(*a*) **By bent-up bars** : Some of the main tensile bars are bent-up from the tensile flange into the compression flange near the ends of the beam where shear stress is greatest. Bending moment which is maximum at the mid-span diminishes towards the supports and the number of main tensile bars required to resist bending moment may thus be reduced near the ends. The number and spacing of bars available for bending-up can be determined from bending moment diagram or as detailed hereafter.

At least one-quarter of the total tensile reinforcement must be carried straight beyond the face of the supports to provide adequate anchorage. In the case of continuous beams the bent-up bars are used for providing some or all of the tensile reinforcement for negative bending moments at the supports.

The angle of bend is about 30 deg. in shallow beams with d less than 1.5 b, and 45 deg. in deep beams. (The bend should not be less than 20 deg.) Bars must be bent-up through depth of beam equal to or greater than the lever arm jd. Shear resistance of bent bars (single bar or single group of parallel bars, all bent-up at the same cross-section) is=

$$S_R = A_W \times fs \sin \phi$$

All the inclined bars should be hooked at the ends as explained under "Bond and Anchorage". The bars should be bent-up as shown in the illustration which errs on the safe side. If the strength of bent-up bars is insufficient to take the shear force, stirrups must be added along the bent-up bar zone to carry the excese shear force. Stirrups should invariably be provided along the bent-up bar zone even though not actually necessary for calculated shear strength.

Where bent-up bars are provided, their contribution towards shear resistance shall not be more than half that of the total shear reinforcement.

Table III. Concrete Mix Proportions per 50 kg of cement

Mix Proportions	Total Quantity of Dry Aggregate*	Quantity of Water	Proportion of Fine Aggregate to Coarse Aggregate
	Kg	Litres	
1:5:10	800	60	Generally 1:2 but subject to an upper limit of 1:1.5 and a lower limit of 1:2.5.
1:4:8	625	45	
1:3:6	480	34	
1:2:4	350	32	
1:1.5:3	250	30	
1:1:2	160	27	

For an average grading of fine aggregate the proportions shall be 1:1.5 ; 1:2 ; and 1:2.5 for maximum size of aggregates 10 mm, 20 mm, and 40 mm, respectively.

*This is sum of the individual volumes of fine and coarse aggregate.

For given aggregates the cement content should be sufficient to provide adequate workability with a low water-cement ratio so that concrete can be completely compacted with the means available.

Notes for Table No. IV

When mild steel Grade II is used, the permissible stresses shall be 90 per cent of the permissible stresses given in col. (2), or the reinforcement shall be increased by 10 per cent of that required for Grade I steel.

Various types of steels are used for reinforcement, the most common type being mild steel. Other steels used are medium and high tensile steels, twisted bars either single or twin twisted, and fabrics consisting of fabricated rectangular meshes of small diameter mild steel and sometimes twisted steel bars and also of hard drawn steel wire. The shapes of the reinforcing bars may be round, square twisted or deformed. Mild steel round bars are the most commonly used. Use of deformed bars is on the increase.

A deformed bar is a bar of steel provided with lugs, ribs or deformations on the surface to minimize the slippage of the bar in concrete, such as "Torsteel".

The ultimate tensile strength of steel is the stress at which it fractures. Yield stress is about half the ultimate tensile strength for mild steel.

The permissible stress in concrete and reinforcement in buildings may exceed those given in the foregoing tables by 33 per cent if loads due to wind pressure (or earthquake) and temperature changes are combined with those due to dead, live and impact loads, in a design, so long as the stress in the reinforcement does not exceed 2600 kg/sq. cm.

Table IV. Permissible Working (or Design) Stress for Steel Reinforcement

Type of Stress in Steel Reinforcement	Mild Steel Grade I Plain or Deformed Bars	Medium Tensile Steel Plain or Deformed Bars Cold Twisted	High Yield Strength Deformed Bars and Ribbed Torsteel	Welded Wire Fabrics
1. Tension (f_s) in bars—tensile and shear				
(a) Up to and including 20 mm	1400 kg/cm^2 (140 N/mm^2)	Half the guaranteed yield stress subject to a max: of 1900 kg/cm^2 (190 N/mm^2)	2300 kg/cm^2 (230 N/mm^2)	2300 kg/cm^2 According to IS: 432-1966 (Part I) Mild Steel has been classified into two grades, grade I & grade II. Mild Steel having ultimate tensile strength of 4200 kg/cm^2 is classified as grade I steel, and that having ultimate tensile strength of 3800 kg/cm^2 as grade II steel. Steels having min: ultimate tensile strengths of 5500 kg/cm^2 and 10000 kg/cm^2 are classified as medium and high tensile strength steels.
(b) Over 20 mm	1300 kg/cm^2 (130 N/mm^2)		2300 kg/cm^2 (230 N/mm^2)	
2. Compression in column bars (f_s')	1300 kg/cm^2 (130 N/mm^2)	1300 kg/cm^2 (130 N/mm^2)	1900 kg/cm^2 (190 N/mm^2)	
3. Compression in bars in a beam or slab when the compressive resistance of the concrete is taken into account.	The calculated compressive stress in the surrounding concrete multiplied by 1.5 times the modular ratio or f_s', whichever is lower.			
4. Ditto. when the compressive resistance of the concrete is not taken into account				
(a) Up to and including 20 mm	1400 kg/cm^2 (140 N/mm^2)	Half the guaranteed yield stress subject to a maximum of 1900 kg/cm^2 (190 N/mm^2)	1900 kg/cm^2 (190 N/mm^2)	
(b) Over 20 mm	1300 kg/cm^2 (130 N/mm^2)		1900 kg/cm^2 (190 N/mm^2)	

Table based on IS Code : 456—1964 and 1978. In the 1964 Code Metric units were used while in the 1978 Code SI units have been used and the values expressed in N/mm^2 (10 kg/cm^2 has been taken as 1N/mm^2).

(*b*) **By Vertical Stirrups**: Stirrups are bars bent into U or rectangular shape, passing round the tensile reinforcement and the bars in the top flange. The stirrups are usually placed vertically and are either two-leg or four-leg.

Stirrups in beams serve the same purpose as binders do in columns. Stirrups should be anchored round the top bars and looped round the tensile bars, or securely fixed with wires to the bars round which they pass. Where no compression steel occurs, extra bars of small section (called hanger bars) should be provided at the top for the stirrups to be tied with.

Stirrups may be used either to take the full shear stress or together with the bent-up bars. Shear force to be resisted by stirrups :

$$S_R = \frac{A_w \times f_s \times d}{p} \quad \text{or} \quad p = \frac{A_w \times f_s \times d}{S_R}$$

When stirrups are used in conjunction with bent-up bars the total shear strength is the sum of the separate shear strengths of each. Where the shear stress exceeds shear resistance of bent bars, the shear taken by the bent bars is deducted from the total shear and the balance provided for by stirrups. Stirrups are placed along the length of a beam at intervals required by the varying intensity of the shear.

Spacing of Shear Reinforcement

The maximum spacing of shear reinforcement measured along the axis of the member shall not exceed 0.75 *d* for vertical stirrups and *d* for inclined stirrups at 45 deg., where *d* is the effective depth of the section under consideration. In no case shall the spacing exceed 45 cm.

(IS : 456—1978)

Stirrups should not be spaced further apart than a distance equal to the lever arm *jd*, and should preferably be spaced closer say, at a distance equal to or less than 1/2*d*, and with a minimum spacing of 1/4*d* at the ends which need not be less than 10 cm from practical considerations. Number of bars can be increased by using smaller diameters. No stirrups are generally needed in the centre-third of a span, but in heavy beams they should be continued with increase in spacing from the close spacing near the ends of the beam to the wider spacing at mid-span, which should not exceed 3/4*d*. This does not apply to beams under bridges carrying heavy rolling loads.

When compression reinforcement bars are provided, the stirrups shall not be spaced further apart than 12 times the diameter of these bars. This spacing should be reduced to 8 times the diameter if thin bars are used. Closer spacing is necessary as thin bars are susceptible to buckle and buldge out.

The stirrup bars are of size 5 to 10 mm diameter—maximum diameter should not be more than *d*/50. Size of stirrups should, where possible, be kept the same throughout the beam.

Distance from centre of support beyond which no web reinforcement is required = 1/2 effective span—(shear which beam can take without web reinforcement ÷ load per unit on the beam.)

(*c*) Shear force resisted by inclined stirrups or a series of bars bent-up at different cross-sections =

$$S_R = \frac{A_w \times f_s \times d}{p} (\sin\phi + \cos\phi)$$

$(\sin\phi + \cos\phi) = 1.41$ for 45° and 1.37 for 60°.

Where :

- A_w = total cross-sectional area of stirrup legs or bent-up bars within a distance p ;
- f_s = permissible tensile stress for shear reinforcement ;
- p = pitch or spacing of the stirrups or bent-up bars along the length of the member ;
- ϕ = angle between the inclined stirrup or bent-up bar and the axis of the member ;
- d = effective depth.

Shear in Thin Slabs. In thin slabs the shear stress is less than the permissible stress except under heavy loads, and no extra shear reinforcement is necessary. If half the tensile bars are bent-up and half carried through to the supports, the permissible shear stress for the section may be taken 4/3 of the permissible stress for the concrete.

See also under "General Design Principles for Slabs and Beams".

Table V. Shear Resistance Values of Bent-up Bars
in kg per each bar

Bar Dia.	f_s = 1400 kg/cm² Bend			Bar Dia.	f_s = 1300 kg/cm² Bend		
mm	ϕ = 30° H/V = 1.73	ϕ = 45° H/V = 1.00	ϕ = 60° H/V = 0.58	mm	ϕ 30° H/V = 1.73	ϕ − 45° H/V = 0.58	ϕ = 60° H/V = 0.58
10	550	778	952	22	2474	3494	4280
12	791	1120	1371	25	3196	4512	5526
16	1407	1990	2438	28	4010	5660	6932
18	1780	2519	3086	32	5234	7393	9054
20	2198	3110	3809	36	6627	9357	11460

Angle ϕ is between the bent-up bar and the axis of the beam. H is the horizontal and V the vertical projection of the bent part of the bar. Values are proportional to the permissible stress and also diameters squared of the bars.

Table VI. Shear Resistance Values of Vertical Stirrups

Values of S_R/d in kg/cm for Single Stirrups (two legged) for *fs* 1400 kg/sq. cm

Stirrup Spacing cm	Stirrup Dia. in mm					Stirrup Spacing cm	Stirrup Dia. in mm				
	5	6	8	10	12		5	6	8	10	12
5	109	158	281	440	633	20	27	40	70	110	158
7	78	113	201	314	452	25	22	32	56	88	127
8	68	99	176	275	396	30	18	26	47	73	106
10	55	79	141	220	317	35	16	23	40	63	90
12	46	66	117	183	264	40	14	20	35	55	79
14	39	56	100	157	226	45	12	18	31	49	70
16	34	49	88	137	198	50	10	16	28	44	63
18	31	44	78	122	176	55	9	14	25	40	57

S_R is strength of shear reinforcement to be provided for.

If S_R is 20,000 kg and *d* the effective depth 200 mm, the total shear value is 20,000/200 = 100. From table VI, 10 mm stirrups at 20 cm centres with a shear value of 110 are suitable.

Table VII. Permissible Shear Stress (S_p) in Concrete in Beams

Without Shear Reinforcement in kg/sq. cm

(Shear capacity of section is $S_p \times bd$)

$\frac{100 As^*}{bd}$	Grade of Concrete			$\frac{100 As}{bd}$	Grade of Concrete		
	M 150	M 200	M 250		M 150	M 200	M 250
0.20	2.0	2.0	2.1	1.10	3.8	4.0	4.2
0.25	2.2	2.2	2.3	1.20	4.0	4.1	4.3
0.30	2.4	2.4	2.5	1.25	4.0	4.2	4.4
0.40	2.7	2.7	2.8	1.30	4.1	4.3	4.4
0.50	2.9	3.0	3.1	1.40	4.2	4.4	4.5
0.60	3.1	3.2	3.3	1.60	4.3	4.6	4.7
0.70	3.3	3.4	3.5	1.70	4.4	4.7	4.8
0.75	3.4	3.5	3.6	1.90	4.4	4.8	5.0
0.80	3.4	3.6	3.7	2.30	4.4	5.1	5.3
0.90	3.6	3.7	3.9	2.50	4.4	5.1	5.5
1.00	3.7	3.9	4.0	3.00	4.4	5.1	5.7

*As is that area of longitudinal tensile reinforcement which continues at least one effective depth beyond the section being considered except at supports where the full area of tension reinforcement may be used. (Column is steel percentage.)

For solid slabs the permissible shear stress in concrete shall be K . Sp, where K has the value given below :

Overall depth of slab in mm	300 or more	275	250	225	200	175	150 or more
K	1.00	1.05	1.10	1.15	1.20	1.25	1.30

When the nominal shear stress S exceeds the permissible shear stress *Sp* given in Table VII shear reinforcement shall be provided in any of the following forms :

A beam to be safe against failure by shear must have sufficient thickness of concrete. Even where shear reinforcement is provided, the cross-section of the concrete should be large enough so that the nominal shear stress S in beams shall not exceed the maximum permissible shear stress given in the table below, and which shall not exceed half the value for slabs. If not, the section should be increased.

Maximum Permissible Shear Stress

Grade of Concrete	M 150	M 200	M 250
Max: Shear Stress—kg/cm^2	16	18	19

Bond and Anchorage

Bond is the surface resistance or adhesion between concrete and the steel bars embedded in it. One of the fundamental assumptions on which the theory of reinforced concrete is based is that there is a perfect bond or adhesion between the concrete and the steel and there is no slipping of the bar in the concrete under loading and both stretch or compress together, within the range of working stresses.

Bond between steel and concrete is destroyed if the steel is greasy or is painted, while, on the other hand, slight rusting of the steel may increase the bond. In order that the bar is not pulled out when subjected to tensile stress the bar has to have sufficient surface area in contact with the concrete. Therefore, it must be embedded a certain minimum length to develop its full resistance to the tensile pull. To work out this length of the bar to be embedded, the tensile stress in the bar is equated to the bond stress, and this will vary with the concrete mix and the permissible maximum working tensile stress in the steel. Bond stress varies along the embedded length, and the average bond stress is therefore considered in determining this length.

The permissible bond stresses for different mixes of concrete are given in Table II.

$$\frac{\pi d^2}{4} \times fs = \pi d \times L \times f_B \quad \text{or} \quad L = \frac{fs \times d}{4 \times f_B} \quad \text{or} \quad f_B = \frac{As \times fs}{L \times O}$$

Where :

d = dia. of bar ; fs = actual tensile stress in steel ; L = (development) length of bar to be embedded ; f_B = permissible average bond stress for anchorage ; O = perimeter of the bar.

Therefore, a reinforcement bar must be embedded a minimum length of $\frac{fs \times d}{4 \times f_B}$ in order to develop full bond stress. This will be greater with high tensile steel and lesser with richer mixes.

The bond length as calculated may be reduced if a hook or bend is provided on the end of the bar. The reduction for a standard right-angled bend is 8*d*, and for a semi-circular (U type) hook 16*d* (see below). The anchorage value of bends shall be taken as 4 times the dia. of the bar for each 45 deg. bend subject to a maximum of 16 times the dia. of the bar. The bond length is measured from the point of maximum bending moment (normally mid-span) towards support on each side. In the case of tensile bars bent up for shear, the bond length is measured from the point at which the bar cuts the neutral axis to the end of the bar.

Bond length for compression reinforcement :

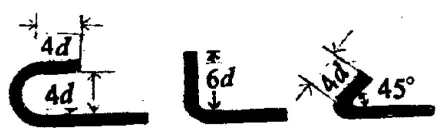
$$L = \frac{fc \times d}{5 \times f_B} \text{ with minimum of } 24d$$

fc is the actual compressive stress in the bar.

Compression steel is normally stressed only to $m \times$stress in the surrounding concrete and not to the full allowable stress.

No hook need be provided for a bar in compression (both in beams and columns), but a bend is desirable if the end of the bar is near an outer concrete face.

End Anchorage—Hooks. Ends of all tensile bars should be hooked to increase their grip or anchorage so as to ensure good bond against slipping in the concrete. In slabs hooks are not essential, but for any tensile bars which are curtailed and do not run through to the supports, hooks are necessary. The most common and the best form of anchorage is semi-circular hook as shown in the illustration which is the standard form of hook. (The dia. of the hook should never be less than 3*d*, which is shown as 4*d* in the illustration.) The length of the straight part of the bar beyond the end of the curve should be at least 4*d*, but not less than 75 mm. The total length of the bar required for the hook is 12*d* measured from the point the bar starts to bend.



Anchorage Value of Hooks and Bends for Bars in Tension

Bar Dia.—mm	6	8	10	12	16	18	20	22	25	28	32	36
Anchorage Value of Hook—cm	9.6	12.8	16.0	19.2	25.6	28.8	32.0	35.2	40.0	44.8	51.2	57.6

Anchorage value of 90 deg. bend is half the anchorage value of hooks.

Sometimes a 90 deg. bend will be required in positions such as where bars are placed one above the other. This bend should have a minimum

straight length of 6*d*, but prefer 10*d*. This is called a square hook or right-angled bend.

At the ends of simply supported beams all bars running through in the lower flange must extend to the centre of the support before hooks or bends begin. In the case of continuous beams, these bars must extend for 20*d* or 0.10L beyond the centre of support before hooks begin.

The hook will not be effective unless the concrete is thoroughly consolidated all round it with a cover of at least 2*d* or 25 mm (min.) on sides and 3*d* or 40 mm (min.) on top. Hooks should not be too close to the free surface of the concrete as they have a tendency to straighten out under stress and may burst off a thin concrete cover.

Deformed bars may be used without end anchorage (hooks) provided development length requirement is satisfied. Without hooks there is less congestion of reinforcement at ends or junctions of heavy beams.

The development (or bond) length of each bar of bundled bars shall be that for the individual bar, increased by 10 per cent for two bars in contact, 20 per cent for three bars in contact and 33 per cent for four bars in contact.

If the stress is beyond the premissible limits use bars of smaller diameter which will increase the superficial area of steel with the same cross-section. It is always preferable to use smaller diameter bars to ensure better bond stress.

As bond depends on shear and on the surface area of the reinforcement bars, bond stress is highest at points of maximum shear and also where the number of bars is minimum. In a simply supported beam, bond therefore, needs to be checked at the supports and at the points where tension bars are bent up.

In the case of simply supported beams some of the main tensile bars may be curtailed when no longer required for resisting the bending moment stresses which are reduced towards the supports, but at least one-quarter of the bars must run through in the lower flange. The bent-up bars should be continued horizontally a small further distance beyond the calculated point of curtailment and the top bent for anchorage, and should be hooked. It is preferable to bend the bars up for shear.

Laps in Bars. When a bar of sufficient length is not available, two bars with an overlap, firmly wired together, must be used. The length of overlapping (or splicing) in joints of bars is the same as the bond length :

(*a*) For bars in flexural tension :

$$L = \frac{fs \times d}{4 \times f_B} \text{ or } 30d, \text{ whichever is greater.}$$

(*b*) For bars in direct tension :

2L or 30*d*, whichever is greater.

The straight length of the lap shall not be less than 15*d* or 20 cm.

(*c*) For bars in compression :

$$L = \frac{fc \times d}{5 \times f_B} \text{ or } 24d, \text{ whichever is greater.}$$

The ends of the overlapping tensile bars should preferable be hooked in which case the length can be reduced as described earlier. The overlap should be arranged at a point where the tensile stress in the bars is the lowest, and laps should be staggered. Length of splice in column bars is also 24*d*. For works in water, take 30*d* for vertical bars and 40*d* for hooked circular rings.

Where steel fabrics are used in slabs, ends should be lapped 30 cm and wired together. Side laps are not necessary.

Lap splices shall not be used for bars larger than 36 mm ; for larger diameters bars may be welded. But welding of bars may be adopted in heavy constructions for eliminating overlaps. It allows the use of larger bars more closely spaced, and is also economical. The strength of the welded bar depends on the quality of the welds. A few rods selected at random should be tested from time to time to verify the joint strength. The welded joints should be staggered and kept nearabouts the points of contraflexure.

The forms of welds commonly employed are single V for bars below 30 mm diameter and double V for larger diameter bars. The ends of the bars are suitably shaped.

Cutting and Bending Bars. Bars up to 12 mm diameter can be cut by hand shears or chisel. Large bars require a simple hand lever machine bolted down to a base. For bars of 25 mm diameter and larger, oxyacetylene flame or power-operated machine may be used.

Bends in bars are made before the bars are placed in position in the formwork. Bars should perferably be bent cold. Usually hot bending should be limited to bars 32 mm diameter and above.

Small bars may be bent easily by the use of two stops or vertical iron pins driven at pre-determined positions in a thick piece of timber securely held in the ground or fixed on a table. A piece of water or gas tubing is used as a lever for bending which is slipped over the end of the bar. Simple hand operated direct lever bending machines may be used where available.

Concrete Coverings Outside Steel

(Exclusive of plaster or other decorative finish)

Thin slabs and walls	not less than dia. of bars—min : 15 mm
Beams—sides, top, bottom	not less than dia. of bars—min : 25 mm
„ —ends	twice dia. of bar—min : 25 mm
Columns	not less than dia. of bar—25 mm min: upto 200 mm side, 40 mm above 200 mm side
Piles	40 mm min: for sides
Water tanks	35 mm with rich concrete 40 mm with 1:2:4 mix
Foundations	40 mm
Marine works	for members totally immersed in sea water, the cover shall be 40 mm, and for members subject to sea spray, the cover of concrete shall be 50 mm more than for ordinary structures.

The above coverings are beyond stirrups and binders.

For all external works exposed to weather, for works against earth faces, and also for internal works where there are particularly corrosive conditions, the cover of the concrete should be increased to 1.5 times beyond those specified above.

It should be noted that dense and strong concrete is a better guarantee for the protection of the reinforcement against the harmful effects of chemicals than an increase in thickness of cover.

The equivalent dia. in the case of a square bar may be assumed to be about 1.125 times the side of the bar, and for a twin-twisted bar 1.5 times the actual dia. of one of the bars.

2. GENERAL DESIGN PRINCIPLES FOR RC SLABS & BEAMS

The *effective span* of a simply supported beam or slab shall be taken as clear span plus the effective depth of slab or beam, or centre to centre of supports, whichever is less. *Effective depth* is from top of concrete to the centre of tensile steel.

Depth (or Thickness) of Beams & Slabs

For normal cases the overall depth of a beam or slab shall not be less than the following fraction of the effective span : (IS Code)

Beams – up to 10 metres span :

Simply supported	1/20 of span
Continous	1/26 „
Cantilever	1/7 „

For spans above 10 metres, the values may be multiplied by 10/span in metres.

Slabs

Simply supported—spanning in one direction	1/30 „
„ „ „ two directions	1/35 „
Continuous „ one direction	1/35 „
„ „ „ two directions	1/40 „
Cantilever	1/12 „

This will obviate deflection.

In cantilever beams and slabs, the span may be taken as the effective overhang. In slabs spanning in two directions, the shorter of the two spans shall be taken to calculate the depth.

Bearings on Walls : Allow the following minimum bearings :—

Solid slabs : 10 cm.

Lintels : Equal to depth of lintel, with 15 cm minimum.

Beams : 20 cm for spans up to 3.5 metres, 30 cm for spans up to 5.5 metres, and 40 cm for spans up to 7.0 metres.

The bearings of the ends of a beam on a wall beyond a certain distance does not strengthen the beam.

DESIGN OF RC BEAMS

Practical Rules

1. The over-all depth of a singly reinforced rectangular beam shall be not less than 1/20 of the span unless shear and other considerations prevail. The greater the depth the less is the steel required and more economical is the beam, but there is a limit to it.

For adequate safety against deformation and cracking, International Code of Reinforced Concrete (1970) recommends a minimum beam depth for both rectangular and T-beams to be equal to or greater than 1/12 of the span length. For designing the beam depth may be assumed as 1/10 to 1/12 of the span for simply supported beams and 1/12 to 1/18 for continuous beams.

2. The breadth of a beam shall normally be 2/3 to 1/2 of the depth, but not less than 1/3 of the depth. A good rule for the breadth is to take 3/5th of the depth of the beam.

Where the span/breadth ratio exceeds 30 (beam whose length between adequate lateral restraints exceeds 30 times the breadth of its compression flange) and it is not practicable to support the compression flange laterally, the permissible compressive stress in the concrete due to bending shall be reduced by a factor (1.75—L/40B). Where L is the length of the beam lateral restraints, *i e.*—free span, and B is the breadth of the compression flange. The permissible stress in the compression reinforcement (where provided) shall also be reduced in the same ratio.

Slenderness Limits for Beams to Ensure Lateral Stability : A simply supported or continuous beam shall be so proportioned that the clear distance between the lateral restraints does not exceed 60 b or 250 b^2/d, whichever is less ; where d is the effective depth of the beam and b the breadth of the compression face midway between the lateral restraints. For a cantilever, the clear distance from the free end of the cantilever to the lateral restraint shall not exceed 25 b or 100 b^2/d, whichever is less.

(IS: 456—1978)

Beams are often used to support slabs which are mostly cast with beams as a monolithic construction and designed as T-beams. (See also under "Lateral Stability of Beams" in Section 10). This condition of lateral support can be deemed to be satisfied by a slab monolithic with beam near its compression flange provided the slab thickness is not less than 1/10 of the beam depth and adequate top and bottom reinforcement, suitably anchored, has been provided at the beam slab junction.

3. The clear distance from the corner of a beam or rib to the nearest longitudinal bar should be not less than 8 cm.

4. If depth of beam exceeds 60 cm, skin reinforcement, on both faces of web, in the form of longitudinal bars (minimum 12 bar dia. and spaced not more than 20 cm) should be provided. Such skin reinforcement on each face should be at least 0·05 per cent of gross web area.

5. The top surface of centering should be given a camber of 5 mm for every metre of span subject to a maximum of 35 mm, to allow for the initial deflection settlement.

Reinforcement

Minimum tensile reinforcement in beams shall be not less than 0·30 per cent where plain bars are used and 0·20 per cent where high-yield strength deformed bars are used, of the gross cross-sectional area of the beam. The maximum area of tension reinforcement shall not exceed 4 per cent. The area of beam being calculated as total cross-sectional area for rectangular beams and as area equal to overall depth multiplied by the width of the web in the case of T or L beam. At least one-fourth of the tension steel should be carried straight into the supports so as to provide anchorage.

Spacing of reinforcement bars

The horizontal clear space between two parallel main reinforcement bars shall be not less than the greatest of the following :

(*a*) The diameter of the bar if the diameters are equal, (*b*) the diameter of the larger bar if the diameters are unequal, (*c*) 5 mm more than the nominal maximum size of the coarse aggregate used in the concrete. Greater horizontal spacing than the minimum specified shall be provided where practicable.

This does not preclude the use of larger size of aggregates beyond the congested reinforcement in the same member ; the size of the coarse aggregates may be reduced around the congested reinforcement to comply with the above provisions.

Where needle or immersion vibrators are intended to be used, the horizontal distance between bars of a group may be reduced to two-thirds of the nominal maximum size of the coarse aggregate provided that sufficient space is left between groups of bars to enable the vibrator to be immersed.

The clear vertical space between two horizontal main reinforning bars shall normally be 15 mm, the maximum size of the coarse aggregate or the maximum size of the bar, whichever is the largest. Bars can also be placed one above the other without any space in-between. Steel space-bars may be introduced to maintain correct horizontal and vertical distances apart of the bars.

Main tensile reinforcement bars in beams shall be not less than 12 mm in diameter. Use as few different diameters as possible. Additional bars at the top corners have usually to be provided in beams for binding the stirrups for shear. Diameters of these bars may be 10 mm when not required to take any bending moments.

DESIGN OF FLOOR & ROOF SLABS

Practical Rules

The overall thickness of a slab shall be not less than 7.5 cm.

The top surface of centering shall be given a camber of 7 mm per metre of span subject to a maximum of 4.5 cm.

Reinforcement

(*i*) The minimum reinforcement in slabs in either direction shall be not less than 0·15 per cent of the gross cross-sectional area of the concrete, and which may be 0·12 per cent where high-yield strength deformed bars or welded wire fabric, are used.

(*ii*) The spacing of main tensile bars shall be not more than three times, and the distributing bars not more than five times the effective depth, or 450 mm, whichever is less. (This is according to the IS Code) Some engineers recommend spacing of the main bars to be twice the effective depth and not more than 300 mm, whichever is less, and for the distributing bars not more than four times the effective depth or 450 mm, whichever is less. Closer spacing is better.

(*iii*) The diameter of the main tensile reinforcement in slabs shall not exceed one-eighth of the total thickness of the slab and be not less than 6 mm in diameter, and bars of diameter greater than 18 mm shall not be generally used. 10 mm and 12 mm bars are the most convenient sizes for common works. Use as few different diameters as possible.

Some engineers recommend that max: bar dia. should not be greater than 1/10th of slab thickness, and spacing not more than 20 cm for main bars and 40 cm for secondary bars.

Bent-up bars

In simply supported single span slabs it is not normally necessary to bend up any bars. But in partially fixed conditions (which are the most common and occur where roof slabs are built into walls) every third bar shall be bent up. In slabs continous over two or more spans, alternate bars may be bent up, or equivalent separate reinforcement may be provided at the top of the supports for the negative moments. Bent-up bars are more economical. In large slabs separate reinforcement over the supports may be necessary.

Such bent-up bars shall extend a sufficient distance beyond the centre of the support to provide adequate bond. See diagram at page 8/11. The points at which some of the reinforcement is bent up for the negative bending moment at the supports depend on the points of contraflexure. Stirrups are not used in slabs.

It is not necessary to check shear or bond stress on a slab except with a superimposed load of over 2000 kg. per metre square. (See also under "Shear in Thin Slabs".) Shear stress in concrete is generally small and bond stress within allowable limits.

Distribution bars. Also called "binders", "cross bars", "temperature reinforcement", "transverse reinforcement" or, "secondary reinforcement". The object of these bars is to resist cracks due to temperature and shrinkage stresses, to assist in distributing local loading, and to take any bending stresses that may be developed. These bars are provided perpendicular to and on top of the main tensile bars in slabs spanning in one direction. Main and cross bars are tied together with 16 gauge soft iron wires. About 4.5 kg of wire is required for one tonne of bars. Well distributed steel reinforcement reduces the shrinkage and expansion of reinforced concrete.

The distributing bars shall be at least 25 per cent of the main tensile bars for roof slabs subject to much temperature changes, where additional bars shall also be provided at the top in thick slabs. In floors the transverse reinforcement may be only 10 to 15 per cent of the main bars. The diameter of such bars varies from 5 mm to 12 mm ; 6 mm and 10 mm bars are most convenient.

Distribution bars should not be hooked or bent as hooks will tend to localize the cracks and make the distribution steel non-effective.

Computing Bending Moments for Slabs and Beams

Slabs and beams generally have uniformly distributed loads for which the maximum bending moments may be taken as follows :—

(*a*) Single span support conditions :

(*i*) Simply supported	+WL/8 at the centre
(*ii*) Partially fixed	+WL/10 ,,
	—WL/24 at the supports
(*iii*) Fixed	+WL/12 at the centre
	—WL/12 at the support

(*b*) Beams continuous over three or more equal spans (approx.) : Twe spans may be considered approximately equal if they do not differ by more than 15 percent.

(*i*) At centre of end spans	+WL/10
(*ii*) Over support next to end support	—WL/10
(*iii*) At centre of interior spans	+WL/12
(*iv*) Over all other interior supports	—WL/12
(*v*) Over end supports (partially fixed)	—WL/24

+ sign is for the positive bending moments occuring at the bottom near mid-span, and — sign is for the negative bending moments at supports. The change in curvature of the beam takes place at the point of contra-flexure where the bending moment is zero. This occurs at distances from the inner supports of approximately L/4 for the inner spans and L/5 for the outer spans. (See Section 3.)

Bending moments should be calculated on effective spans.

Beams and slabs constructed monolithically with the supporting girders, beams or columns, are taken as continuous, and also the slabs spanning between T beams.

Derivation of Moment of Resistance of Singly Reinforced Beams and Slabs in Bending

Based on the following data for a 1:2:4 mix (ordinary concrete) :—

Permissible tensile stress in steel (*fs*)	...	1400 kg/cm^2
Permissible compressive stress in concrete (*fc*)	...	50 kg/cm^2
Modular ratio (*m*)	...	18.7
Percentage of steel (*p*)	...	0.71% *bd*
Neutral axis ratio (*kd*)	...	0.400 *d*
Lever arm of the resisting moment (*jd*)	...	0.867 *d*
Moment of Resistauce (MR)=Bending Moment (BM)		8.67 bd^2

Stress in concrete varies from zero at the neutral axis to *fc* at the top. Distance of the neutral axis from the top of the beam is *kd*, where *d* is effective depth and *k* is neutral axis factor. *d* is measured from the compression edge to the centre of the tensile steel. *j* is called lever arm factor.

(For a design to be based on other values of *fs*, *fc* or *m* see Table at page 8/28).

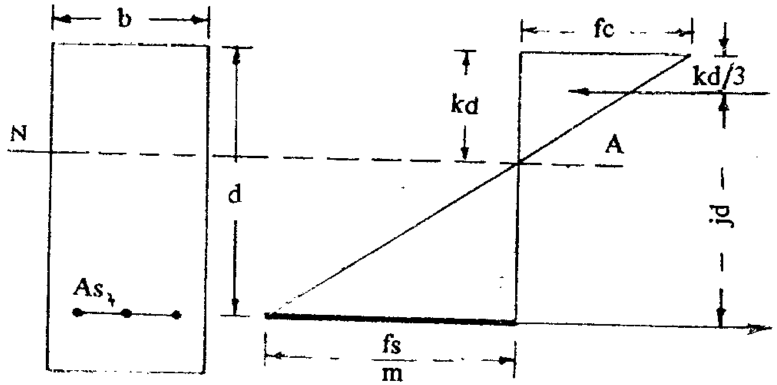
The above values are arrived at as explained on the next page,

In order to design a beam or a slab, the bending moment (BM) caused by external loads is equated to the moment of resistance (MR) of the beam or slab. The moment of resistance of a beam = either total compressive force or total tensile force, multiplied by lever arm (*jd*). The total compression and the total tension which are equal and act in opposite direction form a couple with lever arm *jd*, which is the distance between the centroid of the compression triangle and the centre of the reinforcing steel.

Moment of Inertia of RC Sections may be calculated by considering the entire cross-section by either (*i*) ignoring the reinforcement, or by (*ii*) including the reinforcement on the basis of the modular ratio. Whichever method is adopted it should be followed throughout the design for all the members.

The *stiffness factor* of a member of constant cross-section is the moment of inertia divided by the length of the member.

Working Formulae for Design of Rectangular Beams and Slabs



Stress Diagram

Position of Neutral Axis. Steel will take modular (*m*) times more stress than the concrete, and the strain at any point is proportional to the distance of that point from the neutral axis. The stresses vary from zero at the neutral axis to a maximum at the extreme edges. As such, the position of the neutral axis NA depends on the permissible stresses in concrete and steel and the modular ratio. Before the moment of resistance can be obtained, the position of neutral axis has to be ascertained. (It is

not always necessary to work out the neutral axis for simple designs. Table at page 8/28 gives the necessary data).

$$\frac{fs}{m} \times kd = fc \times (d - kd) \quad \text{from which}$$

$$k(d) = \frac{m \times fc}{m \times fc + fs} = \frac{18.7 \times 50}{18.7 \times 50 + 1400} = 0.400\ (d)$$

also $k = \sqrt{(2\,pm + p^2 + m^2)} - pm$

Total compressive force in the concrete is the area of the triangle above the neutral axis NA multiplied by the breadth (*b*) of the beam and $= 1/2\, fc \times kd \times b$.

This force acts at the centroid of the compression triangle, *i.e.*, at a depth of $1/3\, kd = \frac{0.400\, d}{3} = 0.133\, d$ from the top. Lever arm $jd = d - \frac{kd}{3}$

$$d = \sqrt{\frac{BM}{Q\,.\,b}}$$

Total tensile force $= fs \times As$ acting at depth *d* from the top edge to the centre of the reinforcing steel. *As* is the total sectional area of steel.

Total compressive force in concrete = total tensile force in steel for a balanced design for equilibrium, *i.e.*,

$$1/2\, fc \times kd \times b = fs \times As$$

Therefore total area of steel (*As*) in a balanced section :

$$As = \frac{fc \times kd \times b}{2\, fs} = \frac{50 \times 0.400\, d \times b}{2 \times 1400} = 0.0071\, bd \text{ or } 0.71\%\, bd.$$

This is referred to as the "economic percentage" of steel.

Moment of resistance due to concrete (in compression)
$= (1/2\, fc \times kd \times b)\, jd = (1/2 \times 50 \times 0.400\, d \times b)\, 0.867\, d = 8.67\, bd^2$.

This is equal to the moment of resistance due to steel (in tension), and
$= As \times fs \times jd = (p \times bd \times fs)\, jd$

$= (0.0071\, bd \times 1400)\, 0.867\, d = 8.67\, bd^2$ or $As = \frac{BM}{fsjd} = pbd$

Both the materials are stressed to their permissible working stresses.

Thus, the moment of resistance of any beam or slab with the above working stresses in concrete and steel is $8.67\, bd^2$ which is equated to the bending moment of the beam or slab under loading.

Procedure for design for singly reinforced beams and slabs :

For beams assume breadth about 1/20 of the span and depth 2× breadth. Calculate the maximum bending moment including an allowance for the weight of the beam itself. For slabs, assume one metre wide strip.

Effective depth *d* of any beam can be calculated by assuming breadth *b*

$$d = \sqrt{\frac{2\,BM}{fc\,k\,j\,b}} = \sqrt{\frac{BM}{8.67\,b}} \qquad \text{since } BM = 8.67\, bd^2$$

Checking an existing design. With given depth, breadth and area of reinforcement, the following formulae give the stresses in concrete and steel for a given bending moment BM :

$$fc=\frac{2\ BM}{kd\times b\times jd}\ ; \qquad fs=\frac{BM}{As\times jd}$$

Also

$$fc=\frac{fs\times k}{m\times(1-k)}\ ; \qquad fs=\frac{m\times fc\ (1-k)}{k}$$

Table VIII. Balanced Design Constants
for RCC Rectangular Sections

Stress in steel *fs* kg/cm²	Stress in concrete *fc* kg/cm²	Modular ratio *m*	Neutral axis *kd*	Lever arm *jd*	Steel percentage *p*% cm²	Moment of resistance Q×*bd*² kg/cm²
1260	50	18·7	0·426	0·858	0·845	9·14 bd^2
	70	13·3	0·425	0·858	0·180	12·78 bd^2
	85	11·0	0·426	0·858	1·436	15·53 bd^2
1300	50	18·7	0·418	0·861	0·804	8 95 bd^2
	70	13·3	0·417	0·861	1·110	12·30 bd^2
	85	11·0	0·418	0 861	1·366	15·30 bd^2
1400	50	18·7	0·400	0·867	0 714	8·67 bd^2
	70	13·3	0·399	0 867	0·997	12·10 bd^2
	85	11·0	0 400	0·862	1·214	14·74 bd^2
1900	50	18·7	0·330	0·890	0·434	7·30 bd^2
	70	13·3	0·329	0 890	0·606	10·25 bd^2
	85	11·0	0·330	0·890	0·720	12·20 bd^2
2100	50	18·7	0 308	0·897	0·367	6·91 bd^2
	70	13·3	0·307	0·897	0·512	9·64 bd_2
	85	11·0	0·308	0·897	0·623	11·73 bd^2
2300	50	18·7	0·289	0·904	0·314	6·50 bd^2
	70	13·3	0·288	0·904	0·438	9·11 bd^2
	85	11·0	0·289	0·904	0·534	11·10 bd^2

Q is bending moment co-efficient or factor

$$d=\sqrt{\frac{BM}{Q\ .\ b}}$$ See also Table at page 8/40

Data for RCC Beams Reinforced for Tension per cm width of beam

Mix—1:2:4 Stresses : Concrete—50 kg/sq. cm,
Steel—1400 kg/sq. cm, m—18.7

Overall Depth cm	Effective Depth cm	Moment of Resistance per cm width kg-metre	Tensile Reinforcement per cm width sq. cm	Capable of Shear kg
14	11	10.5	0.078	42
15	12	12.5	0.086	45
16	13	14.7	0.093	48
17	14	17.0	0.100	51
18	15	19.6	0.107	56
19	16	22.3	0.115	59
20	17	25.1	0.121	62
21	18	28.2	0.122	65
22	19	31.4	0.135	68
23	20	34.8	0.143	71
24	21	38.4	0.150	74
25	22	42.1	0.157	77
26	23	46.0	0.164	81
28	24	50.1	0.172	87
29	25	54.4	0.179	90
30	26	58.8	0.185	93
31	27	62.0	0.193	96
32	28	68.2	0.200	102
34	30	78.3	0.215	109
36	32	89.1	0.230	115
38	34	100.0	0.243	112
40	36	112.7	0.256	128
42	38	125.6	0.270	134
44	40	139.2	0.286	141
46	42	153.4	0.300	147
48	44	168.4	0.315	154
50	46	184.0	0.329	160
52	48	200.4	0.342	166
54	50	217.5	0.357	173
56	52	235.2	0.371	179
58	54	253.7	0.385	186
60	56	272.8	0.400	192
62	58	292.7	0.414	198
64	60	313.2	0.430	205
69	65	367.6	0.465	221
74	70	426.3	0.500	244
79	75	489.4	0.536	261

Shear vulues based on IS Code : 456—1978

Table X. Safe Uniformly Distributed Suprimposed Loads on RCC Rectangular Beams Simply Supported per centimetre width in kg/metre

Effective depth d cm	Effective-Span in Metres										
	2.0	2.25	2.50	2.75	3.0	3.5	4.0	4.5	5.0	5.5	6.0
20	64.8	50.2	39.7	32.0	26.1	17.9	12.6	8.9	6.3	4.4	2.9
25	102.7	79.9	63.6	51.5	42.3	29.5	21.2	15.5	11.4	8.3	6.5
30	149.4	116·5	93.0	75.6	62.4	43.9	31.9	23.7	17.8	13.4	10.2
35	204.7	159.9	128.0	104.3	86.3	61.2	44.9	33.7	25.7	19.7	15.2
40	268.8	210.3	168.5	137.6	141.1	81.3	60.0	45.0	34.9	27.1	21.3
45	341.5	267.5	214.7	175.6	145.8	104.2	77.3	58.8	45.6	35.7	28.4
50	423.0	331.6	266.3	218.1	181.3	130.0	96.7	73.9	57.6	45.4	36.3
55	513.1	402.6	323.6	265.1	220.7	158.6	118.4	90.7	71.0	56.3	45.2
60	612.0	480.8	386.5	316.9	264.0	190.1	142.2	109.3	85.8	68.3	55.1
65	719.3	565.2	454.9	373.3	311.1	224.4	168.2	129.6	102.0	81.4	66.0
70	835.3	656.7	529.2	433.2	362.2	261.5	186.4	151.6	159.6	95.8	77.8
75	960.7	755.3	608.4	498.6	417.0	301.6	226.7	175.3	138.6	111.2	90.6

For other details of beams see Table at page 8/29.

Mix—1:2:4

Stresses Concrete—50 kg/sq. cm,

Steel—1400 kg/sq. cm, m—18.7

Table XI. Minimum Overall Thickness of Slab Required for Different Spans in centimetres

Support conditions	Span in Metres												
	2·0	2·5	3·0	3·5	4·0	4·5	5·0	5·5	6·0	6·5	7·0	7·5	8.0
Simply supported slabs spanning in one direction	10	10	10	12	15	15	18	20	20	22	25	25	30
Simply supported slabs spanning in two direction	10	10	10	10	12	15	15	18	18	20	22	22	25
Continuous slabs spanning in one direction	10	10	10	10	12	15	15	18	18	20	22	22	25
Continuous slabs spanning in two directions	10	10	10	10	10	12	15	15	15	18	18	20	20
Cantilever slabs	18	22	25	30	35	40	45	50	50	55	60	65	70
Flat Slabs													
End panels without drops	15	15	15	15	15	15	18	18	20	22	22	25	25
Interior panels, fully continuous, without drops, and for end panels with drops	15	15	15	15	15	15	15	18	18	20	20	22	25
Interior panels, fully continuous, with drops	15	15	15	15	15	15	15	15	15	18	18	20	20

Table XII. Minimum Overall Depth of Beam Required for Different Spans in centimetres

Support conditions	Span in Metres										
	2.5	3	4	5	6	7	8	9	10	11	12	13
Simply supported beams	12.5	15	20	25	30	35	40	45	50	55	60	65
Continuous beams	10.0	12	16	20	24	28	32	36	40	44	48	52
Contilever beams	25.0	30	40	50	60	70	80	90	100	110	120	130

If the steel percentage is less than the economic percentage value, the section will be weaker in tension than in compression and can take only the following moments of resistance (MR) :—(1400, 50, 18.7)

Steel percentage	0.714	0.60	0.40	0.20
MR/bd^2	8.67	7.35	5.00	2.58

Where the steel percentage is greater than the economic percentage value, the section will be weaker in compression than in tension and can take only the following moments of resistance (MR) :—

Steel percentage	0.80	1.00	1.20	1.40
MR/bd^2	9.00	9.57	10.08	10.60

As the steel percentage increases, the neutral axis is lowered. While, by adding more steel above the economic percentage it is possible to increase the moment of resistance, but it is uneconomical owing to the slower rate of increase.

Doubly Reinforced Beams are beams reinforced in compression as well as in tension. Such beams are used where the depth is restricted due to design considerations or when the bending moment exceeds the moment of resistance of a balanced section. It is not economical to reinforce a beam in compression as the compression steel is not stressed fully to its permissible working stress but only to a value—$1.5 \times$ modular ratio (m) $\times$ stress in concrete around the steel, which is about $m \times 2/3\, fc$ only.

In pactice, beams with compression steel have to be used in the case of rectangular beams, lintols, braces and walls of storage reservoirs (where bending moment reverses in sign according to loading conditions). T-beams, when continuous, behave as rectangular beams at supports, and when singly reinforced require considerable depth to take up the bending moment. Steel for compression being automatically available from the two spans adjacent to a support, a doubly reinforced rectangular beam is more suitable.

A doubly reinforced beam may be designed by the following simple and approximate method :

Stress in concrete at the level of compression steel at $(kd—a)$ from NA

$$=\frac{(kd-a)}{kd}\times fc$$

and stress in compression steel $=1.5\ m\times\frac{(kd-a)}{kd}\times fc.$

The stress in compression steel so found should not exceed the permissible value (fs') specified in Table II.

Stress in tensile steel $=m\times fc\left(\frac{d-kd}{kd}\right).$

It is desirable that the increase in moment carrying capacity of beams by providing additional compressive steel is limited to 50 per cent. The percentage of steel in compression should preferably not exceed 4 per cent of the full cross-sectional area of the beam.

RCC Rectangular Beams Reinforced for Tension

per cm width of beam — Ribbed Torsteel

Overall Depth of Beam cm	Concrete mix 1:2:4 Moment of Resistance Kg-metre	Concrete mix 1:2:4 Tensile Reinforcement Sq. cm	Concrete mix 1:2:4 Shear Capacity Kg	Concrete mix 1:1.5:3 Moment of Resistance Kg-metre	Concrete mix 1:1.5:3 Tensile Reinforcement Sq. cm	Concrete mix 1:1.5:3 Shear Capacity Kg
20	38	0.135	64	51	0.180	72
25	66	0.175	82	88	0.235	90
30	101	0.215	99	135	0.254	105
35	142	0.260	119	192	0.345	130
40	194	0.300	136	259	0.400	148
45	252	0.340	153	336	0.451	167
50	317	0.385	170	423	0.501	185
60	470	0.465	204	627	0.602	222
70	653	0.550	238	871	0.735	259
80	866	0.635	272	1155	0.845	296
90	1109	0.715	306	1479	0.955	333
100	1382	0.800	340	1843	1.065	370

Based on Ribbed Torsteel Handbook — Ribbed Torsteel—Grade Tor 40

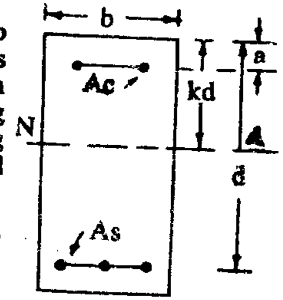
(i) Shear capacity is according to IS Code 456—1978 ; Clear cover to main reinforcement : 2.5 cm.

(iii) For design moment of resistance less or more than the tabulated value, the area of steel can be reduced or increased in direct proportion.

Additional Moment Carrying Capacity by Compression Steel
Ribbed Torsteel (tonne-metre) Concrete Mix 1:2:4

Overall Depth —cm	Ac—sq. cm	2.00	4.00	6.00	8.00	10.00	12.00	14.00	16.00	18.00	20.00	25.00	30.00	35.00	40.00	50.00
	As'— sq. cm	1.75	3.50	5.25	7.00	8.75	10.50	12.25	14.00	15.75	17.50	21.87	26.25	30.62	35.00	43.75
20		0.42	0.84	1.26	1.68	2.10	2.52	2.94	3.36	3.78	4.20	5.25	6.30	7.35	8.40	10.5
25		0.59	1.19	1.78	2.38	2.97	3.57	4.16	4.76	5.35	5.95	7.43	8.92	10.4	11.9	14.9
30		0.77	1.54	2.31	3.08	3.85	4.62	5.39	6.16	6.93	7.70	9.62	11.5	13.5	15.4	19.2
35		0.94	1.89	2.83	3.78	4.72	5.67	6.61	7.56	8.50	9.45	11.8	14.2	16.5	18.9	23.6
40		1.12	2.24	3.36	4.48	5.60	6.72	7.84	8.96	10.1	11.2	14.0	16.8	19.6	22.4	28.2
45		1.29	2.59	3.88	5.18	6.47	7.77	9.06	10.4	11.6	12.9	16.2	19.4	22.7	25.9	32.4
50		1.47	2.94	4.41	5.88	7.35	8.82	10.3	11.8	13.2	14.7	18.4	22.0	25.7	29.4	36.7
60		1.82	3.64	5.46	7.28	9.10	10·9	12.7	14.6	16.4	18.2	22.7	27.3	31.8	36.4	45.4
70		2.17	4.34	6.51	8.68	10.8	13.0	15.2	17.4	19.5	21.7	27.1	32.5	38.0	43.0	54.2
80		2.52	5·04	7.56	10.1	12.6	15.1	17.6	20.2	22.7	25.2	31.5	37.8	44.1	50.4	63.0
90		2.87	5.74	8.61	11.5	14.3	17.2	20.1	23.0	25.8	28.7	35.9	43.0	50.2	57.4	71.7
100		3.22	6.44	9.66	12.9	16.1	19.3	22.5	25.8	29.0	32.2	40.2	48.3	56.3	64.4	80.5

Based on Ribbed Torsteel Handbook Ribbed Torsteel—Grade Tor 40
(i) Cover to main reinforcement : 2.5 cm ; (ii) Ac : Compression steel ; (iii) As' : Additional Tensile Steel ;
(iv) Total Tensile steel in beam is the sum of Tensile steel obtained ;
(v) Lever arm for compression steel = Overall depth minus 8 cm.



Let the total bending moment the beam has to be designed for be = BM. Size of the beam is fixed as bd. Find out the bending moment which this beam singly reinforced can carry with the corresponding economic percentage of tensile steel. Let this bending moment be B′M′. (B′M′ = 8.67 bd^2 and tensile steel 0.0071 bd, for a 1:2:4 mix as worked out earlier.)

The balance bending moment for which extra tensile and compressive reinforcement is required = (BM − B′M′.)

Additional tensile reinforcement for the (BM − B′M′) bending moment =

$$\frac{BM - B'M'}{fs \times (d-a)} = As'$$

Total tensile reinforcement As

$$= As' + (p \times bd) : p \text{ is economic percentage of steel.}$$

Compressive reinforcement at top for the balance bending moment will be :—

$$Ac = \frac{m \times As'(d-kd)}{(1.5\,m-1)\times(kd-a)}\ ;\ \text{or approx: } Ac = \frac{As'(d-kd)}{(kd-a)}$$

$$As' = \frac{(1.5\,m-1)\times Ac \times (kd-a)}{m \times (d-kd)}$$

N.A. can be determined by equating the moments of the area of concrete and equivalent concrete area of compressive steel to the moment of equivalent concrete area of tensile steel, about the N.A.

$$1/2\, b \times kd^2 + (1.5\,\text{m}-1)\, As\,(kd-a) = m \times As\,(d-kd)$$

(The term $(m-1)$ is taken instead of m because the area of steel added results in displacing an equal area of concrete.)

$(1.5\,m)$ Ac is the equivalent concrete area to compressive steel.

Checking a design

Moment of resistance of the beam due to compression concrete and steel =

$$1/2\, fc \times kd \times b\,(d-a) + Ac \times (1.5\,m-1) \times fc \times (d-a) \times \frac{(kd-a)}{kd}$$

Moment of resistance of the beam due to tensile steel = $fs \times As \times (d-a)$. Both the moments must be equal to maintain equilibrium.

The compression bars must be anchored to the tensile bars by means of stirrups or binders to guard against their buckling, even if no stirrups are required for shear.

If steel is crowded into a concrete member, it will be difficult to ensure sound concreting. Owing to the difficulty of compacting the concrete properly due to the mass of steel, air pockets may form causing lack of continuity in the beam and reducing the bond or adherence between the concrete and steel.

Value of the ratio Ac/As′

Stress in steel *fs* kg/cm²	Stress in concrete *fc* kg/cm²	Ac is area of compression reinforcement As′ is area of additional tensile reinforcement			
		a/d			
		0·05	0·10	0·15	0·20
	50	1·19	1·38	1·66	2·07
1400	70	1·20	1·40	1·68	2·11
	85	1·22	1·42	1·70	2·13
	50	2·06	2·61	3·55	5·54
2300	70	2·09	2·65	3·60	5·63
	85	2·12	2·68	3·64	5·69

Bending moment co-efficient Q for *fs* 1400 and 1:2:4 mix, $a/d=0{\cdot}10$, for equal As and Ac.

As=Ac-per cent	1.0	1.5	2.0	3.0	4.0
Q-kg/cm²	12.90	17.90	21.40	28.45	35.90

Steel Beam Theory. A design in which the contribution of concrete to the compressive strength is neglected and in which the resistance moment is based on the tension and compression steel only, is sometimes know as design by steel beam theory. Here a beam has equal steel at top and bottom.

Slabs Supported on Four Sides

Slabs with reinforcement in two directions (two-way reinforced slabs) are more economical than one-way reinforced slabs, when the slabs are square or of a rectangular shape in which the longer side does not exceed the shorter side by about 30 per cent.

Where the length of a slab is less than twice its breadth and is supported on all the four sides it may be designed with tensile reinforcement in both the directions at right angles. Maximum bending moment is worked out on a middle strip of unit width on each span, according to whether the ends are simply supported or built into the walls (semi-fixed), or cast monolithically with the supporting beams (continuous). This bending moment is multiplied by the appropriate factor given in the following table to obtain the proportion of the total bending moment to be designed for in each direction.

Long span/short span	1.0	1.2	1.3	1.4	1.5	1.75	2.0	2.5	3.0
Across long span	.50	.32	.25	.20	.16	.10	0.6	.03	.02
Across short span	.50	.68	.75	.80	.84	.90	.94	.97	.98

Reinforcement for negative bending moments will be 4/5th and 2/3rd of the positive reinforcements across each mid-span where the slabs are partially fixed or continuous. Extra reinforcement should be provided at the corners of the panels where they are fixed, equivalent to half the maximum positive bending moment reinforcement in the middle strip, in the form of a mesh near both the top and bottom faces of the slab for a distance of one-fifth of the long span in both directions from the corners.

(This is an approximate method good for practical purposes).

The thickness of the slab is based on the greater bending moment which will be on the shorter span and it should be equal to short span/35 from stiffness point of view. Long span reinforcement is placed above the short span reinforcement. Alternate bars can be bent up at 1/7th span from the centre of the supports. No distribution reinforcement is provided. In the central half of the spans bars may be spaced according to the calculated maximum bending moment but in the outer portions, the spacings may be increased to 1.5 times.

Load Reactions on Supporting Beams from Slabs Supported on Four Sides :

Load on each short span is taken as on an area formed by the intersection of 45 deg. lines from both corners (of a short span) and that on the long span equal to the load on half of the (long) span minus the load on the short span, as follows :—

Load on short span beam : $W_s = \dfrac{wL_s^2}{4}$

Load on long span beam : $W_L = \dfrac{wL_sL_L}{2} - W_s$

L_L is long span, L_s is short span, w is the load per unit area.

Circular Slabs supported on perimeter, two-way reinforced :—

The maximum bending moment at the centre on a strip of unit width may be taken $wd^2/32$ for simply supported, $wd^2/40$ for partially fixed, and $wd^2/48$ for fixed ends. w is the load per unit area and d is the diameter. (Some engineers use slightly different formulae.) Knowing the bending moments, the depth of the slab and the area of reinforcement required can be calculated as usual. The bending moment is maximum at the centre and is reduced towards the circumference, therefore, reinforcement may also be reduced towards the circumference. The reinforcement is usually provided in the form of a mesh in two perpendicular directions having equal area of cross-section in both the directions. For fixed slabs, the negative bending moments at the supports may be taken the same as the positive bending moments and reinforcement provided radially at the top.

Moment of Resistance per Metre Width	Thickness of Slab	Effective Depth	DATA FOR RCC SLABS — Safe Distributed Superimposed Loads in kg/sq. metre (excluding weight of slab and other dead loads) on Various Spans in Metres.						
kg-m	cm	cm	1·5 m	2·0 m	2·5 m	3·0 m	3·5 m	4·0 m	4·5 m
200	7	5	542	242	—				
300	8	6	873	408	192	—			
425	9	7	1296	634	329	164	—		
560	10	8	1754	880	480	260	125	—	
700	11	9	2236	1146	636	366	196	—	
870	12	10	—	1452	822	485	282	147	—
1050	13	11		1788	1028	621	373	213	103
1250	14	12		2164	1264	774	484	289	159
1470	15	13		—	1520	950	600	375	220
1700	16	14		—	1796	1116	726	466	286
1960	17	15			2092	1332	872	572	372
2225	18	16			—	1548	1028	668	458
2500	19	17			—	1766	1148	794	544
3100	22	19			—	2232	1492	1022	692
3800	24	21				—	1904	1324	924
5000	28	24					—	1828	1298
6200	31	27						—	1706

(*i*) For calculating moment of resistance, dead weight of slab at 24 kg per one cm thickness of one metre square area, or 2400 kg per cubic metre, has been added to the tabulated loads. (See under "Loadings for Design of Roofs" in Section 11).

(*ii*) Where ribbed-torsteel is proposed to be used, the diameter and spacings of the bars shall be reduced in accordance with the notes gives under "Use of Ribbed—Torsteel".

(*iii*) Where Grade II steel is to be used, add 10 per cent extra tensile reinforcement.

(*iv*) For deriving safe loads in continuous and semi-continuous spans, the values shown in the table may be increased by 50 and 25 per cent respectively, and for cantilevers it will be one-fourth.

Mix 1 : 2 : 4, Working Stresses : Steel 1400 kg/sq. cm, Concrete 50 kg/sq. cm, m 18.7			REINFORCEMENT — Area of Tensile Steel per Metre Width	Arrangement of — Main Tensile Steel: Bar Dia.	Main Tensile Steel: Spacing	Transverse Steel: Bar Dia.	Transverse Steel: Spacing	Quantity of Steel per 100 sq. metre of slab area
5·0 m	5·5 m	6·0 m	sq. cm	mm	cm	mm	cm	tonnes
			3.57	8	14·0	5	20	0.391
			4.28	8	11·5	5	20	0.469
			5.00	8	10·0	6	20	0.545
			5.72	10	13·5	6	19	0·633
			6.45	10	12·0	6	17	0·710
			7.15	10	11·0	6	15.5	0.776
—			7.75	10	10·0	6	14.5	0.845
—			8.58	12	13·0	8	24.0	0.932
110			9.30	12	12·0	8	22.0	1.012
160			10.00	12	11·0	8	21.0	1.095
222			10·70	12	10·5	8	19.5	1.153
280	—		11.45	12	9·5	8	18.5	1.263
344	180	—	12.15	12	9·0	8	17.5	1.333
467	292	162	13.57	16	14·5	8	14·5	1.486
634	424	269	15.00	16	13·0	8	12.5	1.635
928	648	438	17.14	16	11·5	10	10.5	1.876
1236	896	636	19.28	16	10.3	10	13.5	2.111

T-Beams and L-Beams

The T-beam is the commonest type of reinforced concrete slab and beam monolithic construction. The part of the floor projecting downwards is called *stem, rib or web* of the T, and the slab portion the *flange* of the T-beam. When there is a flange only on one side of the shape of an inverted L, it is called an ell-beam.

A floor usually consists of main and secondary beams. Between two main beams are secondary beams at right angles to the main beams and are usually spaced 2 to 5 metres, framing into and supported by the main beams. In order to make bending moments in end spans approximately equal to the moments in the interior spans, the end spans should be made about 0.8 to 0.9 of the interior spans. The advantage of such a section is: the horizuntal flange of the concrete supplies resistance to compression while the vertical rib gives depth and lever arm. The arrangement of bars can then be with equal steel in top and bottom at the supports. The top bars in the slab will pass over the bars in the secondary beams.

Moment of Resistance Factor, BM/bd² for Singly Reinforced (under Reinforced) Rectangular Sections—Kg/cm²

Steel percentage cm²	Stress in Concrete—50 kg/cm² Stress in Steel—kg/cm² 1300	1400	1900	2300	Stress in Concrete—70 kg/cm² Stress in Steel—kg/cm² 1300	1400	1900	2300	Stress in Concrete—85 kg/cm² Stress in Steel—kg/cm² 1300	1400	1900	2300
0.20	2.39	2.58	3.50	4.23	2.42	2.61	3.54	4.28	2.44	2.62	3.56	4.31
0.25	2.97	3.19	4.33	5.25	3.01	3.23	4.39	5.32	3.02	3.25	4.42	5.35
0.31	3.64	3.92	5.33	**6.45**	3.69	3.93	5.40	6.54	3.78	4.01	5.44	6.58
0.35	4.09	4.41	5.98	—	4.15	4.47	6.07	7.35	4.19	4.50	6.12	7.40
0.40	4.65	5.00	6.79		4.72	5.08	6.90	8.35	4.76	5.12	6.95	8.42
0.43	4.98	5.36	**7.28**		5.06	5.45	7.39	8.95	5.10	5.49	7.46	9.02
0.44	5.09	5.48	—		5.17	5.57	7.56	**9.11**	5.21	5.61	7.62	9.22
0.50	5.74	6.19			5.84	6.29	8.54	—	5.89	6.34	8.61	10.4
0.53	6.07	6.54			6.17	6.65	9.02		6.12	6.59	8.94	**11.1**
0.55	6.29	6.77			6.40	6.89	9.35		6.46	6.90	9.44	—
0.60	6.83	7.35			6.95	7.48	**10.2**		7.01	7.55	10.3	
0.65	7.36	7.93			7.50	8.07	—		7.57	8.10	11.1	
0.71	8.00	**8.67**			8.15	8.78			8.23	8.87	12.0	
0.72	8.11	—			8.26	8.90			8.34	8.98	**12.2**	
0.75	8.43	—			8.59	9.25			8.67	9.34	—	
0.80	**8.95**	8.90			9.13	9.83			9.22	9.93		
0.85	—	—			9.66	10.4			9.77	10.5		
0.90		—			10.2	11.0			10.3	11.1		
0.95		—			10.7	11.6			10.8	11.7		
0.99		—			11.2	**12.0**			11.3	12.2		
1.05		9.50			11.8	—			11.9	12.9		
1.10		—			**12.3**				12.5	13.4		
1.15					—				13.0	14.0		
1.21									13.6	**14.7**		

Based on "Design Aids to IS : 456—1978". Figures in thick printing are "balanced" value of Q.

See also Table at page 8/28

T-Beams with Tensile Reinforcement Ribbed Torsteel

Overall Depth cm		Concrete Mix 1:2:4 Flange Thickness—cm 10	12	15	18	20	Concrete Mix 1:1.5:3 Flange Thickness—cm 10	12	15	18	20
25	BM	4.85	4.85	4.85	4.85	4.85	6.45	6.45	6.45	6.45	6.45
	As	14.9	14.9	14.9	14.9	14.9	19.9	19.9	19.9	19.9	19.9
30	BM	7.95	7.95	7.95	7.95	7.95	10.6	10.6	10.6	10.6	10.6
	As	19.2	19.2	19.2	19.2	19.2	25.6	25.6	25.6	25.6	25.6
35	BM	10.2	11.8	11.8	11.8	11.8	13.7	15.7	15.7	15.7	15.7
	As	19.4	23.3	23.3	23.3	23.3	25.9	31.1	31.1	31.1	31.1
40	BM	12.5	14.4	16.3	16.3	16.3	16.7	19.2	21.8	21.8	21.8
	As	19.4	23.3	27.5	27.5	27.5	25.9	31.1	36.7	36.7	36.7
45	BM	14.7	17.1	20.3	21.6	21.6	19.6	22.8	27.1	28.8	28.8
	As	19.4	23.3	29.1	31.5	31.5	25.9	31.1	38.8	42.0	42.0
50	BM	16.9	19.7	23.7	27.2	28.6	22.6	26.3	31.6	36.3	38.2
	As	19.4	23.3	29.1	34.8	37.0	25.9	31.1	38.8	46.5	49.4
60	BM	21.4	25.2	30.3	35.2	38.3	28.5	33.6	40.5	47.0	51.0
	As	19.4	23.3	29.1	34.8	38.7	25.9	31.1	38.8	46.5	51.5
70	BM	25.8	30.4	37.0	43.2	47.2	34.4	40.5	49.4	57.6	63.0
	As	19.4	23.3	29.1	34.8	38.7	25.9	31.1	38.8	46.5	51.5
80	BM	30.2	35.8	43.6	51.3	56.0	40.2	47.8	58.1	68.5	74.6
	As	19.4	23.3	29.1	34.8	38.7	25.9	31.1	38.8	46.5	51.5
90	BM	34.8	41.2	50.5	59.4	65.0	46.5	55.0	67.5	79.3	86.6
	As	19.4	23.3	29.1	34.8	38.7	25.9	31.1	38.8	46.5	51.5
100	BM	39.2	46.6	57.0	67.4	74.0	52.3	62.1	76.0	90.0	98.6
	As	19.4	23.3	29.1	34.8	38.7	25.9	31.1	38.8	46.5	51.5

Based on Ribbed Torsteel Handbook
(i) BM : maximum moment of resistance capacity of T-beam with 1 metre wide flange at working load (tonne-metre); (ii) As : corresponding area of steel (sq. cm); (iii) Clear cover to main reinforcement : 2.5 cm; (iv) The max: distance of the centroid of tensile steel has been taken as 7 cm; (v) Compression in web has been neglected; (vi) For design moment of resistance less or more than the tabulated value, the area of steel can be reduced or increased in direct proportion.

RCC T-Beam Size and Reinforcement for Different Spans and Loadings

Mix—1:2:4 Stresses : Concrete—50 kg/sq. cm ; Steel—1400 kg/sq. cm. m—18.7

Load (Super-imposed) kg/m	Effective Span in Metres															
	2.5 metres		3.0 metres		3.5 metres		4.0 metres		4.5 metres		5.0 metres		5.5 metres		6.0 metres	
	$d \times b'$ cm	As cm2	$d \times b'$ cm	As cm2	$d \times b'$ cm	As cm2	$d \times b'$ cm	As cm2	$d \times b'$ cm	As cm2	$d \times b'$ cm	As cm2	$d \times b'$ cm	As cm2	$d \times b'$ cm	As cm2
1300	30×20	3.20	30×20	4.85	30×22	7.15	30×22	9.35	30×22	11.80	35×25	12.00	35×25	15.00	35×25	17.80
1600	,,	3.84	,,	5.85	,,	8.60	,,	11.22	,,	14.20	,,	14.40	,,	18.00	,,	21.40
1900	,,	4.48	,,	7.20	,,	10.00	,,	13.10	,,	16.50	,,	16.80	,,	21.00	,,	25.90
2200	,,	5.12	,,	8.25	,,	11.44	,,	15.00	,,	18.90	,,	19.20	,,	24.70	,,	29.45
2500	,,	6.20	30×22	9.50	,,	12.77	,,	16.80	35×25	17.50	,,	21.60	,,	27.10	45×25	23.70
2800	,,	6.90	,,	10.50	,,	14.30	,,	18.70	,,	19.50	,,	24.70	45×25	22.00	,,	26.00
3100	,,	7.65	,,	11.60	35×25	13.00	35×25	16.80	,,	21.40	45×25	20.00	,,	25.00	,,	28.80
3400	,,	8.55	,,	12.65	,,	14.20	,,	18.30	,,	23.90	,,	21.80	,,	27.10	50×30	27.50
3700	,,	9.25	35×25	11.25	,,	15.40	,,	21 22	,,	25.70	,,	23.90	50×30	25.30	,,	30.70
4000	30×22	10.00	,,	12.10	,,	16.60	,,	21.40	45×25	21.20	50×30	22.40	,,	27.10	,,	32.30
43 0	,,	10.70	,,	13.00	,,	17.80	,,	22.80	,,	21.90	,,	24.00	,,	29.20	,,	34.30
4600	35×25	9.40	,,	13.85	45×25	13.80	45×15	18.00	,,	23.65	,,	25 68	,,	31.60	,,	36.60
4900	,,	10.00	,,	14.70	,,	14.70	,,	19.20	50×30	21.70	,,	27.70	,,	33.20	,,	40.20
5200	,,	10.60	,,	15.50	,,	15.60	,,	20.30	,,	23.65	,,	28.80	,,	35.20	,,	41.75

(Based on "RC Designer's Manual"—The Concrete Association of India.)
Where "commercial quality" untested steel is used, provide 10 per cent extra reinforcement.

Notes for T-Beam Table

(*i*) d is effective depth of the beam, b' is width of rib in cm. As is tensile reinforcement in sq. cm.

(*ii*) The beams have been designed for uniformly distributed loads on simply supported spans.

(*iii*) Beams should be checked for shear as in short beams shear often controls the design.

(*iv*) To arrive at the total load take superimposed load + dead weight of the slab (24 kg per 1 cm thickness times its width supported by the beam) + weight of the rib.

(*v*) Reinforcement rods should be provided in two rows (except for the small beams). In order to provide for possible negative moment at the supports due to accidental partial fixity, about 1/3 to 1/2 of the tensile bars may be cranked (bent) for a length of about 1/4 of the span on either side at 45 deg., and ends hooked.

(*vi*) For binding stirrups where no top reinforcement exists, provide two bars at the top of the rib (slab) as hanger bars. These bars may be of 8 mm dia. for small spans and up to 16 mm dia. for big spans.

(*vii*) Provide stirrups of 6 mm dia. bars 12 cm pitch at the ends (where no stirrups are required for shear). The pitch of stirrups between end of beam and mid-span may be gradually and uniformly increased to a maximum equal to the effective depth of the beam.

(*viii*) For simplicity of design the decking slab of the T may be designed as a simple slab with semi-fixed ends with negative reinforcement provide at the top of the web in the slab) of the T.

Simple Procedure for Design of T-beams

For a T-beam the breadth b of the flange shall not exceed the least of the following :

(*i*) One-third of the effective span of the T-beam ; (*ii*) the distance between the centres of the ribs of adjacent beams ; (*iii*) the breadth b' of the rib plus 12 times the thickness t of the flange. The flange breadth should be not less than 1/20 clear span.

For an ell-beam, the breadth of the flange is usually half of the breadth of the flange of a T-beam, and shall not exceed the least of the following :

(*i*) One-sixth of the effective span of the ell-beam ; (*ii*) the breadth b' of the rib plus one half the clear distance between ribs ; (*iii*) the breadth b' of the rib plus four times the thickness t of the slab.

Design of top slab or flange. In designing a T-beam, the flange forms part of the slab spanning across the beams. Determine t, the thickness of slab which is usually designed as a normal continuous slab spanning from rib to rib transversly to the line of ribs. Thickness of slab is between 10 to 15 cm according as the bending moment is light or heavy. With heavy loads, it may be even 18 cm. Where a slab spans across secondary T-beams which frame into main T-beams, the main reinforcement of the slab will run parallel to the main T-beams ; in that case at least 0.3 per cent of steel must be provided in the slab at right angles to the main T-beams. The maximum economic span for a slab is 3.5 metres. The slab is designed first so that the flange thickness t is decided.

Design of Rib. The overall depth of a T-Beam "d", should not be less than 1/20 of the clear span and is usually 4 to 6 times the thickness of the flange "t". Since the resisting moment varies as the square of the depth, the depth of a T-beam is of great importance. The greater the depth, the greater the lever arm and consequently, less the quantity of steel required, but there are limitations to the depth. As a guide, the depth of the rib may be taken :

(*a*) for light loads—1/15 to 1/20 of the span, (*b*) for medium loads—1/12 to 1/15 of the span, (*c*) for heavy loads –(1400 kg/sq. m and over)—1/12 of span.

Economic depth of a T-beam can also be based on the ratio of the cost of concrete and cost of steel per unit volume :

$$\text{Economic depth of a T-beam} = \sqrt{\frac{R \times BM}{fs \times b'}} + \frac{t}{2}$$

$$R = \frac{\text{cost of a unit volume of steel}}{\text{cost of a unit volume of concrete}}$$

Width of a rib—b' : Since width of the rib does not affect the moment of resistance of a T-beam, this may be kept as narrow as possible, but it has to be sufficiently wide to accommodate reinforcement bars. As a rough rule, a width of 2.5 times the sum of diameters of bars is required. As a first approximation, the rib width b' may be taken as a little over half the rib depth. The rib cannot be less than 15 cm wide where more than one bar is required. It is most economical to make $b' = 1/3\text{rd } d$, but it should not be less than $1/3\ (d - t)$ or less than $2t$, and not more than $2/3d$. Under distributed loads it may have the minimum size prescribed, but under concentrated loads check for shear.

Shear is the deciding factor in fixing the size of a rib and the cross-section must be sufficient to resist the shear force. The rib takes the entire shear, and its depth is measured from the top of the flange. The flanges of T-section are of negligible value in resisting shear.

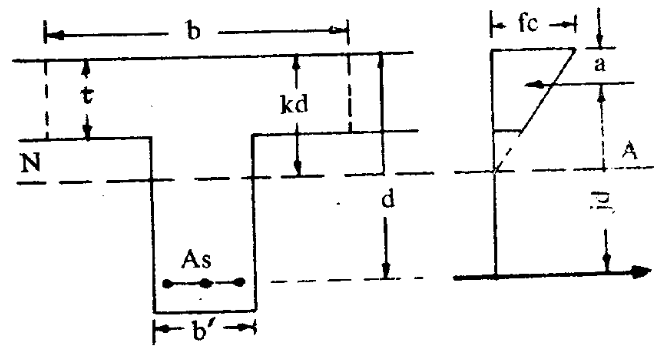
Total shear force $= b' \times (d - 1/2t) \times fs'$

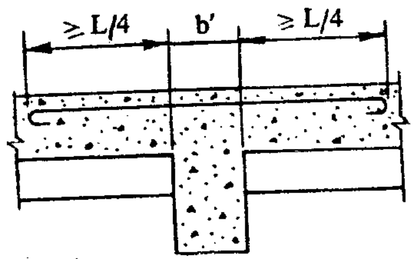
fs' is the permissible shear stress as given for Beams.

If the calculated depth d comes out to be excessive, tapered web may be provided to take care of shear as shown in the sketch.

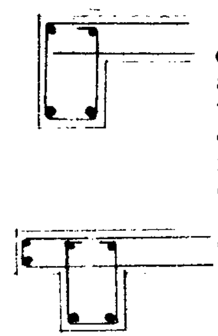
Derivation of Moment of Resistance of a T-beam

Position of neutral axis. The neutral axis, kd, is determined as for a rectangular beam. The neutral axis will normally lie below the flange. Where the slab is so thick that the neutral axis lies within the flange thickness, the T-beam is designed as a rectangular beam.





Transverse Reinforcement in Flange of T-Beam when Main Reinforcement of Slab is Parallel to the Beam. IS: 456—1978.



To simplify calculations, the small amount of compression taken by the concrete of the rib above the neutral axis (shown dotted), is usually neglected, and it is considered that the whole of the compression is taken by the concrete flange of area $b \times t$. The compressive force in the concrete is assumed to act at a point $(t/2 - a)$ below the compression edge.

Stresses. The compressive stress varies from a maximum fc at the extreme compression edge to zero at the neutral axis. Total compressive force on the concrete

$$= \frac{fc}{2}\left(2 - \frac{t}{kd}\right) \times b \times t.$$

Moment of resistance of beam. The compressive force acts at a distance $t/2$, and tensile force acts at a distance d from the extreme compression edge. Therefore, the lever arm $jd = d - t/2$.

Moment of resistance due to concrete in compression

$$=\frac{fc}{2}\left(2-\frac{t}{kd}\right)\times b\times t\times jd \qquad \text{which is} = As\times fs\times jd$$

$$\text{or} = \frac{fc\times b\times t}{2}\left(d-t/2\right) = As\times fs\left(d-t/2\right) \text{—approximately}$$

The approximate method gives safe results.

$$\text{Therefore, } fc=\frac{BM\times kd}{bt(kd-t/2)\times jd} \qquad \text{and } As=\frac{BM}{fs\times jd}$$

fc is compressive stress in the concrete and fs is tensile stress in the steel.

Steel percentage. "p" at equal strength ratio

$$=\frac{fc}{fs}\left(\frac{kd-t/2}{kd}\right)\frac{t}{d}$$

The cross-section for area is taken—$b\times d$ and not $b'\times d$.

For rough calculations reinforcement may be taken at 2 to 3 per cent of the rib section of secondary beams and 4 to 5 per cent of the main beams. The flange should have adequate reinforcement transverse to the beam and it should be built integrally with the beam or effectively keyed together with the beam. $kd=\frac{mfc}{mfc+t}\times d$ for a balanced section. When there are main and secondary beams, the top slab will span from one secondary beam to another at right angles to them and the tensile reinforcement of the slab will be in a direction parallel to the main T-beam. In such cases transverse reinforcement in the flange of the T-beam, at right angles to their ribs (over the top of the beam and near the top surface of slab), extending to a length of at least one-quarter of the span on either side of the beam should be provided. The quantity of this transverse reinforcement should be not less than 60 per cent of the main reinforcement in the centre of the slab constituting the flange.

The following table will be found useful for designing T-beams

Stresses	t/d	.10	.12	.14	.16	.18	.20
fs—1400	Q	4.17	4.81	5.40	5.92	6.40	6.83
fc—50	p	.31	.36	.41	.45	.49	.53
fs—1300	Q	4.19	4.84	5.45	5.99	6.49	6.92
fc—50	p	.34	.39	.44	.49	.54	.58
Stresses	t/d	.22	.24	.28	.32	.36	.40
fs—1400	Q	7.21	7.53	8.07	8.42	8.61	8.67
fc—50	p	.56	.60	.65	.68	.70	.71
fs—1300	Q	7.31	7.66	8.23	8.63	8.87	8.98
fc—50	p	.62	.65	.73	.75	.78	.80

Q (moment factor)—BM/bd^2; p is steel percentage ; Mix: 1:2:4.

Continuous T-beams. The design is similar to that of a doubly reinforced rectangular beam $b'\times d$. The upper flange portion is in tension and the lower rib portion is in compression over the supports. For tension at the top, the flanges at the supports are ignored and steel is provided to take up the whole of the tension. The concrete area of the ribs is not usually sufficient for the compression at the supports. To meet this the beam is either splayed out at the supports to increase the depth of the rib, or compression reinforcement is provided to supplement the compressive resistance of the rib, or a combination of both the methods is employed. The depth of the splay of the haunch near the supports is made equal to $2d$, and length equal to L/6, measured from the centre of the support ; L is length of the span. The tensile bars of the mid-span of the T-beam which are carried through to the support should be bent down with the splay.

In L-beams, a bending moment applied in a vertical plane produces twisting moment. This is resisted by the top slab and any beams which frame into the L-beams. To help the twisting moment, extra top steel should be provided along the length of the L-beam and also a strong system of stirrups.

Main and Secondary Beams. Main beams are normally more heavily reinforced than secondaries. It is desirable that the depth of the main beams should be greater than those of the secondary beams as the tensile bars in the secondaries have to pass over the tensile bars in the main beams.

The loads on the main beam are a series of point loads due to the reactions of the secondaries. Usually not more than one or two secondaries will be carried by one main beam and it will thus carry either a concentrated load at mid-span or at the third points.

It may simplify the arrangement of bars if the moment at mid-span and supports are considered to be equal, *i.e.*, design for the max: moment out of the two.

Shear on main beams must be carefully checked. It is likely to be heavy.

Ribs and slab are placed (cast) monolithically, and ribs are not further apart than 1.5 m centre to centre and depth not more than four times (prefer not over three times) their width. Depth of ribs is usually taken as at least L/20 with free support and at least L/25 with fixed support. Width not less than 65 mm (prefer 75 mm) to 150 mm. Slab thickness is 40 to 75 mm. Span of the ribs may be up to 7 metres ; when the span exceeds 3 metres, lateral ribs of the same width as the main longitudinal ones are provided at intervals of between 1 to 3 metres.

A strip of solid slab at least 10 cm wide shall be left adjacent to the supporting beams or walls. Generally ribs should be formed along each edge parallel to the span of one way slabs. When the edge is built into a wall or rests on a beam, a rib at least as wide as the bearing shall be formed along the edge.

Light reinforcement is provided in the slab at right angles to the ribs and just above the hollow tiles, where a slab, which is continuous over supports, has been designed as simply supported ; reinforcement shall be provided over the supports to control cracking. This reinforcement shall have a cross-sectional area of not less than one-quarter that required in the middle of the adjoining spans and shall extend at least one-tenth of the clear span into adjoining spans.

Ribbed or Hollow-Tile Floors

Where spans are large and super-load light, self weight is a major item, economy and lightness can be effected by the incorporation of hollow earthenware or pre-cast concrete tiles in the lower part of the concrete slab. In a normal RC slab, the concrete below the neutral axis being in tension, serves no useful purpose as far as strength is concerned, and is neglected in design. Part of this concrete is replaced by hollow tiles. The tiles are placed in rows so that concrete ribs are formed between the rows, and the floor becomes a series of T-beams. The tiles are ribbed on their outer surface and thus key-in with the surrounding concrete. A ribbed floor is considerably lighter and cheaper than a solid RC slab floor of similar depth.

Design of a ribbed floor follows the same principles as for T-beams. The concrete above the tiles takes compression. Width of the compression flange is centre to centre of the ribs.

The maximum shear at the supports should be checked and should be considered to be resisted by the area of the rib only, lever arm being taken for the respective case.

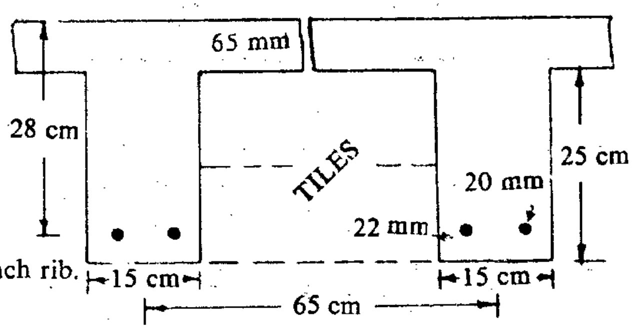
Example—typical ribbed floor :

Span—7 metres

Live load—300 kg/sq.m

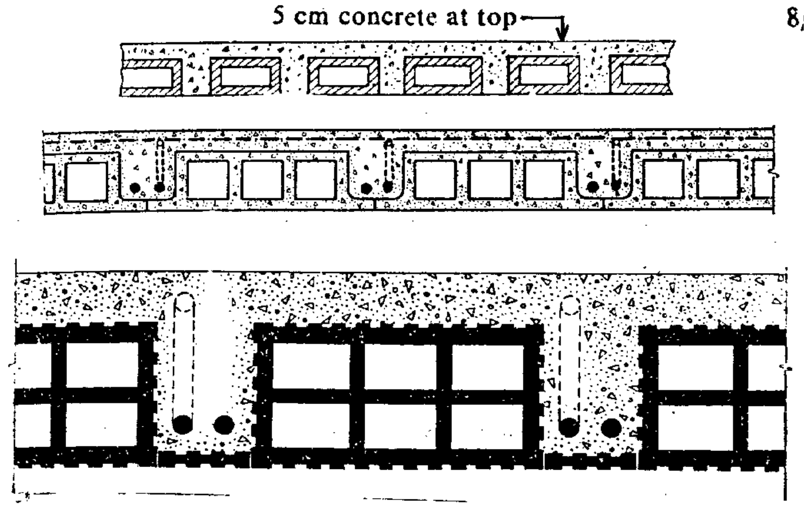
BM: $\frac{WL}{10} = 2100$ kg-m

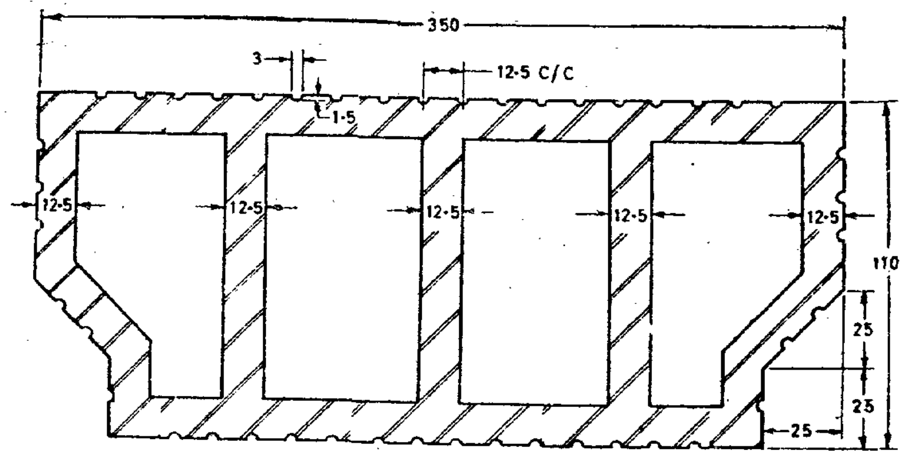
$A_s = 6.6$ sq. cm in each rib.



(Concrete is very much under-stressed) (Work already built)

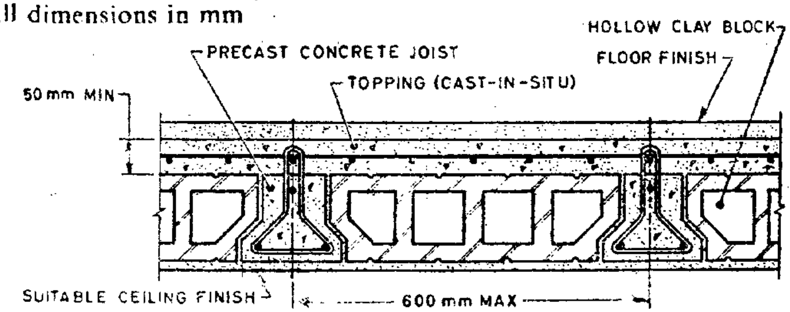
Hollow tile floors consist of a series of T beams spaced at 45 to 60 cm centres. The ribs are 10 to 12·5 cm wide and the flange thickness varies from 4 to 6 cm. The necessary reinforcement to balance the concrete strength in the flanges is embedded 2.5 cm or more from the bottom of the ribs. Hollow tile floors should not be used for live loads of more than 500 kg/sq.m.





TYPICAL HOLLOW CLAY FILLER BLOCK IS : 6061

All dimensions in mm



PLACING HOLLOW BLOCK IN BETWEEN JOISTS

Areas, perimeters and spacings of round bars

Diameter of bar, mm		Area of steel and perimeter per metre width for spacings of									
		6 cm	7 cm	8 cm	9 cm	10 cm	11 cm	12 cm	13 cm	14 cm	15 cm
5	Area, cm2	3.27	2.80	2.46	2.18	1.96	1.78	1.63	1.51	1.40	1.31
	Perimeter, cm	26.15	22.40	19.60	17.45	15.70	14.25	13.05	12.08	11.20	10.45
6	Area, cm2	4.72	4.04	3.54	3.14	2.83	2.57	2.36	2.17	2.02	1.89
	Perimeter, cm	31.35	26.85	23.50	20.85	18.80	17.10	15.65	14.45	13.45	12.55
8	Area, cm2	8.38	7.18	6.28	5.58	5.03	4.57	4.19	3.87	3.59	3.35
	Perimeter, cm	41.75	35.80	31.35	27.90	25.10	22.80	20.90	19.30	17.90	16.70
10	Area, cm2	13.08	11.22	9.82	8.72	7.85	7.14	6.54	6.04	5.61	5.24
	Perimeter, cm	52.30	44.90	39.25	35.00	31.40	28.55	26.15	24.20	22.50	20.95
12	Area, cm2	18.84	16.16	14.14	12.56	11.31	10.28	9.42	8.70	8.08	7.54
	Perimeter, cm	63.00	53.95	47.15	41.90	37.70	34.35	31.50	29.00	26.95	25.20
16	Area, cm2	33.52	28.72	25.14	22.34	20.11	18.28	16.76	15.47	14.36	13.41
	Perimeter, cm	84.00	72.00	63.00	56.00	50.30	45.75	42.00	38.75	36.00	33.55
20	Area, cm2	52.34	44.88	39.28	34.92	31.42	28.55	26.17	24.16	22.44	20.94
	Perimeter, cm	104.10	89.75	78.50	69.90	62.80	57.00	52.15	48.25	44.90	41.80
22	Area, cm2	63.34	54.30	47.52	42.24	38.01	34.55	31.67	29.24	27.15	25.34
	Perimeter, cm	115.00	98.90	86.50	77.00	69.10	63.00	57.85	53.10	49.50	46.10
25	Area, cm2	82.00	70.20	61.50	54.70	49.03	44.70	41.00	37.90	35.10	32.75
	Perimeter, cm	130.80	112.00	98.10	87.20	78.50	71.40	65.40	60.15	56.05	52.25

Areas, perimeters and spacings of round bars (*Contd.*)

Diameter of bar, mm		Area of steel and perimeter per metre width for spacings of									
		16 cm	17 cm	18 cm	19 cm	20 cm	22.5 cm	25 cm	27.5 cm	30 cm	40 cm
5	Area, cm2	1.23	1.16	1.09	1.03	0.98	0.87	0.79	0.71	0.65	0.49
	Perimeter, cm	9.80	9.25	8.72	8.25	7.85	6.98	6.28	5.70	5.21	3.93
6	Area, cm2	1.77	1.66	1.57	1.49	1.41	1.25	1.13	1.02	0.94	0.71
	Perimeter, cm	11.75	11.05	10.45	9.90	9.40	8.35	7.52	6.82	6.25	4.70
8	Area, cm2	3.14	2.96	2.79	2.65	2.51	2.23	2.01	1.82	1.67	1.26
	Perimeter, cm	15.65	14.75	13.95	13.20	12.55	11.15	10.04	9.10	8.35	6.28
10	Area, cm2	4.91	4.62	4.36	4.13	3.93	3.50	3.15	2.85	2.62	1.97
	Perimeter, cm	19.65	18.50	17.45	16.55	15.70	14.00	12.56	11.40	10.50	7.85
12	Area, cm2	7.07	6.65	6.28	5.95	5.65	5.02	4.52	4.11	3.76	2.83
	Perimeter, cm	23.60	22.20	21.00	19.85	18.85	16.80	15.08	13.70	12.60	9.43
16	Area, cm2	12.57	11.83	11.17	10.58	10.05	8.90	8.02	7.30	6.70	5.03
	Perimeter, cm	31.50	29.60	28.00	26.50	25.15	22.40	20.12	18.35	16.80	12.58
20	Area, cm2	19.64	18.48	17.46	16.54	15.71	13.95	12.58	11.40	10.45	7.36
	Perimeter, cm	39.25	36.90	34.90	33.00	31.40	27.90	25.12	22.80	20.90	15.70
22	Area, cm2	23.76	22.36	21.12	20.01	19.01	16.85	15.22	13.85	12.70	9.51
	Perimeter, cm	43.25	40.65	38.50	36.45	34.55	30.75	27.64	25.20	23.10	17.28
25	Area, cm2	30.75	28.95	27.35	25.80	24.54	21.80	19.65	17.85	16.40	12.27
	Perimeter, cm	49.20	46.20	43.55	41.30	39.25	34.95	31.40	28.50	26.10	19.63

Sectional areas of groups of round bars, cm²

Diameter of bar, mm	Number of bars: 1	2	3	4	5	6	7	8	9	10	11	12
5	0.20	0.39	0.59	0.78	0.98	1.18	1.37	1.57	1.77	1.96	2.16	2.35
6	0.28	0.56	0.85	1.13	1.41	1.70	1.98	2.26	2.54	2.83	3.11	3.40
8	0.50	1.00	1.51	2.01	2.51	3.01	3.52	4.02	4.52	5.03	5.53	6.04
10	0.79	1.57	2.36	3.14	3.93	4.71	5.50	6.28	7.07	7.85	8.64	9.42
12	1.13	2.26	3.39	4.52	5.65	6.78	7.91	9.05	10.18	11.31	12.44	13.57
16	2.01	4.02	6.03	8.04	10.05	12.06	14.07	16.08	18.09	20.11	22.12	24.13
20	3.14	6.28	9.42	12.57	15.71	18.84	21.99	25.14	28.28	31.42	34.56	37.70
22	3.80	7.60	11.40	15.21	19.01	22.81	26.61	30.41	34.21	38.01	41.81	45.62
25	4.91	9.82	14.73	19.63	24.54	29.45	34.36	39.27	44.18	49.09	54.00	58.91
28	6.16	12.31	18.47	24.63	30.79	36.94	43.10	49.26	55.42	61.58	67.73	73.89
32	8.04	16.08	24.13	32.17	40.21	48.26	56.30	64.34	72.38	80.42	88.47	96.51
36	10.18	20.36	30.54	40.72	50.90	61.07	71.26	81.43	91.61	101.79	111.97	122.15
40	12.57	25.13	37.70	50.26	62.83	75.40	87.96	100.53	113.09	125.66	138.23	150.80
45	15.90	31.81	47.71	63.62	79.52	95.42	111.33	127.32	143.14	159.04	174.94	190.85
50	19.64	39.27	58.91	78.54	98.18	117.81	137.45	157.08	176.72	196.35	215.99	235.62

3. DESIGN OF COLUMNS

Types of Columns. Columns may be rectangular, square, circular or hexagonal. Reinforced concrete columns have two types of steel reinforcement : (i) main or longitudinal reinforcement consisting of vertical bars which share the load with the concrete and also take any tensile stresses caused by lateral forces or eccentric loads, and (ii) transverse or lateral reinforcement which bind the longitudinal reinforcement. The object of the transverse reinforcement is to prevent the buckling or spreading out of the longitudinal bars and also to prevent the concrete from splitting outwards on planes of greatest shear stress which is at 45 deg. to the axis of the column. The more closely spaced these links are, the greater the load the column will carry.

The columns are divided into the types according to as the transverse reinforcement is fixed. These are *tied columns* in which the longitudinal bars are tied by independent links (also called hoops, ties or binders) at certain vertical distance apart, and *spirally reinforced or helix columns* in which the binders or links are either spirally round the vertical reinforcement in continuous helix closely spaced or are provided as separate hoops, on the outside face of the column. Links are tied with 16 gauge wire with the vertical reinforcement. This makes the column much tougher and this type of columns are used for heavy loads or where the size is restricted. A *composite* column (with metal core) is one in which a steel or cast iron section is completely encased in concrete.

The cross-sectional area of the metal core shall not exceed 20 per cent of the gross area of the column. If a hollow metal core is used, it should be filled with concrete. A clearance of at least 75 mm shall be maintained between the spiral and the metal core at all points, except that when the core consists of a structural steel H-column, the minimum clearance may be reduced to 50 mm.

Short and Long Columns

A column or strut is a compression member, the effective length of which exceeds three times the least lateral dimension.

A compression member may be considered as short when both the slenderness ratios (or buckling factor) L_1 / D and L_2 / b are less than 12, or the least radius of gyration does not exceed 40 :

Where :

L_1 is the effective length in respect of the major axis ; D is depth in respect of the major axis ; L_2 is effective length in respect of the minor axis and b is the width of the member (minor axis).

The unsupported length between end restraints shall not exceed 60 times the least lateral dimension of a column.

If, in any given plane, one end of a column is unrestrained, its unsupported length, L, shall not exceed $100\ b^2 / D$.

Where : b is width of that cross-section, and D is depth of the cross-section measured in the plane under consideration.

Columns generally fail by buckling unless the length is small. Columns with slenderness ratio within the above limits are safe against buckling. If effective column length to least lateral dimension exceeds 12, to obtain safe loads on long columns multiply by a "reduction factor" given in the following table.

Reduction Factor (R) for Long Columns

Ratio L/*b*	12	15	18	21	24	27	30	33	36	39	42	45
Factor	1.00	0.94	0.88	0.81	0.75	0.69	0.63	0.56	0.50	0.44	0.38	0.31

$$R = 1.25 - \frac{L}{48b}$$

Where :

R is reduction factor ; L is "effective length" of column ; *b* is least lateral dimension ; for column with helical reinforcement, *b* is the diameter of the core.

Recommended Values of Effective Length of Compression Members
(IS: 456—1978)

Degree of End Restraint of Compression Member	Effective Length
1. Effectively held in position and restrained against rotation at both ends.	0.65 L
2. Effectively held in position at both ends, restrained against rotation at one end.	0.80 L
3. Effectively held in position at both ends, but not restrained against rotation.	1.00 L
4. Effectively held in position and restrained against rotation at one end, and at the other restrained against rotation but not held in position.	
5. Effectively held in position and restrained against rotation at one end, and at the other partially restrained against rotation but not held in position.	1.50 L
6. Effectively held in position at one end but not restrained against rotation, and at the other end restrained against rotation but not held in position.	2.00 L
7. Effectively held in position and restrained against rotation at one end but not held in position nor restrained against rotation at the other end.	2.00 L

L is the unsupported length of the column.

Permissible Loads on Columns

Axially loaded columns. An axial load is a load having its resultant acting at the centroid of the column section.

(*i*) Safe axial load P a short RC column can carry with longitudinal bars and lateral ties or rings :

$$P = fc \times A + fs' \times As$$

(*ii*) Safe axial load P a short RC column spirally reinforced can carry :

$$P = (fc \times A') + (fs' \times As) + (2fh \times Ah)$$

Where :

fc = permissible stress in concrete in direct compression ; A = net cross-sectional area of concrete excluding cross-sectional area of longitudinal bars ; *fs'* = permissible compressive stress in steel for column bars (1300 kg/sq. cm) ; A' = cross-sectional area of concrete in core of column excluding cross-sectional area of longitudinal bars (core may be taken 6 to 8 cm less in dia. than the external dia. of the column) ; A*s* = cross-sectional area of longitudinal bars ; *fh* permissible stress in helical reinforcement (1000 kg/sq. cm) ; A*h* = volume of helical reinforcement per unit (cm) length of column (*i.e.*, volume of one round of binder divided by the pitch).

The value of $fc \times A' + 2fh \times Ah$ should not exceed $1/2 \times$ crushing strength of concrete × A.

Permissible stresses are given in Tables II and IV.

When in spirally reinforced concrete columns the permissible load is based on the core area, the least lateral dimension and the radius of gyration of the column shall be taken of the core of the column.

Safe loads obtained from the above equation for short columns shall be multiplied by the reduction factor given in the table given at page 8/50 to obtain safe loads on long columns.

Care should be taken to see that the main longitudinal reinforcing rods are truly in plumb and remain so while concreting. The centre line of all the columns one above another on different floors should be the same.

In the case of a column continuing through two or more stories, L is the length of the column between floor levels, or between floor level and any adequate intermediate bracing or support. Floor beams may be taken as lateral restraints for determining the value of L.

The rigidity of RCC ensures good conditions of end fixity and the effective length normally is between 0.75L and 1.25L. The number and size of beams framing into a column at a floor level and the type of foundation in the ground floor columns will affect the degree of end fixity. For most of the RCC column and beam works, actual length will be the effective length.

The length of a column should not be more than 45*b* for axially loaded columns and 20*b* for columns subjected to bending in addition to direct loads. *b* is least lateral dimension of column.

Eccentrically Loaded Columns Producing Bending Moments

Eccentric loading on columns may be caused by brackets carrying heavy loads and from unsymmetrically loaded beams which frame the column : external columns are subject to greater bending moments than internal columns.

Bending moments due to a bracket may be calculated as explained in Sections 3 and 10, with the equation $\frac{W}{A} \pm \frac{M}{Z}$. A is the equivalent concrete area when the column is reinforced and is $= bd + (1.5\ m - 1)\ As$.

$$I = \frac{bd^3}{12} + \left[(1.5\ m - 1)\ As \times \left(\frac{d}{2} - a \right)^2 \right]; \quad Z = \frac{I}{d/2}$$

Where : I = moment of inertia of column section about neutral axis ; *bd* are sides ; A*s* is total area of steel ; *a* = distance of centre of steel from outside edge of column.

It has been explained in Section 7 that when the resultant pressure falls away from the centre at a distance equal to one-sixth of the base width, there is neither tension nor compression at the heel (extreme edge) but the pressure on the toe (edge nearer the load) is twice the average pressure. In RC columns if *e* (eccentricity) is less than about 0.2*b*, no tension will occur in the column and the maximum compression can be calculated from the equation given above. The compressive stress (which is combined axial and bending stress) must not exceed the permissible compression on concrete in bending given in Table II (50 for a 1:2:4 mix and not 40), multiplied by the reduction co-efficient for long columns where applicable. A maximum tension of one-sixth of the permissible direct compressive stress may be allowed in the concrete. If this stress is exceeded on the tensile side, section of the column must be increased.

The centre line of all the columns, one upon another on different floors, which are axially loaded, should be the same.

In the case of unsymmetrically loaded beams framing into columns, it requires lengthy calculations for the exact design of columns for the eccentric loadings. Large number of columns carrying beams have, however, been designed and constructed for a direct axial load only, the column load being taken as the end reaction of the beams framing into it and the end reaction being calculated as if the beams were simply supported. This may be taken to be sufficiently accurate and adequately safe for normal column design in the field. A negative bending moment should, of course, be allowed for in the design of the beam at its junction with the column.

Another simple approximate rule erring on the safe side is to consider a bending moment at the top of a monolithic corner column equivalent to one-third of the maximum positive bending moment of the supported beam. If the base of the column is considered fixed, take one-sixth of the positive bending moment for the base as well. For internal columns supporting continuous beams (not monolithic with the column), take additional vertical load on the column equivalent to 15 per cent of the total dead and live loads on the column. Consider the worst position for the loads.

In a building of more than one floor, the effect of an eccentricity of load at a given floor is considered to have disappeared at the floor below. The column in the floor below is designed for an axial load only from the floor above.

Longitudinal reinforcement. The cross-sectional area of longitudinal reinforcement varies from 0.8 per cent to 6 per cent of the gross cross-sectional area of the column. From 1 to 4 per cent of steel is commly used in tied columns and greater percentage in spiral columns.

The longitudinal reinforcement should be not less than 0.15 per cent in the case of columns and pedestals in which the longitudinal reinforcement is ignored for the purpose of calculating the permissible load on the column.

Sizes of longitudinal bars usually vary from 12 mm min: to 36 mm. The minimum size is governed by the need to ensure that the bars are stiff enough to stand up straight in the column boxes during concreting.

A column bar may be left straight at its end, no hook or bend being necessary.

While filling in concrete care should be taken to see that all the rods stay truly straight.

The entire length of column from top of lower floor or foundation to the soffit of the beam under the upper floor should be filled in one operation.

For small columns, four bars, one in each corner, are used. For larger sizes, up to eight or twelve bars may be used. The minimum number of vertical bars is four in tied columns and six in spiral (circular) columns. All bars should be placed equidistant. The clear space between the bars within the periphery of the column core should be not less than 1.5 times the diameter of round bars or twice the side dimensions of square bars.

Extended longitudinal reinforcement or dowels of at least 0.5 per cent of the cross-sectional area of the supported column or pedestal and a minimum of four bars shall be provided. Where dowels are used, their diameter shall not exceed the diameter of the column bars by more than 3 mm.

Column bars of diameter larger than 36 mm, in compression only can be dowelled at the footings with bars of smaller size of the necessary area. The dowel shall extend into the column a distance equal to the development length of the dowel.

For joints in the longitudinal reinforcements, the bars should be overlapped for at least 24 times the diameter of the bars, and for works in water, for 30 diameters. The joints should be staggered. Joints should be made at floor levels or beam intersections. The transverse reinforcement should be spaced closer at the joints. No hooks need be provided at the ends of the compression bars where joining.

In any column, that has a larger cross-sectional area than that required to support the load, the minimum percentage of steel should be based upon the area of concrete required to resist the direct stress and not upon the actual area. Also, in case of columns having the ratio of length to least radius of gyration less than 12, the requirement regarding minimum amount of steel need not apply.

When a column continues up through a floor from one storey to the next, the main longitudinal bars of the column must pass up either within or outside the reinforcement in the floor beams which frame into the column. In the latter case the width of the column should be at least 7.5 cm more than the width of any beam framing into it. If in addition the section of the column is smaller above the floor than it is below, the main bars should be bent inwards at floor level, or be stopped off short below floor level.

Transverse reinforcement. The diameter of the transverse reinforcement (binders) should be not less than one-fourth of the diameter of the largest longitudinal bars, nor less than 5 mm, except for very small columns when 10 gauge wire can be used. 16 gauge wire is used for tying the bars. The maximum size of binders used is 12 mm for independent ties and up to 20 mm for spirals. The binders pass round the outside of the main bars and are anchored by hooking over one of the bars.

The *pitch* of transverse reinforcement shall be not more than the least of the following distances : (*i*) The least lateral dimension of the column ; (*ii*) Sixteen times the smallest diameter of the longitudinal reinforcement bar to be tied ; (*iii*) Forty-eight times the diameter of the transverse reinforcement. The pitch need not be less than 15 cm.

The volume of transverse reinforcement should be not less than 0.2 per cent of the gross volume of a column with up to 2 per cent of main bars, or 0.4 per cent of a column with over 2 per cent of main bars. In calculating volume of links, the end hooks are not included.

In the case of *helical reinforcement* the pitch of the helical turns should be not more than 75 mm nor more than one-sixth of the core diameter of the column, whichever is less, and need not be less than 25 mm, nor less than three times the diameter of the steel bar forming the helix. The volume of the spiral steel should be at least 1 per cent of the core. The helical binding should be regular, evenly spaced and anchored at the ends. (See also page 21/12).

Column Footings

The type of foundation used extensively in buildings are isolated spread footings under columns. Such footings may be square or rectangular in plan, and in section they may be of the slab, stepped or sloping type. The stepped footing results in better distribution of load than a slab footing. A sloped footing is more economical in concrete but is less economical in construction than the stepped footing because of the sloping surface. The isolated spread footing may be in plain concrete or reinforced concrete. In plain concrete footing the column load is transferred to the soil through dispersion in the footing. In reinforced concrete footing, the footing slab is treated as an inverted cantilever bearing the soil pressure and supported by the column.

Different methods are used for the design of footings, the undernoted are the most common and simple.

Area of a footing ($a \times a$ for a square footing).

Total load to be carried including the weight of the column + weight of footing block is equated to the safe bearing pressure of the soil.

The weight of footing may be taken about 10 per cent of the total load for determining the area of the footing.

The depth of the thickness of the footing is determined by one of the three—(*a*) Punching shear ; (*b*) Diagonal shear and (*c*) Bending moment—whichever gives the maximum result.

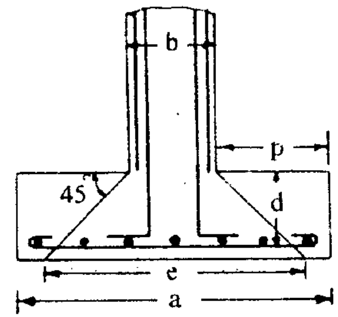
A footing is, as a rule, strong in resistance to bending moment. Hence the depth is generally determined by the punching shear or diagonal shear, whichever gives greater result.

(*a*) **Punching Shear** on footing. The footing slab or block must be thick enough to resist the tendency of the column to penetrate and punch a hole through the foundation block. The permissible punching shear stress on concrete is usually taken equal to twice the ordinary shear—10 kg/cm^2 for a 1:2:4 mix.

The area over which the punching shear would act is the perimeter of the column multiplied by the effective depth d of the footing, *i.e*, $4b \times d$. The intensity of the upward reaction of the soil below the footing caused by the load W of the column is W/a^2 and acts on the area $a^2 - b^2$ with a tendency to lift the footing while the load W on the column presses the column down. These two forces opposite to each other in direction cause a shear which acts along the sides of the column through the thickness of the footing block. Equating these two : $\frac{W}{a^2}(a^2 - b^2) = 4b \times d \times s$, where s is permissible unit punching stress. This gives the effective depth due to punching shear.

(*b*) **Ordinary Diagonal Shear.** Normal shear stress which occurs outside 45 deg. lines drawn from the edges of the column downwards through the block, must not exceed the permissible shear stress as given below.

Total shear = unit upward soil pressure $(W/a^2) \times (a^2 - e^2)$ and acts on the area $4e \times jd$. Therefore,



$$d = \frac{W(a^2 - e^2)}{a^2 \times 4e \times jd \times s}$$

s is permissible diagonal shear stress—5 kg/sq. cm for a 1:2:4 mix ; 7.0 kg/sq. cm for a 1:1.5:3 mix and 8.0 kg/sq. cm for a 1:1:2 mix.

jd is lever arm.

(*c*) **Bending Moment** due to the cantilever action of the footing projection *p*. (The bending moment is usually taken at the centre line of the column although the projection of the footing is fixed below the column and it should be considered at the edge of the column.)

$$\text{B.M.} = \frac{W}{8a}(a-b)^2 \text{ on unit width of foundation block.}$$

Equate this with moment of resistance due to concrete $= Q \times bd^2$ or $8.67\, bd^2$ (for 1:2:4 mix) from which d can be found.

Take the maximum depth that works out from either of the above four stresses. Depth due to punching shear is usually maximum.

In reinforced or plain concrete footings, thickness of the edge should be not less than 15 cm for footing on soils, nor less than 30 cm above the tops of piles for footings on piles. The footing may have sloped upper surface provided the edge thickness is kept a minimum of 15 cm.

Reinforcement

Area of steel (As) will be (as usual) $= \frac{BM}{fs \times jd}$

for unit width of block and is distributed uniformly across *p*. The tensile reinforcement may be provided either one-way or two-way, which should be able to resist the full bending moment in the former, and at least 85 per cent in the latter case. Prefer two-way reinforcement for all heavy loads.

Intensity of bond stress should also be checked as for beams.

All the bars should be hooked at the ends.

Load Carrying Capacity of Longitudinal Bars in Columns

Bar Dia. mm	Area sq. cm	Force per each bar: Mild-steel tonnes	Force per each bar: Tor-steel tonnes	
12	1.13	1.47	2.15	Grade I mild steel has been taken for the reinforcement with allowable compressive stress of 1300 kg/sq. cm. Where grade II steel is used, add 10% extra steel for vertical reinforcement or alternatively, reduce safe load by 10%.
16	2.01	2.61	3.82	
18	2.54	3.31	4.83	
20	3.14	4.08	5.97	
22	3.80	4.94	7.22	
25	4.91	6.38	9.33	
28	6.16	8.00	11.70	
32	8.04	10.45	15.28	
36	10.18	13.23	19.34	
40	12.57	16 33	23.88	

Compressive stress for mild steel grade I—1300 kg/sq. cm ; for tor-steel—1900 kg/sq. cm.

The safe loads given apply only to short columns and which are fixed adequately in position and direction. The sections are intended only for interior supports where no bending moments need to be considered. Columns which are subjected to bending stresses due to eccentricity of loads, the section must be designed accordingly.

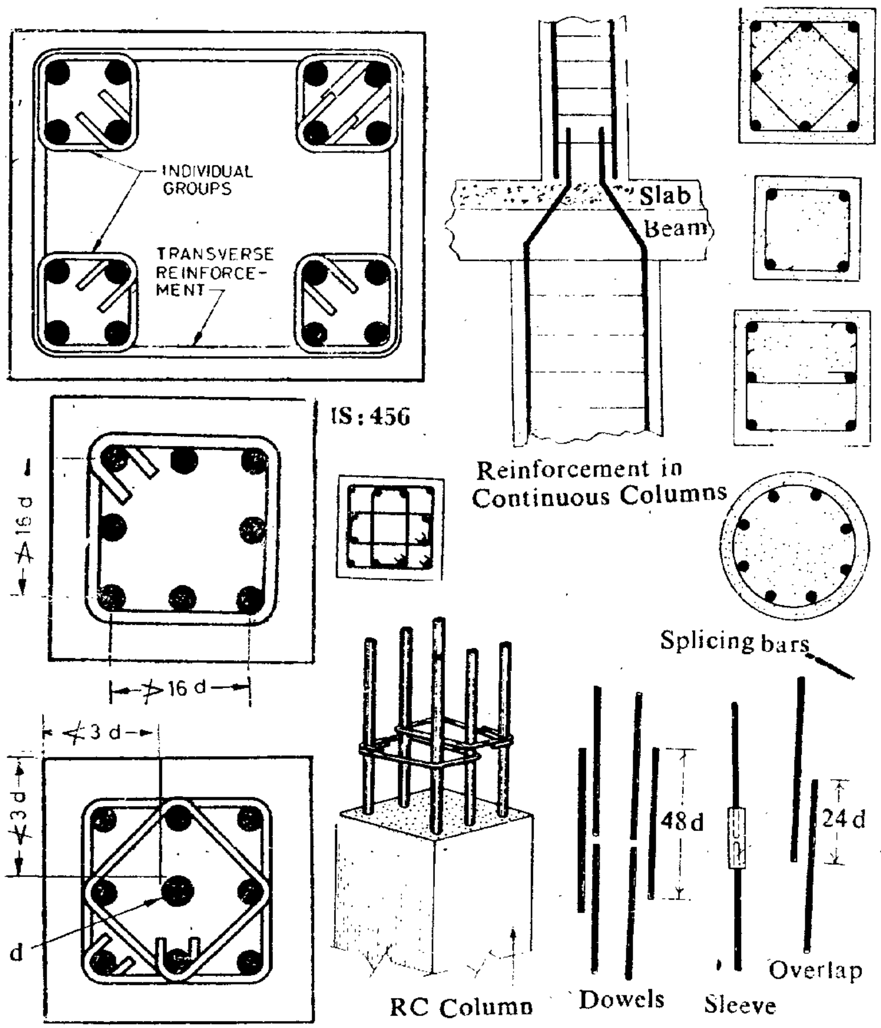
Safe Axial Loads on RCC Circular Columns With Helical Reinforcement

Dia. outer cm	Vertical bars (300 kg/sq. cm) nos.	Vertical bars dia. mm	Helical or Spiral reinforcement pitch mm	Helical or Spiral reinforcement bar dia. mm	Load Carried by Column: mix 1:2:4 tonnes (40 kg/sq. cm)	mix 1:1.5:3 tonnes (50 kg/sq. cm)	mix 1:1:2 tonnes (60 kg/sq. cm)
15	8	10	32	6	17	18	18
	8	12	28	6	21	22	23
	8	16	26	6	31	31	32
20	8	12	32	6	27	29	30
	8	16	28	6	37	39	40
	8	22	26	6	56	57	59
25	8	12	32	8	43	46	49
	8	16	28	8	54	57	60
	8	25	26	8	85	88	91
30	8	12	30	8	57	62	69
	8	22	28	8	86	90	95
	12	25	38	10	122	125	129
35	8	12	30	8	69	76	82
	8	22	40	10	102	108	115
	12	28	36	10	161	167	174
40	8	16	30	8	114	128	142
	8	28	38	10	189	201	214
	12	32	44	12	211	219	227
45	8	16	32	8	114	128	142
	8	32	38	10	189	201	214
	12	36	42	12	276	286	297
50	8	20	34	8	143	160	177
	8	36	38	10	233	249	265
	12	40	42	12	324	337	350
55	8	20	34	8	165	186	207
	8	36	36	10	259	279	299
	12	40	70	16	352	369	386
60	8	22	34	8	195	221	246
	8	40	45	12	321	346	370
	12	40	70	16	391	412	432

Design of Square Spread Footings under RCC Columns for Bearing Pressure of Soil

Column Size (outer) "b" cm	Load on Column tonnes	Depth of Footing "D" cm	Side of Footing and Reinforcement: 5 tonnes/sq. m			10 tonnes/sq. m			15 tonnes/sq. m			20 tonnes/sq. m			25 tonnes/sq. m			30 tonnes/sq. m		
			Side of Footing "a" cm	Reinforcement nos.	Reinforcement bar dia. mm	Side of Footing "a" cm	Reinforcement nos.	Reinforcement bar dia. mm	Side of Footing "a" cm	Reinforcement nos.	Reinforcement bar dia. mm	Side of Footing "a" cm	Reinforcement nos.	Reinforcement bar dia. mm	Side of Footing "a" cm	Reinforcement nos.	Reinforcement bar dia. mm	Side of Footing "a" cm	Reinforcement nos.	Reinforcement bar dia. mm
15×15	14	36	180	10	12	125	8	12	100	8	12	90	8	12	80	8	12	75	8	12
20×20	21	38	220	18	12	155	12	12	125	12	12	110	12	12	100	12	12	90	12	12
,,	26	48	240	18	12	170	12	12	140	10	12	120	10	12	105	10	12	100	10	12
25×25	30	42	265	16	16	190	12	16	155	12	16	135	12	16	120	12	16	110	12	16
,,	35	48	275	16	16	195	10	16	160	10	16	140	10	16	125	10	16	115	10	16
,,	40	60	300	14	16	210	10	16	175	10	16	150	10	16	135	10	16	125	10	16
,,	44	70	310	14	16	220	10	16	180	8	16	155	8	16	140	8	16	130	8	16
,,	49	75	330	16	16	235	10	16	190	8	16	165	8	16	150	8	16	135	8	16
30×30	46	50	320	22	16	225	14	16	185	12	16	160	12	16	145	12	16	130	12	16
,,	51	60	335	20	16	240	14	16	195	12	16	170	12	16	150	12	16	140	12	16
,,	55	70	350	20	16	245	14	16	200	10	16	175	10	16	155	10	16	145	10	16
,,	60	75	370	20	16	260	14	16	210	12	16	185	10	16	165	10	16	150	10	16
,,	74	75	405	18	20	285	20	16	235	14	16	200	14	16	180	14	16	165	14	16
35×35	64	60	380	18	20	265	12	20	220	12	20	190	12	20	170	12	20	155	12	20
,,	68	70	390	18	20	275	12	20	225	10	20	195	10	20	175	10	20	160	10	20
,,	73	75	400	18	20	285	12	20	235	10	20	200	10	20	180	10	20	165	10	20
40×40	83	70	425	22	20	300	16	20	250	12	20	215	12	20	190	12	20	175	12	20
,,	88	75	440	22	20	310	16	20	255	12	20	220	12	20	200	12	20	180	12	20
,,	95	85	455	22	20	325	16	20	265	12	20	230	12	20	205	12	20	190	12	20
,,	113	90	500	22	20	355	16	22	290	12	22	250	12	22	225	12	22	205	12	22
45×45	105	75				340	20	20	280	18	20	240	14	20	215	14	20	200	14	20
,,	112	85				350	20	20	290	16	20	250	14	20	225	14	20	205	14	20
50×50	131	85				380	24	20	310	20	20	270	16	20	240	16	20	220	16	20
,,	149	90				405	22	22	330	18	22	290	16	22	255	16	22	235	16	22
,,	162	95				420	26	22	345	20	22	300	16	22	265	16	22	245	16	22
55×55	152	85				410	24	22	335	18	22	290	18	22	260	18	22	235	18	22
,,	170	95				435	26	22	355	20	22	305	16	22	275	16	22	250	16	22
,,	183	100				450	28	22	365	22	22	315	16	22	285	16	22	260	16	22
60×60	193	95				460	32	22	375	24	22	325	18	22	295	18	22	265	18	22
,,	206	100				475	32	22	390	26	22	335	20	22	300	20	22	275	20	22
,,	225	110				500	36	22	405	26	22	350	18	22	315	18	22	290	18	22

(*i*) Table based on "RC Designer's Manual"—Concrete Association of India.
(*ii*) Stress :—50 kg/sq. cm (1:2:4 concrete) ; Punching shear—10 kg/sq. cm ; Steel—1400 kg/sq. cm.
(*iii*) The number of reinforcement bars given is for both directions. The bars should be spaced equally at the bottom of the footing in both directions in two layers.
(*iv*) Weight of the footing block and that of the column itself (approximately 10 per cent) has been added to the "Load on Column," for determining the area of the footing.



Safe Axial Loads on Short RC Square Columns

Size (outer) cm	Vertical bars nos.—dia. mm (1300 kg/sq. cm)	Links		Load Carried by Column			Area of vertical bars sq. cm
		pitch cm	dia. mm	mix 1 : 2 : 4 tonnes (40 kg/sq. cm)	mix 1 : 1.5: 3 tonnes (50 kg/sq. cm)	mix 1 : 1 : 2 tonnes (60 kg/sq. cm)	
15×15	4—12	15	5	14	16	18	4.52
	4—16	15	5	18	20	23	8.04
	4—20	15	5	24	26	26	12.57
	4—22	15	5	28	30	32	15.21
16×16	4—12	16	5	16	18	20	4.52
17×17	4—12	16	5	17	19	21	4.52
18×18	4—12	18	5	18	21	23	4.52
20×20	4—12	19	5	21	25	29	4.52
	4—16	20	5	26	30	34	8.04
	4—20	20	5	31	35	39	12.57
	4—25	20	5	40	44	48	19.63
	4—32	20	5	56	60	64	32.17
22×22	4—12	20	5	25	29	33	4.52
	4—20	20	5	35	39	43	12.57
24×24	4—12	22	5	28	33	38	4.52
	4—25	22	6	48	53	58	19.63
	8—25	22	6	74	78	83	39.27
25×25	4—12	22	5	30	36	42	4.52
	4—16	24	5	35	41	47	8.04
	4—20	24	5	40	47	52	12.57
	4—22	25	6	44	50	56	15.21
	4—25	25	6	49	55	61	19.63
	4—28	25	8	55	61	67	24.63
	8—28	25	8	87	92	98	49.26
27×27	4—20	24	5	45	52	59	12.57
	4—32	24	8	66	73	80	32.17
	8—32	25	8	104	111	118	64.34
30×30	4—16	24	5	46	54	63	8.04
	4—20	24	5	51	60	69	12.57
	4—22	28	6	55	64	73	15.21
	4—25	28	6	60	68	77	19.63
	4—36	28	8	87	95	104	40.72
	8—22	28	6	74	83	92	30.41
	8—32	28	8	117	125	133	64.34
	8—36	28	8	145	154	163	81.43

Size (outer) cm	Vertical bars nos.—dia. mm (1300 kg/sq. cm)	Links		Load Carried by Column			Area of vertical bars sq. cm
		pitch cm	dia. mm	mix 1 : 2 : 4 tonnes (40 kg/sq. cm)	mix 1 : 1.5: 3 tonnes (50 kg/sq. cm)	mix 1 : 1 : 2 tonnes (60 kg/sq. cm)	
35×35	4—20	24	5	64	76	88	12.57
	4—22	28	6	68	80	92	15.21
	4—25	8	6	73	85	97	19.63
	8—25	28	6	98	110	122	39.27
	8—28	30	8	111	123	134	49.26
	8—36	30	8	152	163	175	81.43
40×40	4—22	28	6	83	98	114	15.21
	4—25	28	6	88	104	120	19.63
	4—28	30	8	94	111	127	24.63
	8—25	30	8	113	129	145	39.27
	8—28	30	8	126	142	157	49.26
	8—36	30	8	166	181	196	81.43
	8—45	30	8	224	238	253	127.32
45×45	4—25	30	8	105	125	145	19.63
	4—28	30	8	112	132	152	24.63
	8—20	30	8	113	133	153	25.14
	8—28	30	8	143	163	183	49.26
	8—32	30	8	160	180	199	64.34
	8—36	30	8	188	208	228	81.43
	8—40	30	8	207	226	245	100.53
	8—50	30	8	278	297	315	157.08
50×50	4—28	30	8	130	156	180	24.63
	8—20	30	8	131	156	181	25.14
	8—22	30	8	138	163	188	30.41
	8—25	30	8	149	174	199	39.27
	8—28	30	8	161	187	211	49.26
	8—32	30	8	182	206	231	64.34
	12—36	30	8	254	277	301	122.15
	10—50	30	8	346	369	392	196.35
55×55	4—28	30	8	151	182	212	24.63
	8—22	30	8	159	189	219	30.41
	8—25	30	8	170	200	243	39.27
	8—28	30	8	182	214	243	49.26
	12—28	30	8	215	244	273	73.89
	12—40	30	8	310	339	368	150.80
	12—50	30	8	417	444	472	235.62

Size (outer) cm	Vertical bars nos.—dia. mm (1300 kg/sq. cm)	Links		Load Carried by Column			Area of vertical bars sq. cm
		pitch cm	dia. mm	mix 1 : 2 : 4 tonnes (40 kg/sq. cm)	mix 1 : 1.5: 3 tonnes (50 kg/sq. cm)	mix 1 : 1 : 2 tonnes (60 kg/sq. cm)	
60×60	8—25	30	8	193	228	264	39.27
	8—28	30	8	206	242	278	49.26
	8—30	30	8	225	260	295	64.34
	12—28	30	8	237	272	308	73.89
	12—32	30	8	266	301	336	96.51
	12—36	30	8	306	341	376	122.15
	12—45	30	8	384	418	452	190.85
	12—50	30	8	440	473	507	235.62
65×65	8—24	30	8	216	258	300	36.19
	12—36	30	8	333	370	412	122.15
	12—45	30	8	365	423	500	190.85
70×70	8—25	30	8	250	305	345	42.47
	12—38	30	8	375	425	472	136.08
	12—45	30	8	448	498	540	190.85

4. WATER TANKS AND SMALL RESERVOIRS

(Reservoirs have also been described elsewhere also—see Index)

Economic Proportions of Tanks of a Given Capacity. The most economic proportions for a covered circular tank are when the diameter equals the height, and that for an open circular tank when the diameter is made twice the height. The most economic shape for a closed prismatic rectangular tank is that of a cube, and for an open-topped tank when the base is square and the height is half the length of one side.

Shapes generally adopted for water tanks are circular and rectangular. Circular shape is the simplest and has the shorter wall length for a given capacity and requires least amount of material. A rectangular shape takes up less space and requires simpler shuttering.

Ordinary 1:2:4 cement concrete mix does not make an impermeable construction, therefore, a richer mix of not less than 1:1.5:3 is prescribed (with minimum quantity of cement not less than 330 kg/cu. m. and not more than 530 kg/cu. m of concrete), except in the case of thick sections and parts of structure neither in contact with water on any face nor enclosing the space above the water, or for mass work where a 1:2:4 mix may be used. Porous stone or brick aggregate should not be used for parts of structure either in contact with water on any face or enclosing the space above the water. (See under "Water-proofing Concrete".)

"Controlled Concrete" or vibrated concrete should be used where practicable with minimum amount of water. Water-proofing compounds should be added with the concrete in contact with water as there will always be some "sweating" on the outside surface even if there are no cracks. In calculating the resistance of the concrete to cracking it shall be assumed that the concrete is capable of sustaining tensile stress as specified in Table I but in calculating the strength of the structure no tensile stress shall be assumed to exist in the concrete.

Practical Rules

(*i*) No reinforced concrete wall or floor slab shall be of thickness less than 25 mm+1/40 depth below top water level with a minimum value of 10 cm.

(*ii*) The minimum reinforcement in walls, floors and roof shall be not less than 0.3 per cent in each direction at right angles for sections upto 100 mm thick. For sections greater than 100 mm thick and less 450 mm, the minimum reinforcement in each of the two directions shall be linearly reduced from 0.3 per cent for 100 mm to 0.2 per cent for 450 mm thickness of the gross cross-section. This may be 0.25 per cent and 0.16 per cent when high yield deformed bars are used, instead of 0.3 and 0.2 per cent.

(*iii*) The bars used should be of the smallest diameter practicable.

(*iv*) Distribution reinforcement should be put in where other cross reinforcement is not provided, in both faces of the wall (where the main reinforcement is on both faces) and shall consist of small diameter bars 10 mm or less at fairly close spacing (say, 25 to 30 cm centres). Distribution rods placed outside the main rods are easier to fix and are more effective.

(*v*) The height of any layer of concrete shall not exceed 200 cm unless precautions are taken to ensure thorough consolidation throughout the height of the layer. Super imposed layers shall be cast at as short intervals as practicable.

(*vi*) Water tanks may be insulated against external temperature changes : by covering with earth; by lagging externally with timber, fibre boards or other materials of low thermal conductivity and filling the air space with saw dust. Encasing the tanks with walls, with or without an air space, will also reduce temperature effects considerably. (See also at end of "Overhead Tank"—"Core Walls".)

(*vii*) Surfaces in contact with water should be dense and smooth. In order to attain this the formwork should be removed as soon as practicable and the concrete surface treated as follows :—

All projecting imperfections should be rubbed down flush with carborundum stone and thoroughly washed with water. Then as a separate operation a 1:1.5 cement and sand mixture should be worked smooth into the pores over the whole surface with a float. No more material should be left over the concrete face than is necessary to completely fill the pores. Some water-proofing compound should preferably be incorporated into the mixture.

(*viii*) Thick timber shuttering should be avoided as it prevents easy escape of heat of hydration of cement from the concrete mass and produce cracks. The risk of cracking can also be reduced by reducing the restraints on the free expansion or contraction of the structure.

(*ix*) Structures may be lined with impervious materials such as asphalt, tiles, to prevent percolation or chemical action.

Capacity of Circular Tanks

Dia. in metres	Capacity in litres per metre depth	Dia. in metres	Capacity in litres per metre depth	Dia. in metres	Capacity in litres per metre depth
1.00	785	3.00	7069	5.00	19635
1.25	1227	3.25	8296	5.25	21648
1.50	1767	3.50	9621	5.40	23758
1.75	2405	3.75	11045	5.75	25957
2.00	3142	4.00	12566	6.00	28274
2.25	3976	4 25	14186	7.50	44179
2.50	4909	4.50	15904	9.00	63617
2.75	5940	4.75	17721	10.00	78540

Permissible Tensile Stresses in Concrete for Calculations Relating to Resistance to Cracking

Grade of Concrete mix	Permissible Stresses in kg/sq. cm		
	Direct Tension	Tension due to Bending or Flexural tensile stress	Shear s/b jd
M-150—1:2:4	11	15	15
M-200—1:1.5:3	12	17	17
M-250—1:1:2	13	18	19

These permissible stresses are according the IS code.

Permissible Stresses in Steel Reinforcement to Prevent Craking of Concrete

	Plain Mild Steel Grade I	Deformed Bars or spot welded	High Yield Deformed Bars
	kg/sq. cm	kg/sq. cm	kg/sq. cm
Steel placed within 225 mm from water face	1000	1125	1500
Steel placed beyond 225 mm from water face	1250	1400	1900

Although reasonably low allowable steel stresses are desirable, but these should not go below what is prescribed in the table, because the construction will not be economical. Furthermore, very low steel stresses *increase* the shrinkage stresses.

Total pressure in the curved surface (semi-circular) is the same as that in the diametrical plane which is=w×h×D. The section of the wall of such a tank is in direct tension and this tension or pressure is resisted by the two sides of the wall ring, which is called *hoop tension, ring tension or circumferential tension.*

Circumferential tension T at any depth h :

T=1/2 w×h×D and this tension is to be taken up wholly by the reinforcement.

$$\text{Therefore As} = \frac{T}{fs} = \frac{w\times h\times D}{2\times 1000} = \frac{1000\times h\times D}{2\times 1000} = \frac{hD}{2} \text{ sq. cm}$$

As is cross-sectional area of steel to resist the circumferential tension per metre height of wall. fs is allowable tensile stress in steel for ring tension.

The walls substain the hydrostatic pressure of the water and are subjected to triangular loading varying from zero at the top to a maximum at the bottom.

Sufficient thickness of concrete is provided to resist this tension against cracking, and steel is provided for strength.

In the tension zone of a reinforced concrete structure the steel does nor fully take the stress until the concrete has first cracked by having exceeded its tensile stress limit. In water retaining structures it is not desirable to let the concrete crack as it would lead to leakage. A reasonable factor of safety against cracking and leakage has to be kept.

Thickness of wall t to resist the circumferential tension is based on the composite section, *i. e.* area of concrete plus equivalent area of steel, *viz.*,

$$T=1/2\ w\times h\times D=[100\times t+As\,(m-1)]$$

12 is permissible tensile stress for 1:1.5:3 mix in direct tension. Modular ratio *(m)* for 1:1.5:3 mix is 13.3.

100 is the height of wall considered in cm under pressure; As is area of tensile steel in sq. cm

Empirical rules for determining approximate thickness t in cm : minimum thickness is 15 cm :

(*i*) $t=8+\frac{h}{40}$ h is depth of water in cm ;

(*ii*) t=3 H+5 H is depth of water in metres.

Thickness may be reduced towards the top and walls made tapering according to the depth of water.

Steel at other depths can be determined similarly and reduced towards the top. Hoop reinforcement may be placed in the centre of the wall in small tanks with thin walls and, partly on the inner and partly on the outer side in big tanks with thick walls.

Pre-stressing. The walls of cylinderical tanks may be reinforced by hooping external to the walls, such hooping being stressed in tension by turn-buckles or otherwise, so as to produce circumferential compression in

the concrete prior to filling the reservoir. This method enables the concrete to be placed under initial compression, and as there is no tendency to crack the provision of joints is unnecessary.

Provide vertical bars on the inside of the hoops as distribution steel. Small tanks with monolithic floor and walls may, however, be designed according to the above rules to simplify calculations, taking additional precautions for making the wall and floor joint perfect as stated hereafter.

Walls rigidly fixed to the floor in Circular Tanks

The wall of a circular tank may be fixed rigidly to the base or it may be set in grooves in the base. In both the cases it is taken as "fixed" at the base and designed as a vertical cantilever. The bending moment varies from a maximum at the base to zero at the top of the wall. The intensity of pressure is w×h at the base, and the maximum bending moment is

$$=(w\times h\times h/2)\ 1/3h=1/6\ w\times h^3$$

Fixed base gives an economical design. When the wall of a circular tank is rigidly fixed to the base (monolithically) the horizontal ring tension is combined with vertical bending. The values of the restraint moment and the distance at which maximum hoop tension occurs depends on a number of factors and it involves laborious calculations to work out exact stressess. The following simple method based on Reissner's theory has been suggested for practical purposes :

(*i*) Max : negative bending moment (called restraint moment) at the bottom of the tank wall=a×wh³. From this the vertical reinforcement is worked out. (Value of "a" is given in the table at page 8/73.

When steel and concrete are assumed to act together, for checking the tensile stress in concrete for avoidance of cracks, the tensile stress in the steel will be limited by the requirement that the permissible tensile stress in the concrete is not exceeded, so the tensile stress in steel shall be equal to the product of modular ratio of steel and concrete, and the corresponding allowable tensile stress in concrete.

Permissible Stresses in Concrete

Design of reinforced concrete members should be made ignoring the tensile resistance of concrete. Additionally, it should also be ensured that the caculated tensile stress on the liquid retaining face of the equivalent concrete section (after allowing for the steel area in equivalent concrete units) does not exceed the permissible limits prescribed below, assuming in the calculations that the entire section of the concrete participates in resisting the direct and flexural loads. The premissible limits of tensile stress in the concrete for calculations relating to resistance to cracking will naturally provide a much smaller margin of safety against ultimate tensile strength of concrete because the consequence of cracking are usually much less serious than those of structural failure.

The permissible tensile stresses due to bending apply to the face of the member in contact with the water. In members less than 225 mm thick and in contact with the water on one side, these permissible stresses in bending apply also to the face remote from water, Plain concrete water retaining structures or members may be designed against structural failure by allowing tension in plain concrete as per the permissible limits for tension in bending given in Table No. 1.

Area of steel $= \dfrac{BM}{ft \times j \times t}$ Where :

ft = permissible stress in steel for ring tension ; j = lever arm factor ;
t = effective thickness of concrete wall. (Approximate thickness of wall is assumed to arrive at the value of the factor "a"—as given under empirical rules.)

"a" is a factor—values given in the table at page 8/73.

Equivalent vertical reinforcement is provided on the water side of the wall for a distance of about one-sixth the height from the bottom. Alternate bars may, however, be continued up to the top. To meet the positive bending moment on the outer face, which is only about 1/3rd of the negative restraint moment, half the bars may be provided for the full height of the wall. (The vertical reinforcement is very light, therefore, full height bars have been suggested, to which the horizontal bars can be tied.)

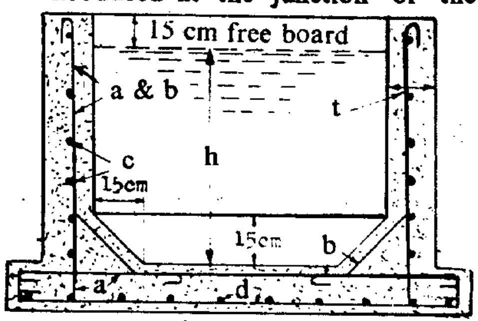
(*ii*) Maximum hoop tension :

$T = b \times \dfrac{w \times h \times D}{2}$ "b" is a factor, value given in the table at page 8/73

(*iii*) Position of maximum hoop tension above the base $= c \times h$, "c" is a factor. Max. hoop tension in a fixed base and free top construction generally occurs at a point 0.6h from the top.

Area of steel for hoop tension should be worked out from T, *i.e.*, T/permissible stress in steel for hoop tension. Area of steel thus obtained is gradually reduced towards the bottom as the ring tension is replaced by cantilever effect. A reduction is also made towards the top as the hydraulic pressure decreases with a reduction in depth. (The spacing of rods should not be more than 3 times the thickness of wall).

If the base is not perfectly fixed with the wall the design will not be safe. A splay or haunch should be introduced at the junction of the wall and base so as to increase the stiffness of the corner. The splay may be 15 cm to 23 cm. Additional horizontal steel must be added at the corners in L shape through the walls and floor, and also sufficient haunch bars provided. Similarly, horizontal reinforcement and haunch bars must be provided at the vertical edges.



In shallow tanks of large diameters, the tendency of the walls is to act more like vertical cantilevers and hoop stresses are small. When the height is very great compared to the diameter, there is only hoop action

and no cantilever action. Where the diameter is greater than $24\sqrt{h}$, it is economical to design the walls as cantilever beams anchored to the floor.

Illustrative Example for a circular tank :
Depth of water H—3 metres; Diameter—7.5 metres.

Thickness t in cm $= 3 \times H + 5 = 14$ cm. Take t as 15 cm
h/t = 300/15 = 20 (here h and t are in cm)
H/D = 3.0/7.5 = 0.4 (here H and D are in m)
(*i*) Restraint moment $= a \times w \times h^3 = 0.014 \times 1000 \times 3^3$ kg m
= 37800kg cm
0.014 is the value taken from the table, 1000 is weight of water.
Taking effective thickness as 12 cm, lever arm $= 0.80 \times 12 = 10$ cm.

Reinforcement required $= \dfrac{37800}{1000 \times 10} = 3.78$ sq. cm

(Min : requirement according to the table given below is 4.30 sq. cm)
Use 10 mm dia. bars at 18 cm on centres as vertical steel.
(*ii*) Max. hoop tension

$= \dfrac{0.57 \times 1000 \times 3 \times 7.5}{2} = 6{,}400$ kg

Reinforcement with allowable tensile stress of

1000 kg/sq. cm $= \dfrac{6400}{1000} = 640$ sq. cm

12 mm dia. bars at 17.5 cm on centres as hoop steel will suffice.
Use 0.6 steel of above, at the bottom, for fixidity between the wall and the base.

(*iii*) Position of max : hoop tension "L"
$= c \times H = 0.4 \times 3 = 1.2$ m above the base
0.4 value for c is taken from the table.
(Example based on "RC Designer's Manual".)

Rectangular and Square Tanks.—Rectangular or square overhead tanks are not generally economical or suitable except for small sizes. In plane walls the water-pressure is resisted by both vertical and horizontal bending moments.

Factors for Circular Tank Design According to Reissner's Theory

Factor	h/D	0.2	0.3	0.4	0.5	1	2	3
a	h/t=10	.045	.030	.024	.020	.012	.006	.005
	h/t=20	.028	.018	.014	.012	.006	.005	.003
	h/t=30	.023	.012	.010	.008	.005	.003	.002
b	h/t=10	.30	.38	.43	.47	.61	.73	.77
	h/t=20	.38	.51	.57	.61	.72	.81	.84
	h/t=30	.50	.60	.64	.68	.77	.85	.87
c	h/t=10	...	.55	.50	.45	.38	.30	.27
	h/t=20	.50	.44	.40	.37	.28	.22	.21
	h/t=30	.44	.38	.34	.31	.24	.19	.18

Rectangular tanks with walls fixed at the base

Longitudinal walls are designed as vertical cantilevers fixed at the base, for the water pressure varying from a maximum at the bottom to zero at the top. The bending moment at any point is $wh^3/6$. This moment causes tension in the face next to water. Provide some extra steel on the outer face as well.

Horizontal steel in the long walls must be sufficient to take the tension due to water-pressure tending to force the long walls away from the end walls. This reaction is considered maximum at a depth of about 3/4th from the top which is : $T=3/4\text{th}\times w\times 1/2$ short walls length, for unit height of long wall. Area of steel required to resist this horizontal tension=T-permissible stress in steel per unit of height. This can be reduced both above and below the maximum tension point.

Check this with the quantity of distribution steel required which should be at least 20 per cent of the main steel—see under "Practical Rules" and provide whichever is greater.

End walls (or shorter walls) are designed as slabs spanning horizontally between longitudinal walls to resist a positive outward bending moment of $wL^2/16$ at midspan and a negative (inward) moment at each corner of $wL^2/12$. Maximum bending moment at any depth h

$$=\frac{w\times h\times L^2}{12 \text{ or } 16} \text{ per unit of height}$$

w is weight of water, and L is effective span.

Maximum bending moment occurs when depth of water in the tank is equal to half the length of the short wall.

15 cm×15 cm fillets should be provided to all corners to give added strength and extra horizontal reinforcement on the inside face for a distance of about one metre in both the long and short walls. See also "Corners and Edges" under Circular Tanks.

For reservoirs up to about 3.7 m in depth the walls may take the form of a vertical slab diminishing in thickness from base to top and those of greater depth may be provided with vertical conterforts at about 2.5 metres centres and the wall slabs designed to span horizontally between them.

Non-monolithic walls. Where the sides are not monolithic with the floor as in the case of large open reservoirs, sliding joints are provided; the design of walls is then identical with that of retaining walls.

No beams are required under walls of reservoirs as the walls themselves form deep girders and should be reinforced as lintels as explained in the Section "Foundations," if no other equivalent horizontal reinforcement has been provided.

Floors and Foundations of Tanks. Where plain or reinforced concrete foundations or floors are founded on earth, a mass-concrete screed of mix 1:3:6 or 1:2:4 not less than 75 mm thick shall first be spread over the ground and covered with a sliding layer of plastic sheet or other suitable material.

Floors of tanks resting on ground are designed for the weight of the structure and water stored pressing downwards and the reaction of the soil pressing upwards. The floor slab requires only sufficient reinforcement to enable it to span over possible weak patches of the ground.

It is common practice to make a monolithic connection between the walls and the base slab. The base slab may be extended beyond the walls (as shown in the sketch) so that the weight of the earth on the projecting part of the base may assist in preventing the tank from being pushed up due to up-thrust when the water-table rises.

A bending moment of $wD^2/24$ is taken for circular slabs. The steel computed from this bending moment is provided in both directions. A floor slab thickness of 15 cm to 20 cm is common with reinforcement of 10 mm dia. bars at 15 to 22 cm centres in both directions in the bottom of the slab. If two mats, one top and one bottom, are laid, one mat should be staggered relatively to the other. Where the soil is weak or there is possibility of ground water pressure, the base slab is projected as shown in the illustration. The floor should be laid in two superimposed layers of which the bottom layer may comprise or replace the mass-concrete screed described above.

In monolithic walls the floor and wall joints should be in line. The slabs of the different layers shall be arranged to break joint. Joints shall be filled with bitumen. (See under "Joints in Concrete Structures" in the following pages and also Joints in RC Roads).

Surfaces in contact with water may be treated during the concreting operation with dry cement evenly distributed thereon and worked in with a steel trowel on the initial set of the concrete so as to produce dense and smooth surface.

Joints. (See also under "Joints in Concrete Structures" in the following pages). Joints should be provided in concrete walls at a maximum spacing of 7.5 metres in reinforced walls and 6 metres in unreinforced walls, which should be with complete discontinuity in both reinforcement and concrete, with very small initial gap between the faces. In reinforced concrete floors, joints should be spaced at not more than 7.5 metres apart in two directions at right angles. The wall and floor joints should be in line except where sliding joints occur at the base of the wall. For floors with only nominal percentage of reinforcement, the concrete floors should be cast in panels with sides not more than 4.5 metres. Floor layers should be placed in "squares" or segments. These are more of contraction joints in small structures than expansion joints which need be provided only 30 to 40 metres apart. Metal strips should be provided in both construction and contraction joints. In hydraulic structures, V- shaped sealing slots are considered more advantageous than plane vertical slots.

A suitable type of a joint (called "strip joint") in walls consists of a steel, copper or zinc strip 20 cm wide and about 1.5 mm thick, fixed vertically in the joint. The strip has a crimped cross-section (*i.e.*, corrugated in the centre) as shown in Fig. B under "Joints in Concrete

Structures". 12 mm dia. holes at 20 cm centres are punched near the edges of the strip to securely anchor it to the concrete. Steel strips should be coated with bitumen. Rubber "water-stops" having a dumb-bell section have also given good results. The walls should be constructed in alternate panels with as long a pause as practicable before the concrete is placed in the intervening panels so that they may contract fully.

Horizontal construction joints which are due to temporary cessation of placing of concrete should be avoided as far as practicable by concerting the walls in as few lifts as possible. Such joints may be made with a rebate or V—groove. The existing surface should be hacked and grouted with rich cement mortar in 5 cm thick layer.

Sliding joints are made at the base of walls where walls are not rigidly fixed to the floor or the ground There is complete discontinuity in both reinforcement and concrete and a layer of bitumen is interposed which facilitates movement in the plane of the layer. See the illustration under "Overhead Tanks", and Fig. E. under "Joints in Concrete Structures" in the following pages. "Tongued and grooved" joints are also made between walls and floor slabs. Free sliding joints should be provided between the roof and walls of large reservoirs or where the temperature movement is abnormal; this can be done by interposing a lead or bituminous sheet.

Reservoirs built on water-logged soils or areas subject to floods are apt to fail by floating due to upward pressure when the reservoir is empty. This can be guarded against by making the reservoir sufficiently heavy to resist upward pressure, (the minimum weight of the reservoir should exceed the uplift pressure by at least 20 per cent) or by providing underground drainage where site and soil conditions permit.

Notes for tank tables at pages 8/77 and 8/78.

(*a*) Thickness of concrete and reinforcement given are a little more than required theoretically.

(*b*) Horizontal reinforcements can be reduced towards the top to about 2/3rd of the bottom reinforcement, and thickness of walls can also be diminished towards the top.

(*c*) Light reinforcement should be provided at the top in the floor slabs.

(*d*) Small tanks up to 9000 litres capacity can be built overhead supported on peripheral beams, and beyond this capacity floor slabs should be designed according to the arrangements of the supporting beams. If the beams form a square panel, each beam takes one-fourth the total load and has a triangular loading. The BM at the centre of each such beam will be $=WL/6$. The floor slab should be designed two-way reinforced. Small tanks of 10,000 to 90,000 litres capacity can be made where floors are supported on firm ground.

(*e*) Bars (*a*) and (*b*) are alternate and the distance between each bar will be half of the spacing given.

Design Data for Small RC Circular Water Tanks

Capacity	Dia. (inside)	Depth including free board	Thickness of wall and floor	Reinforcement					
				Wall				Floor	
				Horizontal Steel at bottom half depth		Vertical Steel		Bars each way	
				Bars c		Bars a & b		Bars d	
				Bar dia.	Spacing	Bar dia.	Spacing	Bar dia.	Spacing
Litres	m	m	cm	mm	cm	mm	cm	mm	cm
2,000	1.75	1.00	10	10	20	10	20	10	20
3,500	2.00	1.40	10	10	20	10	20	10	20
5,000	2.25	1.45	10	10	20	10	20	10	20
7,000	2.50	1.55	10	10	20	10	20	10	20
9,000	2.75	1.70	13	10	20	10	20	10	20
14,000	3.00	2.10	13	10	20	10	20	10	20
23,000	3.75	2.25	15	10	17	10	20	10	20
34,000	4.25	2.55	15	10	17	10	20	16	20
45,000	4.75	2.65	18	12	20	12	30	12	30
90,000	6.00	3.25	18	12	12	12	20	12	20

See Notes at page 8/76

Design Data for Small RC Square Water Tanks

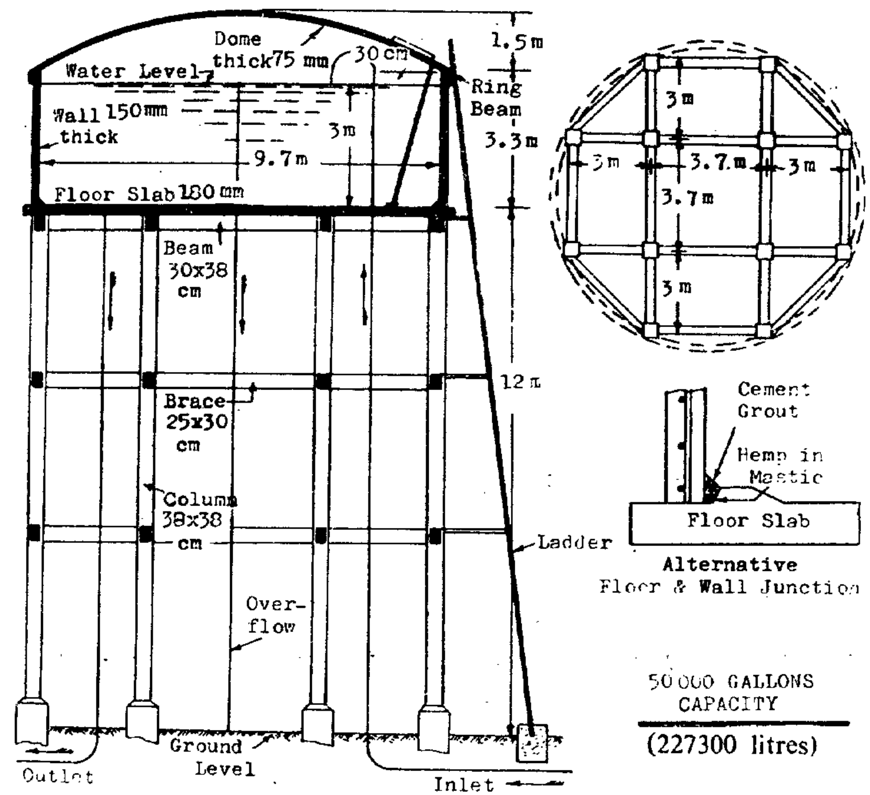
Capacity	Length × Breadth (inside)	Depth including free board	Thickness of wall and floor	Reinforcement – Wall – Horizontal Steel – Bars c – Bar dia.	Horizontal Steel – Bars c – Spacing	Vertical Steel – Bars a & b – Bar dia.	Vertical Steel – Bars a & b – Spacing	Floor – Bars each way – Bars d – Bar dia.	Floor – Bars d – Spacing
Litres	m	m	cm	mm	cm	mm	cm	mm	cm
2,000	1.5×1.5	1.00	10	6	15	6	15	6	20
3,500	1.8×1.8	1.25	10	6	15	6	15	6	20
5,000	2.0×2.0	1.40	10	6	15	6	15	6	20
7,000	2.3×2.3	1.50	11.5	10	22	10	22	6	20
9,000	2.5×2.5	1.60	11.5	10	20	10	15	6	20
14,000	2.8×2.8	2.00	13	10	18	12	18	6	14
23,000	3.5×3.5	2.10	15	10	16	16	20	6	12
34,000	4.0×4.0	2.30	16	10	15	16	18	6	10
45,000	4.2×4.2	2.70	18	12	20	16	15	6	10

See Notes at page 8/76

Overhead Tanks

The illustration shows general arragements for an overhead tank and is a typical design for 50,000 gallons (227300 litres) capacity. (See also Elevated Tanks under "Water Supply"). There are numerous designs which are followed based more or less on the principles shown in the illustration. An access shaft about 1.5 metres diameter can be made through the centre of the tank with walls taken up to the roof. Stairs can be built from ground level to the top for entry into the tank through the shaft. Partition walls are built in big size tanks. Roof may be made flat instead of circular. A cantilever gallery about one metre wide (projection of the floor slab) can be provided all round the tank. Bottoms are now generally made flat instead of hemi-spherical. The outer pillars can be kept a little inside so as to give a cantilever projection to the whole tank ; it adds to the appearance of the structure.

Foundations are designed according to the bearing capacity of the soil. If the soil is weak, raft foundations of RCC can be made over the whole area. There should be no possibility left for any settlement.



(227300 litres)

Freeboard 30 cm

(Freeboard may not be provided in all cases. If allowed, additional weight of this water should be taken).

Height above ground level 12.2 m

Wall thickness 152 mm

Floor slab : (for 3 m depth of water) 178 mm

Central slab has the maximum stress. Design for full water load as two-way reinforced slab fixed on all sides. (The thickness should be increased by 2.5 cm if 1:2:4 concrete is used. Reinforce with 16 mm dia. bars, 100 mm c/c both-ways. For side slabs reinforcement may be reduced.

Where floor slab is projected as cantilever, reinforcement at top for the portion will be necessary.

Roof (dome) thickness 76 mm

Provide 6 mm dia. bars 22 cm c/c radially and circumferentially. Binding wire can be No. 16 gauge.

Floor Slabs of Overhead Tanks. For circular tanks supported on beams monolithic with the floor slab, a bending moment of $wL^2/26$ may be taken in both directions. Alternate bars of each layer should be bent up over the beams to serve as negative reinforcement. Some of the vertical rods in the walls should be bent round into the floor slab. The floors should be cast in panels of sections as described above.

A flat roof slab may be designed as simply supported on the walls and reinforced both for bending and for tension due to the water pressure forcing the walls outwards away from the roof.

Roof Ring 23 cm × 30 cm

Provide 4 Nos. 22 mm dia. bars and 10 mm dia. stirrups at 22 cm c/c.

Beams : Central beams 3.66 m span 30 cm × 40 cm — rib portion.*

Design central beams as continuous with WL/12 ; for this size of beam reinforcement consisting of 4 bars 28 mm dia. at top and bottom is necessary. If the depth of the beams is increased, lesser reinforcement is required. All beams are generally made of the same size. For outside beams of 3.66 m span reinforcement of 4 bars 25 mm dia. at top and bottom is required ; while for outside diagonal beams, 4 bars 20 mm dia. at bottom and 2 bars at top are necessary.

Columns ... ... 38 cm × 38 cm

Reinforce with 8 bars 16 mm dia. and 6 mm helical binders at 75 mm c/c, as these columns are subjected to bending in addition to vertical load.

*Beams are made monolithic with the floor slab and considered to act as T-beams. Top reinforcement will be placed in the floor slab portion.

Bracings ... ... 25 cm × 30 cm

Reinforce with 4 bars of 16 mm dia. at top and 4 bars at bottom, and 6 mm dia. stirrups at 22 cm c/c.

Provide deep haunches at junctions of braces with columns, with additional (diagonal) reinforcement.

Reinforcements of columns, braces, floor slab and ring beam etc., should be properly anchored into the jointing members.

Ladder-Iron

Can be made of the following size—

Sides— Plates 63 mm × 10 mm

Rungs— 45 cm wide—20 mm dia.

Another arrangement for overhead tanks in hot climates can be with "core wall", that is, 10 cm thick brick wall in cement is made outside and inside the concrete wall. In this case the concrete wall is of the same design as for an RC tank but a leaner mix can be used, and is generally monolithic with the bottom. The inside brick wall can rest on the wall and floor-joint splay and outside wall on the floor projection.

Design Data for RC Circular Water Tanks

Capacity 1000 Litres	Dia. "D" (inside) m	Depth of Water m	Thickness of Wall "t" cm	Reinforcement				
				Horizontal Hoop Steel			Vertical Steel	
				Height "L" m	Bar dia. mm	Spacing cm	Bar dia. mm	Spacing cm
450	11.30	4.50	25.00	1.89	16	14.5	12	14.0
360	10.10	,,	22.50	1.68	16	15.0	12	15.5
225	8.00	,,	20.00	1.58	16	17.5	12	17.5
135	6.20	,,	17.50	1.35	12	11.5	12	18.5
450	12.65	3.50	22.50	1.82	12	12.0	12	15.0
360	11.30	,,	20.00	1.61	12	12.5	12	16.5
225	8.95	,,	17.50	1.40	12	13.0	10	15.5
135	6.95	,,	17.50	1.26	12	15.0	10	16.0
450	13.80	3.00	20.00	1.80	12	14.5	12	16.0
360	12.30	,,	17.50	1.50	12	15.5	12	18.0
225	9.75	,,	15.00	1.29	12	15.5	10	15.5
135	7.55	,,	15.00	1.17	12	17.0	10	18.0
450	15.20	2.50	17.50	2.00	10	13.5	10	13.5
360	13.55	,,	17.50	1.75	10	14.5	10	15.0
225	10.72	,,	15.00	1.40	10	15.5	10	16.5
135	8.30	,,	15.00	1.20	10	16.5	10	18.0

Based on "RC Designer's Manual"—The Concrete Association of India.

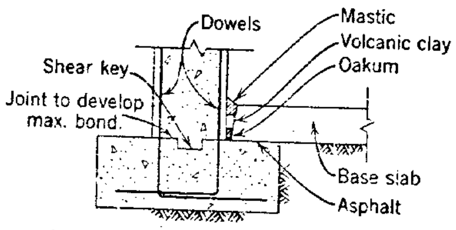
"L" is the position of max : hoop tension above bottom of tank.

Tanks with fixed covers may have a circular top slab or, if the diameter is large, one or more columns spaced evenly throughout the area may be provided; roofs designed as flat slabs. Such tanks, with their thin shell tops and bottoms are very economical.

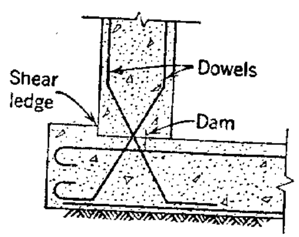
Design of Circular Tanks. Water exerts at any point a uniform radial pressure in all directions in diametrical plane at right angles to the curved surface of the cylinder. The pressure increases as the depth below the surface and is=w×h at the bottom of h. Where : w is the weight of a cubic unit of water, h is the depth below the surface of water, D is the internal dia. of the tank,

Tanks Built Below Ground Level. Walls are designed as cantilever retaining walls to resist the bending moment due to pressure of earth when the tank is empty and also when the tank is full of water with no earth pressure of the back-fill. Walls are thus reinforced in both the faces with vertical (main) reinforcement. See under "Retaining Walls" in Section 7. The walls of rectangular tanks are similar to long straight retaining walls with counterforts.

The base slab may be designed as a one-way slab or a slab supported on all four sides. A base slab may be subject to considerable upward pressure, which may be caused by high water-table. In such cases the buoyancy of the structure should be computed and proper provision made for the anchoring of the structure. Proper keyway and dowels should be used between the slab and the side walls.



Base details of Fixed Walls



Base details of Hinged Walls

5. MISCELLANEOUS STRUCTURES

Design of Staircases

Staircases in general have been described in the Section—"Masonry Structures". Stairs are of several types. The most common types are : (*i*) A sloping slab spanning from one floor to another floor or from end to a landing, and supported on the two side walls or on sloping stringer beams ; (*ii*) Separate slabs for each step attached to one central sloping beam ; (*iii*) Steps cantilevered from a side wall (one end fixed into the wall) where the wall is of ample thickness : (*iv*) Spiral stairs with slabs cantilevered out from a central column ; (*v*) Free spanning spiral stairs.

Stairs, landings and cantilever access balconies may be designed for the imposed or live loads allowed on the floors served by them, or for a load of 300 kg/sq. m for residential and office buildings, and

500 kg/sq. m for public places and warehouses. Service stairs for maintenance, such as in water tanks, backside courtyards of residences, cat-walks, etc., may be designed for 150 kg/sq. m. See Section 6—"Super-imposed Loads on Floors." To the imposed load, the dead-load of the structure is added, acting at right angles to the flight.

The load is considered on the horizontal projection "L" of the sloping stairs (flight). The span for determining the bending moment is also this horizontal projection. If trimmer beams are omitted, the slab has to be designed for a wall to wall span—inclusive of the landing lengths.

The slab portion under the (saw-tooth) projections of steps is called "waste" and its depth is measured normal to the slope of the slab. This "waste" is considered to take the whole load and is designed as an ordinary slab partially fixed with WL/10 where ends are built into walls, and with WL/12 where ends are monolithic with the transverse beams at bottom and top.

It is common practice to provide transverse beams at top and bottom—at junctions of landing and sloping flight. Where transverse beams are omitted, flight is a continuation of the landing or landings. Landings usually have the same size as the "waste" slab. Where a landing serves two flights of stairs at right angles to each other, it should be reinforced in two directions.

In most of the cases, slabs supported transversely over logitudinal stringer beams are more economical than the logitudinal slabs.

It has been stated earlier that the "pitch" or slope of a staircase should not be more than 42 deg.—30 deg. pitch is common. Treads usually adopted are 23 cm to 30 cm with risers of 15 cm to 19 cm. For concrete steps, normally adopted tread width is 27 cm with a riser height of 18 cm. The following proportions may be taken :—

Tread—cm	23	25	28	30	33	35	38
Riser —cm	19	17	16	15	14	12	11

(See also pages 7/45, 21/12)

Where the waist slab is supported transversely on end walls or beams, the total thickness of the waist may be 6 cm up to 110 cm wide stairs, 7 cm up to 140 cm, and 9 cm up to 185 cm wide stairs with ordinary loads. Reinforcement may be 10 mm dia. bars 15 cm apart laid across, and transverse bars 6 mm dia. laid 22 cm apart parallel to the supporting walls or beams.

Horizontal loads on balustrades or parapets for stairs may be taken at 75 kg/m run.

Cantilever steps are designed for 150 kg. live load at the free end of each step. Each step slab is embedded for 25 to 30 cm inside wall for anchorage.

A cantilever 1:2:4 RC step slab/1.2 m projecting outside the wall, thickness of step slab 120 mm and width 350 mm will require 2 bars 10 mm each on top side with cross bars 5 mm at 30 cm c/c.

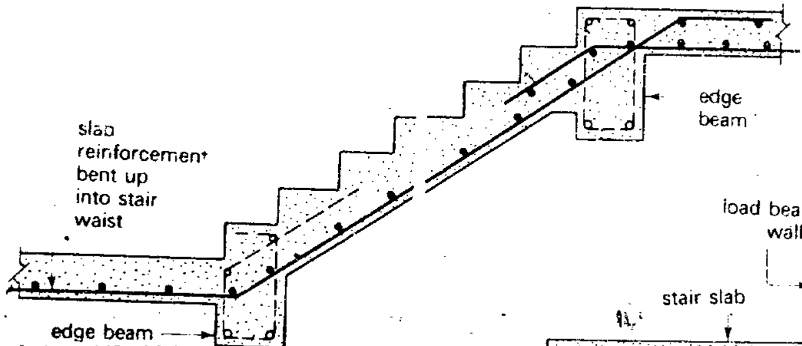
The following sizes of RC waists may be taken for the stairs supported longitudinally on both sides either by walls or by stringer beams in residential, office and light warehouse buildings :—

Span L metres	Thickness of Waist mm	Spacing of Reinforcement Bars Main-Logitudinal Bars		Transverse Bars	
1.5	100	10 mm	@15 cm c/c	6 mm	@30 cm c/c
2.0	100	10 ,,	13 ,,	6 ,,	25 ,,
2.5	115	10 ,,	10 ,,	6 ,,	22 ,,
3.0	130	12 ,,	14 ,,	6 ,,	15 ,,
3.5	150	12 ,,	12 ,,	10 ,,	30 ,,
4.0	170	16 ,,	14 ,,	10 ,,	25 ,,
4.5	200	16 ,,	12 ,,	10 ,,	17 ,,
5.0	225	16 ,,	10 ,,	10 ,,	12 ,,
5.5	255	16 ,,	9 ,,	10 ,,	12 ,,
6.0	270	16 ,,	7 ,,	10 ,,	12 ,,

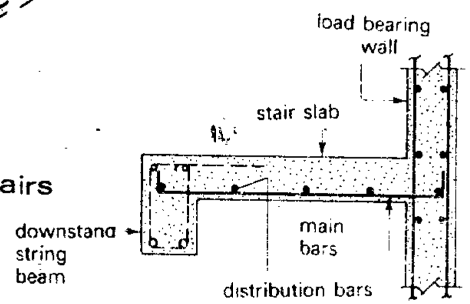
Sometimes pre-cast steps consisting of a tread only are built supported on two stringer beams. Each step slab is 32 cm wide and is reinforced with 3 longitudinal bars 6 mm dia. Each upper step projects about 4 cm over the end of the lower step. Stringer beams are designed as ordinary rectangular beams with half the load of the steps (half on each beam). Length is taken as the horizontal span. Stringer beams are notched for resting the step slabs on them.

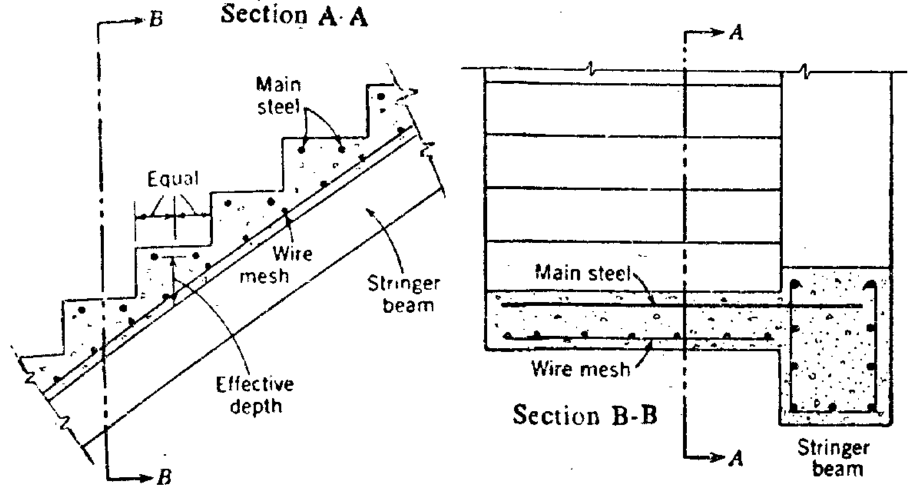
Half the bars should be bent up at the ends of the slab for the negative bending moments.

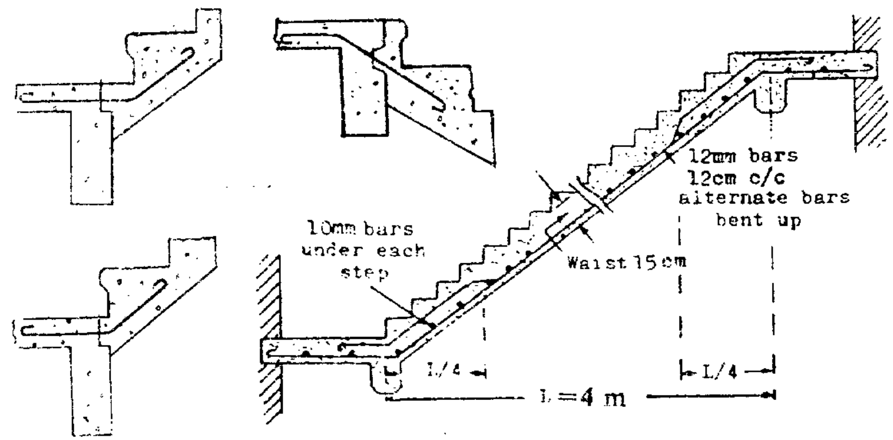
Where a central stringer beam is provided and the steps are cast with it, the stringer beam may be designed as a T-beam.

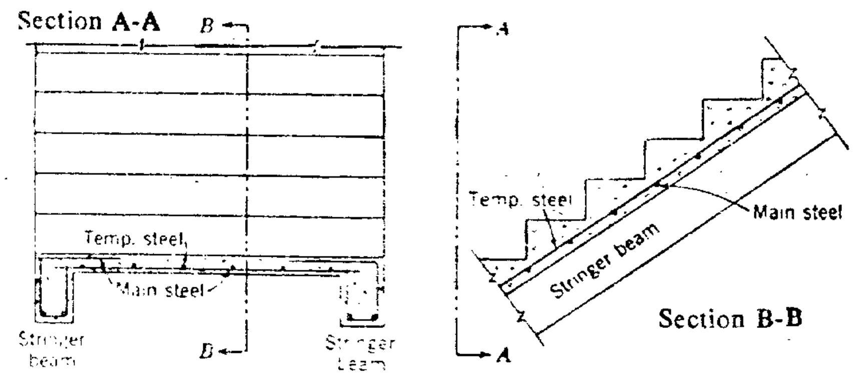


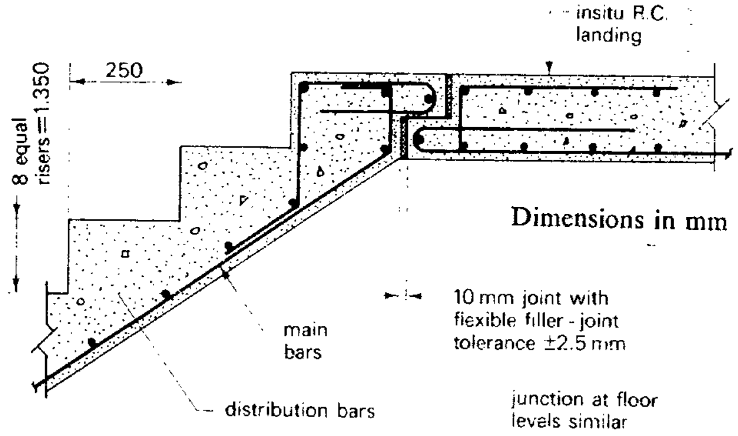
Concrete Straight Flight Stairs



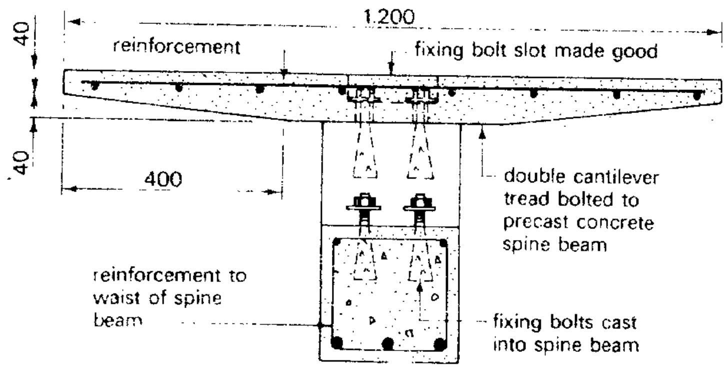




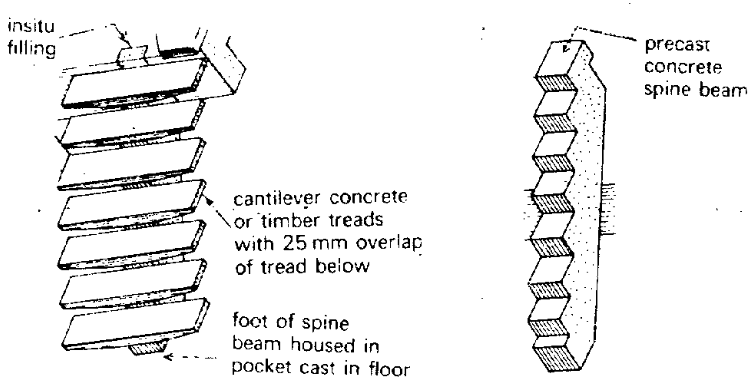


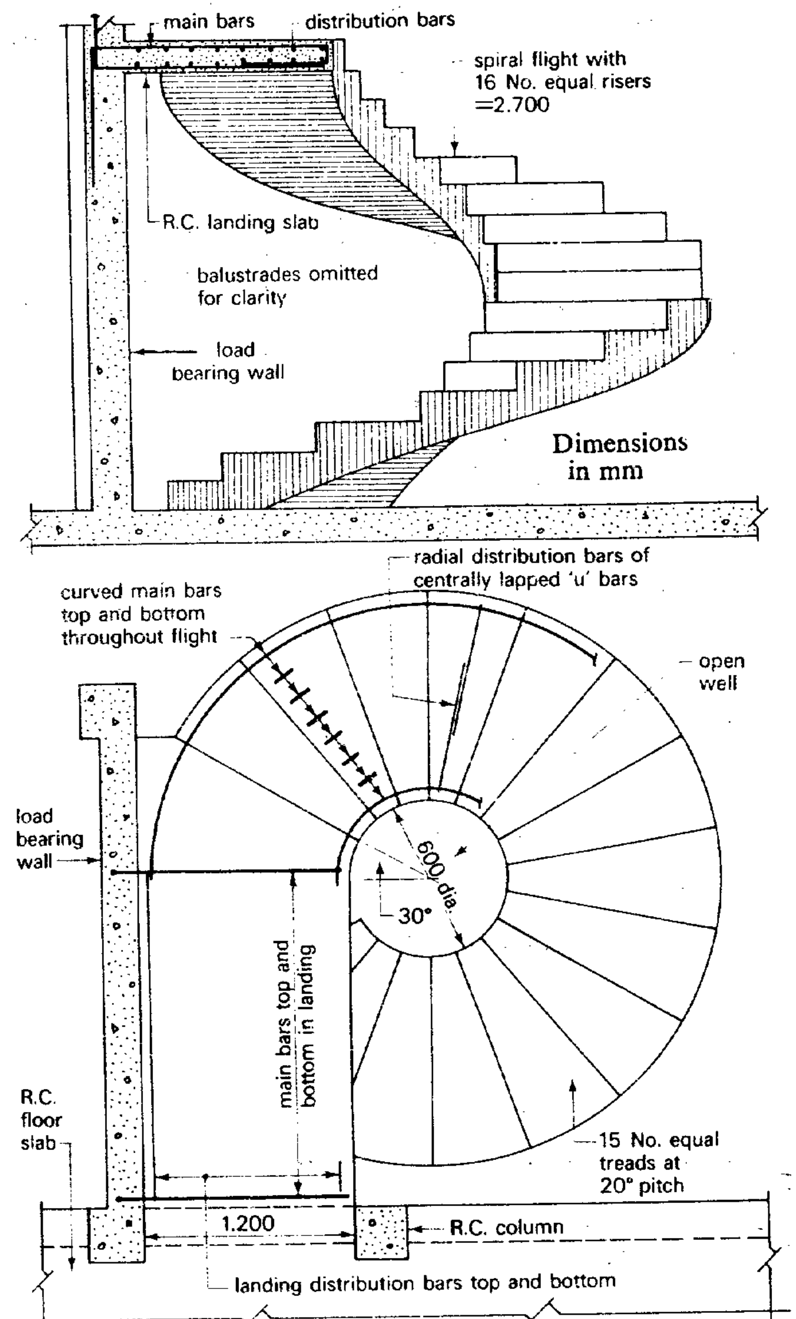


Precast Concrete Straight Flight Stairs

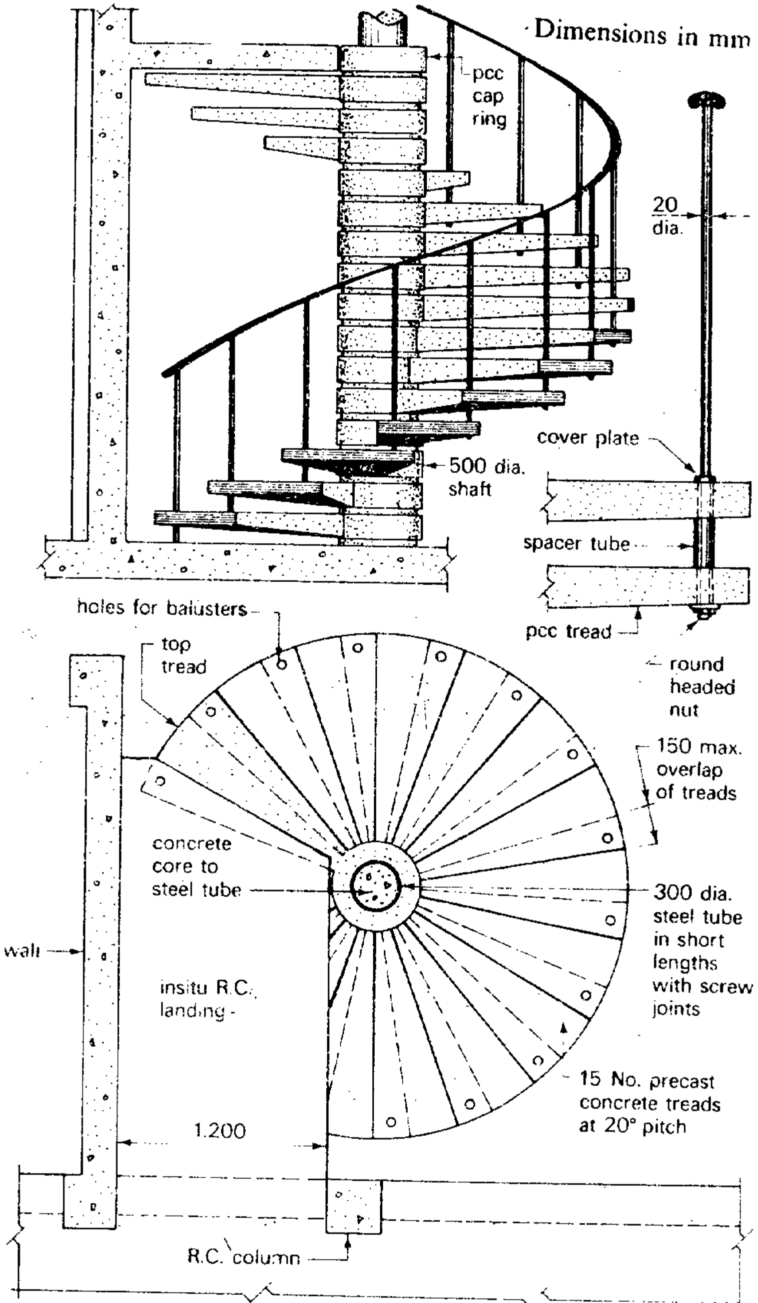


Precast Concrete Open riser Stairs

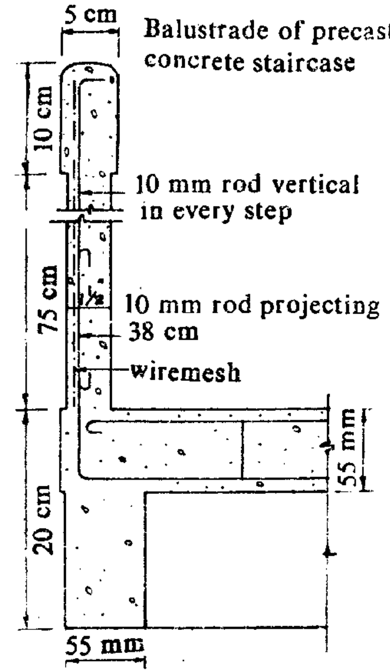




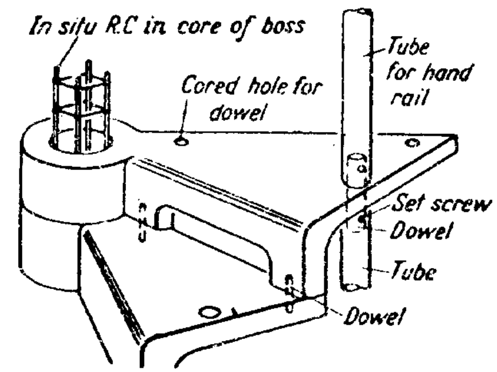
Open Spiral Concrete Staircase



Precast Concrete Spiral Staircase



DETAIL OF PARAPET



Pre-cast reinforced concrete circular staircase

Cement Concrete Hollow Blocks

Concrete blocks may be made of dense or light weight concrete, or a lean mix of cinder concrete and may be either solid or hollow. Block 10 cm or more in thickness are often cast hollow.

Hollow concrete blocks shall comply with the following specifications in general :—

(*a*) Concrete blocks may be of any size but some standard sizes should be used so that they can be bonded with brick masonry if necessary. The following sizes are generally recommended :—

Type	Nominal size in cm			Actual size in cm		
	Length	Breadth	Height	Length	Breadth	Height
A	40	30	20	39	30	19
B	40	20	20	39	20	19
C	40	10	20	39	10	19

Blocks are generally referred to by their nominal dimensions which include the block and an allowance for joints. A length to height ratio of 3 to 1 is probably the most desirable from the point of wall strength.

(*b*) The shell thickness of blocks shall be not less than 65 mm in any part (some specifications prescribe only 40 mm for type A and B and

30 mm for type C, while some others prescribe 50 mm thickness for all walls and webs.) Keep thickness according to the strength desired. Volume of concrete in any block shall be not less than half the gross volume of the block, and the total width of the cavities shall be not less than two-thirds of the overall thickness of the block at any point.

(*c*) Steel wires may be embedded in each block.

(*d*) Where walls are exposed to weather, 15 metres should be the maximum length without any expansion joint. Where walls are not exposed to weather, they may be made up to 30 metres length.

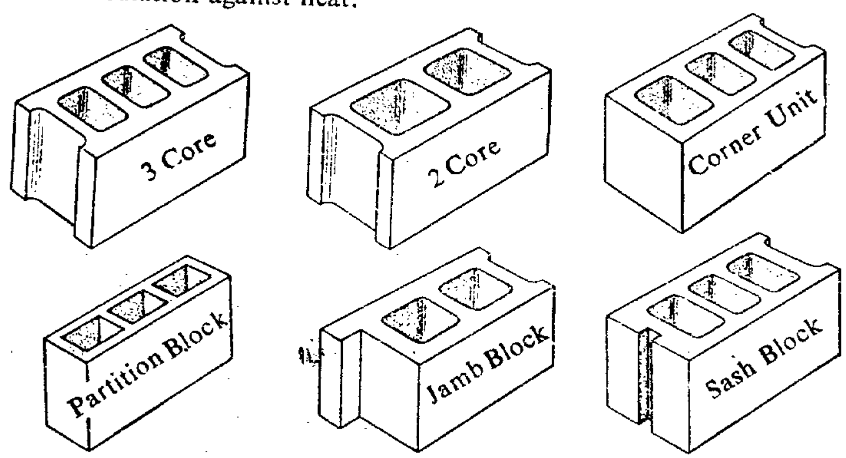
(*e*) The course immediately below each floor should be built of solid blocks.

(*f*) Blocks should be thoroughly cured and dried out before placing.

Hollow blocks are manufactured in special machines. Each unit should be cast in one operation. Concrete should be thoroughly compacted in the moulds with blunt ended steel rods, or by external form vibrators. Alternatively, vibrating tables may be used. Dense concrete is made in ordinary way with normal aggregates. Cement and coarse sand with small size aggregate are used with very low water/cement ratio. Due to high compression and very dry consistency of the mix, the blocks can be removed from the machine for curing immediately they are cast. Rapid hardening cement may be used.

These blocks are usually made having a trapezoidal outline. The side of the greater width is placed upwards so that it remains held by concrete properly.

Hollow blocks have better thermal properties than solid blocks of the same material and total thickness. Light weight concrete provides still better insulation against heat.



For joining concrete blocks rich or strong mortars are usually inadvisable as they make a wall too rigid, localizing the effects of minor move-

ments and cracking of the blocks. Hydrated lime should be mixed with cement/sand mortar. 1 cement : 1 hydrated lime (or lime putty) : 4 to 6 sand by volume, is usually recommended. 1:2:9 or 1:3:12 proportions are also used. Walls and isolated piers subject to severe conditions requiring extra strength should be laid with a mortar composed of 1 cement, 2 to 3 sand, and 1/4 part hydrated lime.

Blocks may be made of 1 cement, 1 hydrated lime, 5 aggregate—for load bearing walls; and 1 cement, 2 hydrated lime, 8 aggregate—for non-load bearing walls. 1 eminently hydraulic lime, 2 aggregate, is also suitable,

(See also page 8/48)

Notes by Chief Engineer of Western Railway regarding **Prefabricated Quarters** built at various stations :

"These houses were built with a material called gunite. It consists of cement and sand mixed in a certain proportion and shot under a pressure of 3 to 4 kg/sq. cm by means of an air compressor. To attain rapidity of manufacture and erection all parts of the houses—walls, columns and roofs—were pre-fabricated and erected by manual labour. The process required no factory. What is needed is an air compressor and two mixing machines called cement guns and hose pipes. Walls, roofs and columns are pre-fabricated on the ground and erected by manual labour.

Usually the foundations are dug up to 1.2 m below the ground for columns and a cement foundation of 1:5:10 concrete is laid, say, 45 cm × 30 cm for walls and then the columns are laid with a mortar joint of 1:4 one over the other with cement joints in between. The columns blocks are also cast 30 cm × 90 cm in sheet moulds. They have grooves for the walls slabs. These column blocks are laid one over another with a cement joint and after one meter builtup slabs are slid in with cement mortar in between.

Gunite specialists in America opine that structures made by this method outlast brick and mortar constructions. The compressive strength of the material is 700 kg/sq. cm compared to 280 kg/sq. m of controlled concrete.

By using the reinforced concrete formula the workable stress in concrete is raised to 100 kg/sq. cm. Cement concrete has a stress of only 50 kg/sq. cm. The roof slabs are erected over pre-cast beams and there is a 6 mm bar truss projecting over 65 mm thick slabs to which temperature reinforcement and rebending reinforcement are tied and slabs kept with 6 mm space in between. Later a guniter goes up and shoots about 40 mm gunite, thus making the slab 100 mm thick.

The advantages of such constructions are : (1) The buildings being light there is a saving in the cost of foundation; (2) No centering or scaffolding is needed to support roofs and floors thus saving considerably in cost; (3) Wall being 50 mm thick one gets wider rooms; (4) No weathering course is needed as the gunite itself is impermeable.

Temperature measurements indicate that a 50 mm wall house has the same temperature as a 340 mm brick wall. A double wall building with an anular space of 50 mm is three or four degrees cooler than its conventional counterpart."

Underground Storage Cellars

Sizes given for water tanks can be adopted for places subject to floods or water logging. For dry locations, wall reinforcements can be slightly reduced. For partition walls single reinforcement in the centre of the wall should be provided and of about half the quantity used for outer walls. Walls, floor and roof are generally made monolithic and of about the same thickness, and corners splayed and adequately reinforced.

Overhead Electric Transmission Line Poles; Telegraph and Telephone Poles & Lamp Standards

The forces acting on poles are:

(*i*) The self-weight of the pole and weight of the wires supported by it.

(*ii*) The wind pressure acting on the pole, cross arms and wires.

(*iii*) The unbalanced horizontal pull along the transmission line which may be caused by the snapping of the wires in one span causing a bending moment as well as a twisting moment, tending to twist the pole about its vertical axis.

Poles are usually made with top section of 115×115 mm to 130×130 mm with a taper of 2 cm per metre length. A tapering central duct of diameter not less than 32 mm at the top is left for taking the cables from the base of the fittings at the top, and also for reducing the weight of the pole. Perforations are also sometimes made to reduce the weight and cost. Poles may be square or circular in section. A hollow circular section is advantageous. The concrete used may be of 1:2:4 mix where the poles are mechanically vibrated or spun, and of 1:1.5:3 mix for hand tamped poles. At least four longitudinal bars are provided with sufficient number of stirrups. All reinforcement shall have an external cover of at least 25 mm. Suitable aperatures are provided in the poles at or below ground level for the entry of electric cables and pipes etc. where required. The ratio of the bending moment to the direct load being very high, the section is designed purely for the bending moment. For a uniformly tapering pole the critical section is not necessarily at ground level, but is at a point where the pole dimension is 1.5 times the top dimension and this section should be checked for the stresses in the concrete and steel. The shear stress in poles is very low in the section above the ground level and only nominal reinforcement is generally necessary. Shear, however, is very heavy in that portion which is buried in the ground.

Bending moment is designed to be carried by the reinforcement, the effective depth d being the distance between the reinforcement on the opposite faces.

$As = \frac{BM}{fs\ d}$ = Area of reinforcement on one side in case of square poles.

Foundations

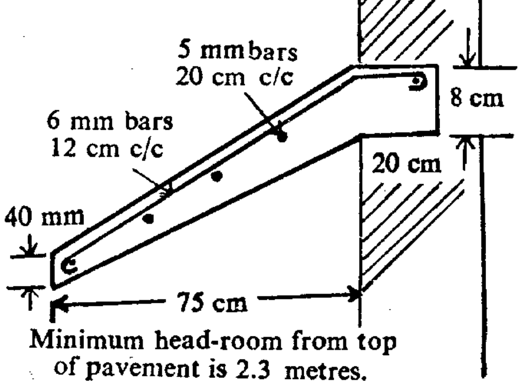
Small poles or poles which do not have heavy bending moments can be buried directly into the ground but for heavy poles foundations of mass concrete have to be provided. Depth for directly buried poles varies from 1/6 to 1/10 of total pole height depending upon the nature of the soil and is usually about 1.5 to 1.8 metres.

Design Data for RC Cantilevers, Canopies, Balconies & Sun-Shades

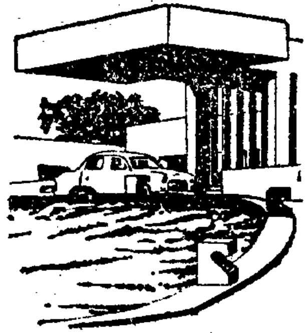
Projection (Overhang) metres	Overall Depth at Support cm	Reinforcement			
		Main Tensile at Top		Distribution	
		Dia. mm	Spacing cm	Dia. mm	Spacing cm
0.5	7	6	15	5	20
1.0	9	6	10	6	20
1.5	13	10	18	6	20
2.0	17	12	20	6	16
2.5	20	12	15	8	14
3.0	25	12	12	10	18
3.5	30	12	8	10	16

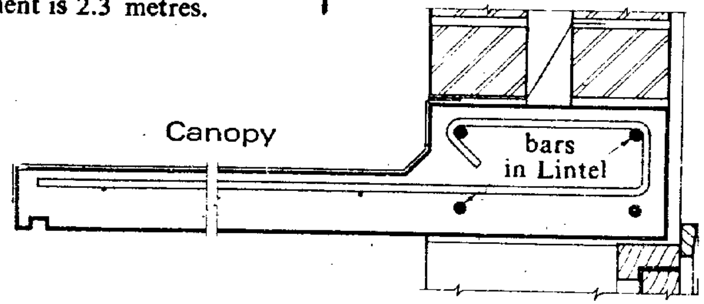
Table is based on 1:2:4 mix. Uniformly distributed live load 200 kg/sq. m (to which dead load of the cantilever is added). Reduce thickness of the projection at the free end to about 1/2 to 3/4th of the thickness at the support. Make top surface sloping 1 in 40 to drain off rain water. Provision has to be made against overturning of the cantilever for which it has to be made monolithic with the beam over the opening of the verandah and a wall built over this beam to counter-balance the over-turning moment of the projection for which total weight over the beam (including its own weight)×1/2 width of the beam should not be less than 1.5×B.M. of the cantilever projection at the support end. Tensile reinforcement should be provided at the top and shall be continued (embedded) into the beam (or roof slab if there is one) for at least 1.2 m and which may be for a distance of not less than 60 diameters from the edge of the support for small projections. Every alternate bar to be curtailed (tensile as well as distribution) at half the projection from the fixed end for projections above 2.5 m. Some light reinforcement should be provided on the compression side of the cantilever as a safety measure. Where sufficient load is not practicable over the verandah beam to counter-balance the projection moment, anchoring rods should be provided on both ends of the projection through the supporting wall or pillars.

RC Sunshade

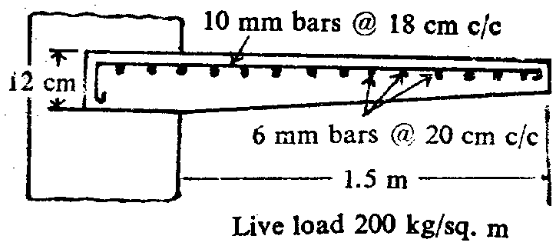


Canopy

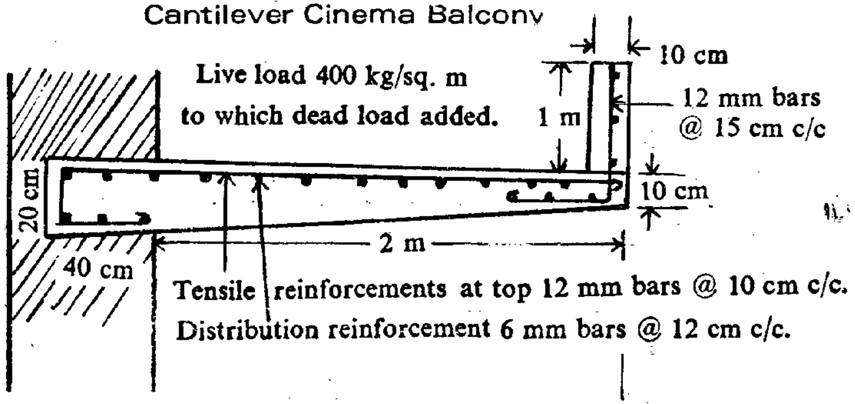




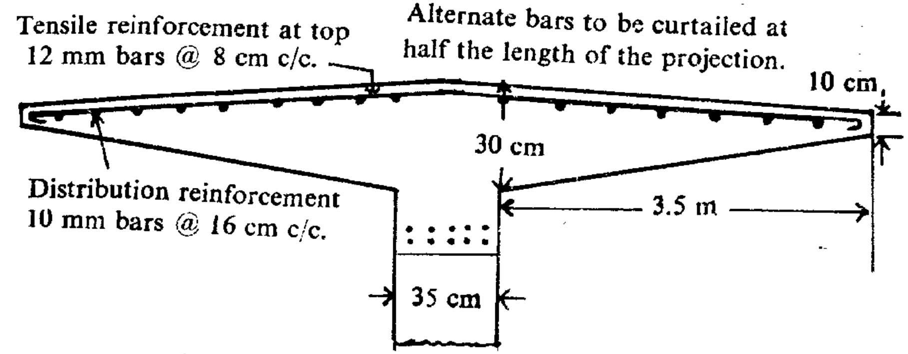
Cantilever



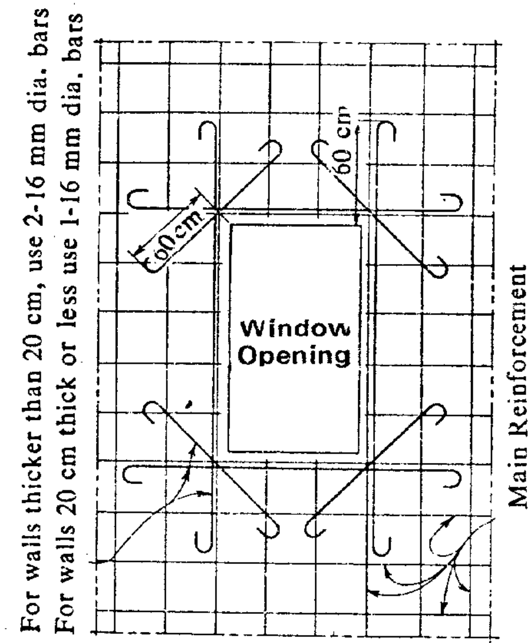
Cantilever Cinema Balcony



Cycle Shelter



Central beam supported on columns 6 m apart made monolithic with the projections. Reinforcement to be 12—25 mm dia. bars in two layers. Central beam will need shear reinforcements



Reinforcement Around Wall Openings

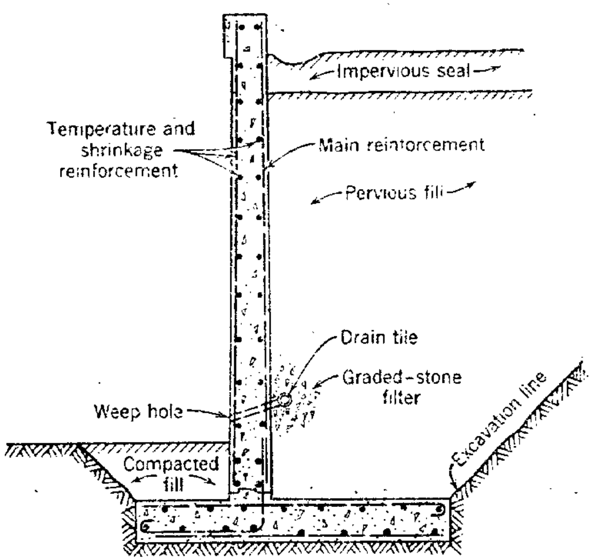
Reinforced Concrete Walls

Load bearing walls are seldom built of reinforced concrete. In case reinforced concrete walls are required for sustaining loads, they should be designed as tied columns, the design being made for a unit width of wall. Wall thickness is made not less than 1/25 of the unsupported height or length, whichever is shorter, and which may be 1/30 of the distance between the supporting or enclosing members for panel or filler walls.

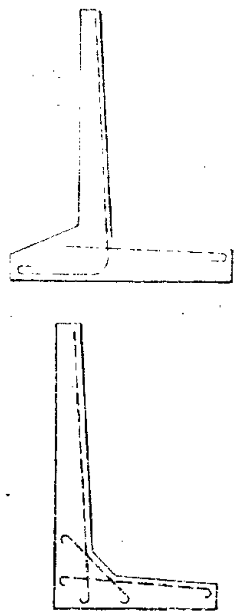
Minimum thickness prescribed for load bearing walls is 10 cm and for filler walls 7.5 cm. Filler walls are often built in brickwork or hollow concrete blocks of ordinary or light-weight concrete.

Exterior basement walls, foundation walls, and fire walls should be not less than 20 cm thick whether reinforced or not.

Reinforcement. For load bearing walls, both vertical and transverse reinforcement provided is not less than 0.2 per cent and which may be 0.15 per cent with deformed bars or welded wire fabric. For walls not designed to carry any load, the transverse reinforcement may be reduced to only 0.1 per cent. The diameter of a vertical reinforcement rod should be not less than 10 mm, and the distance between two vertical reinforcements and the distance between two lateral reinforcements should not exceed 30 cm (45 cm max :). In addition to the reinforcement prescribed above there should be not less than two 16 mm dia. bars around all window or door openings. Such bars should extend at least 60 cm beyond the corners of the openings. Walls more than 25 cm in thickness should have the reinforcement for each direction placed in two layers parallel with the faces of the wall.



Tee Wall : reinforcement and drainage details



Retaining Walls

Walls should be anchored to the floors, columns, of intersecting walls with reinforcement of at least 10 mm bars 30 cm centres for each layer of wall reinforcement. Whenever possible the reinforcement should be made up into mats in advance. The complete mat is then placed in position and kept at the correct distance by spacers which are fixed between the both side forms.

Formwork. When a wall is cast in 60 to 90 cm lifts and compacted by hand, the most satisfactory type to use are board spacers, which can be raised as concreting proceeds The formwork should be fixed with wire ties or bolts to resist the outward pressure of the concrete : these pass through the wall and, acting together with the spacers, hold the formwork in the correct position. Bolts are extracted while the concrete is still green, the holes being made good after the formwork is removed.

The concrete in each lift should be placed in 15 cm layers and each layer thoroughly punned before the next is shovelled in. Each lift of 60 or 90 cm should be given four or five hours to settle before the next lift is placed. If immersion vibrator is used for compacting, the vibrator head should be placed at the bottom of the wall before the concrete is placed, and gradually drawn up as concreting proceeds. Joints are best made at such points at sill or window head level.

Footings. For common one and two-storey houses erected on soils of average load-carrying capacity, the concrete footing is generally made twice as wide as the thickness of the wall it supports. The depth of the footing is usually one-half its width or equal to the thickness of the wall.

Fence Posts

Ordinary line posts may be fixed 3 metres apart. Every 10th, last but one post, and corner post shall be strutted on both sides, and end post on one side only. Corner and end posts are similar to straining posts.

For the length of a post to be underground for fixing, a good rule in normal soils for line posts is "one-third of their length below ground", and for straining and corner posts, not less than three-seventh below ground"

For normal cases the total length of a line post may be 1.8 metres and that of struts 2.0 metres (min :). Posts are cast in cement concrete 1:2:4 and reinforced with 4 nos. 6 mm dia. mild steel bars. The size of such line posts may be 10×10 cm at top with a taper of 5 cm per metre length. Struts may be 10×10 cm straight. Where it is proposed to use thicker posts, the size of the reinforcing bars may be increased to 12 mm for top size of 20×20 cm. Reinforcing rods must be protected by adequate thickness (25 mm) of dense concrete, otherwise moisture may penetrate and rust the reinforcement.

Base blocks. Pits 45×45×75 cm deep for line posts and 70×45×75 cm deep for struts shall be excavated in the ground. Posts and struts are placed in the pits and filled with 1:5:10 concrete so that posts are embedded in the blocks of size 45×45×60 cm and struts in blocks of size

70×45×50 cm. Posts blocks will be thus 15 cm below ground and strut blocks 25 cm below ground. Posts and struts are surrounded by 15 cm of concrete on all sides.

Posts project 1.2 metres above ground.

(See also pages 21/13,14)

CPWD specification

1.2-metre high fencing with 1.8 metre RC posts placed every 3 metre apart, embedded in cement concrete blocks, corner, end and every 10th post to be strutted provided with 9 horizontal lines and two diagonals of barbed wire 9.38 kg/100 m (min :) between the two posts fitted and fixed with GI staples on wooden plugs tied to 6 mm bar nib with binding wire.

6. PHYSICAL PROPERTIES OF CONCRETE

Workability is that property of a concrete which determines the case with which it can be placed in position and compacted. Workability is normally measured by the 'slump test' as a guide. In order to obtain concrete of maximum strength, good compaction is essential and this can only be achieved if the concrete has adequate degree of workability in relation to the method of compaction to be used. The workability of a concrete should be just sufficient to enable the concrete to be compacted fully by whatever method is employed. Concrete that is to be placed in narrow forms congested with reinforcement will require a much higher degree of workability (*i.e.* fluidity) than that for unreinforced mass concrete. Concrete which is to be compacted by mechanical vibration may be much drier than that which is to be tamped by hand. This concrete needs about 20 per cent less water and about 15 per cent less cement. As such, a drier concrete which has been compacted by vibrations gives more strength and density for the same quantity of materials. The principal factors which effect the workability of concrete are :—

Consistency. For RC works, concrete which will flow sluggishly into the forms and around the reinforcement without any segregation of coarse aggregate from the mortar, has to be used. The degree of consistency, which shall depend on the nature of the work and whether the concrete is to be vibrated or hand tamped, shall be determined by slump tests.

The quantities of materials may be regulated by carrying out regular slump tests. More water will be needed for dry conditions and less for wet and cold conditions. Structures in contact with water should be made of drier cement.

Rapid hardening cement needs about 4.5 litres more water per 50 kg of cement than ordinary cement.

Water/Cement Ratio or Water Content of a Concrete Mix. Water/cement ratio is the ratio of the water in a mix (excluding water already absorbed by the aggregate) to the weight of cement therein, and this is the

most important factor governing the strength of a concrete. The strength of a concrete depends mainly upon the amount of cement and the amount of water in it. The correct quantity of water required for a particular mix depends upon various factors such as : Mix proportions, type and grading of aggregate (the quantity of water varies with the size and shape of both the fine and coarse aggregate), method of compaction applied (whether with vibrations or hand tamping), and the weather conditions.

Water Content and Workability. The workability of a concrete increases as the water content of the mix is increased—water lubricates the mixture. But increase in water content would cause a decrease in strength. Excess of water weakens a concrete, produces shrinkage cracks (shrinkage increases with increase in water content) and decreases density. (See under "Contraction and Expansion Joints"). Water occupies space in concrete and as it evaporates it leaves voids and cracks. The volume of water voids may be as much as 10 per cent of the total volume of concrete. An excess of 10 per cent of water may reduce the strength by about 15 per cent and an excess of 30 per cent of water may reduce the strength by half. Generally speaking, lower the water content the stronger the concrete but the quantity of water must be sufficient to produce a workable mix required for the particular method of compaction to be adopted.

Concrete made with low water/cement ratio is unworkable. If stiff or dry concrete is used honey-combing will result decreasing density and strength. An unworkable concrete results in incomplete compaction giving rise to air voids. Presence of 5 per cent air voids will cause a 30 per cent strength loss and 10 per cent air voids may cause as much as 50 per cent strength loss. Therefore, there is an optimum value of the water/cement ratio for every mix. The quantity of water has to be restricted within certain minimum limits. Concrete should be just plastic enough to be worked around the reinforcement rods.

Sometimes strength has to be sacrificed by adding more water to obtain a higher degree of workability where concrete has to be placed in narrow and thin sections. The best mix is the one which gives the maximum workability with the minimum amount of water. An increase in water content must be accompanied by a proportionate increase of cement if strength is to be maintained.

Grading of Aggregate. Other things being equal the workability of concrete is greater with aggregates of larger maximum sizes. For dry mixes workability is generally greater with rather coarse aggregate gradings but for wet mixes better results are often obtained with finer gradings.

Shape of aggregate particles. A smooth and rounded aggregate will produce a more workable concrete than sharp angular aggregate (crushed rock or crushed gravel). A flaky aggregate produces the harshest or most unworkable concrete. (Aggregates producing more workable concrete need less water and hence give higher strengths.)

Cement content. The higher the cement the greater the workability and the less the effect of grading. As such, much greater latitude in grading can be permitted with a rich mix (high cement content) than with a lean mix (low cement content), but that is uneconomical. A slight increase in the quantity of cement increases correspondingly the concrete strength provided water/cement ratio is kept constant.

Segregation is the separating of the coarse aggregate from the rest of the mix or the separating of the cement-water paste from the aggregate. Segregation generally indicates poor aggregate grading or mix design. Segregation may occur in mixes which are too wet or too dry, and most frequently in under-sanded mixes.

Segregation can generally be reduced by altering the water or sand content or by using a finer sand. Even with a mix of satisfactory design, segregation may be caused by mishandling during transport, faulty placing or over-compaction. Segregation leads to lack of uniformity causing honey-combing which reduces the strength and durability of the structure.

If segregation occurs the larger particles of aggregate tend to move to the bottom and this causes undesirable variation of strength through the thickness of the slab.

Bleeding is the appearance of a watery scum (also called laitance) on the surface of a concrete after compaction. It is an indication that there is too much water or deficiency of fine material in the mix, or that too much tamping, floating or trowelling has been done. The result is a porous, dusty and weak surface. This scum should be removed. Bleeding makes weak joints between successive lifts in structural work. Bleeding can be reduced by using less water, a finer sand, or by adding a finely ground inert material (stone dust).

The aggregate commonly used are seldom found in a perfectly dry state in the field. Moreover, aggregates have to be washed very often for removing impurities which further add to the moisture content. The moisture content varies considerably from time to time with the changing weather conditions, and this is especially so in the case of sand. The aggregate when dry will absorb water from the concrete and when wet at the surface the mixture will have excess of water. Therefore, while computing the quantity of water due consideration must be given to the surface conditions of the aggregate that would exist at the time of preparing the mix.

Small size of aggregate need more water than big size and angular aggregate need more than rounded aggregate. In other words, a concrete containing a finely graded aggregate will require more water for a given workability than one containing an aggregate with a coarser grading. Consequently, the more finely graded aggregate, or that containing a larger proportion of fine aggregate (and similarly a concrete with angular aggregate) will produce a weaker concrete.

Hydration of Cement. When water is added to cement, the cement hydrates, calcium hydroxide or hydrated lime is liberated. During the chemical reaction which takes place while cement is setting and hardening

an increase in temperature occurs and a considerable quantity of heat is evolved. Shrinkage occurs on subsequent cooling resulting in cracks. Hydration of cement is incomplete without an adequate quantity of water. Less water impedes complete setting of cement and decreases strength. The amount of water required to hydrate cement is about 25 per cent of the weight of the cement. The amount of mixing water is rarely less than twice this quantity.

If water/cement ratio is less than 0.4 to 0.5 complete hydration of cement will not occur. Roughly water/cement ratio is 0.60 for a 1:2:4 mix, 0.5 for 1:1.5:3 mix, and 0.45 for 1:1:2 mix.

Quantity of Water and Aggregate per 50 kg (one bag)—about 35 litres of cement for Hand Compaction.

Mix	Water litres	Fine Aggregate litres	Coarse Aggregate litres	
1:1:2	21 to 27	35 0	70.0	For vibrated concrete, the quantity of water can be reduced by about 20 per cent.
1:1.5:3	26 to 30	52.5	105.0	
1:2:4	29 to 32	70.0	140 0	
1:3:6	34 to 36	105.0	210.0	
1:4:8	45 to 47	140.0	280.0	

If sand is wet, increase its quantity by 25 per cent and reduce quantity of water by 20 per cent.

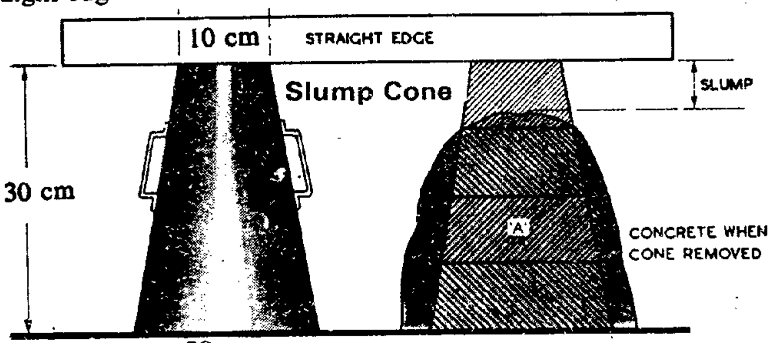
Litres of water per 50 kg of cement	Water/Cement Ratio by Weight	Cube Crushing Strength at 28 days in kg/sq. cm.	
17.5	0.35	530	Mix too dry for hand compaction.
20.0	0.40	470	
22.5	0.45	420	
25.0	0.50	370	
27.5	0 55	320	Mix workable for hand compaction.
30.0	0.60	280	
32.5	0.65	250	
35.0	0.70	220	
37.5	0 75	200	
40.0	0.80	180	

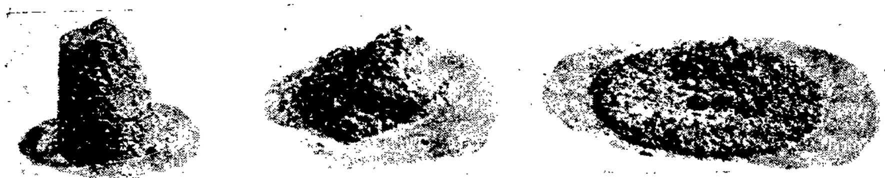
The Slump Test

Although the slump test is not entirely satisfactory since it gives widely varying results and also does not give a true measure of workability but it is of value in the field as a control test and is useful in comparing the consistence of successive batches of concrete made with the same ingredients, and is one of the simplest tests to carry out. Provided no change is made in the aggregate or its grading, slump tests will indicate whether correct water and cement contents are being maintained. For a given slump and aggregate grading, the water required for unit volume of concrete is constant irrespective of the change of cement content. The amount of slump depends not only on the amount of water in the mix but also on the nature of the aggregate ; rounded stones give a greater slump than angular stones for the same mixture.

The slump test should not be used to compare mixes of different proportions or of different types of aggregates. This test is not applicable to lean dry mixes where the water/cement ratio is low as the slump recorded is very small. All aggregates of size 50 mm and above should be removed from the sample concrete before the test.

The apparatus for determining the slump (slump cone) is a steel mould in the form of a truncated cone. Its top diameter is 10 cm, the bottom diameter 20 cm, and the height 30 cm, open at both ends and fitted with handles and foot pieces on sides. The cone is placed on a smooth non-absorbent surface and freshly mixed concrete is placed in the mould in four successive layers, each layer being rodded 25 times with a bullet-pointed rod 16 mm in diameter and 60 cm long. When filled to top (after ramming) and top struck level, the mould is immediately withdrawn and the slump or subsidence of the concrete measured from a straight-edge held across the top of the mould.





Stiff Medium Wet

Types of Slump

"Slump" is the vertical settlement of the concrete after the mould has been withdrawn, *i.e.*, the difference between the height of the mould and the highest point of the subsided concrete.

Recommended Values for Slump in Millimetres

Types of work	With vibrations	Without vibrations
Mass concrete, large sections, roads	10 to 25	50 to 75
Foundations, footings, sub-structures, walls and other heavy sections.	26 to 50	40 to 115
Thin sections such as slabs, beams, columns, with congested reinforcement.	40 to 80	100 to 175

A concrete with 0 to 25 mm slump has very low degree of workability, and with 100 to 175 mm slump a high degree of workability which is not normally suitable for vibrations.

Strength. The strength of a hardened concrete largely depends upon : (*i*) the water/cement ratio, (*ii*) the quality and characteristics of cement, (*iii*) the degree of compaction obtained in the concrete, (*iv*) curing, and (*v*) the age of the concrete. The strength increases as the concrete becomes older.

Generally speaking, strength is largely independent of the type or grading of the aggregate and the mix proportions, these factors influence the water content required to produce a given degree of workability and therefore affect the strength indirectly. A rounded aggregate requires a lower water/cement ratio than does an angular one to obtain the same workability, therefore, a rounded aggregate gives a higher strength. Similarly, a coarser aggregate grading will permit a lower water/cement ratio than a finer grading for a given workability and will thus give a higher strength. Greater strengths are also possible with richer mixes or more thorough compaction.

In general, the type of aggregate has little effect on the crushing strength of a concrete but it has an appreciable effect on the flexural strength : the more angular aggregate give higher flexural strength.

Concretes of identical design and materials produced under identical conditions differ considerably in strength.

Shrinkage. Concrete shrinks during setting and drying due to hydration of cement and produces shrinkage cracks. The drying shrinkage increases with an increase in cement content or an increase in water content. Shrinkage is greater with richer mixes (more of cement) and also with aluminious cements. Other things being equal, shrinkage of concrete is almost directly proportional to the amount of water in the mix. The type of aggregate used does not generally affect the shrinkage seriously though it has an indirect effect due to the difference of water/cement ratio depending on the type of the aggregate ; with large size of aggregate shrinkage is low. Where shrinkage may give rise to high tensile stresses such as in road slabs, lean dry mixes are desirable. (See also under "Joints") Rich mixtures are uneconomical and are used only for impermeable constructions to ensure water-tightness.

Expansion. The thermal expansion of concrete depends largely on the type of the aggregate and the amount of the cement used : Concretes made with silicious aggregates expand more than those made with calcareous aggregates such as limestone. Rich mixes expand more than lean ones as the co-efficient of expansion of the cement paste is greater than that of the aggregate. (See also under "Contraction and Expansion of Joints".)

Load Tests on Built Structures. If the strength of a structure is doubtful, a test load of 1.5 times the super-imposed design load should be applied not before 28 days after concreting. For floor and roof slabs, the tests need not be made until 56 days of effective hardening of the concrete. During the tests, struts strong enough to take the whole load should be placed under the members but leaving a small gap below the member. The test loads on floors and roofs should be maintained for 24 hours. There should be 75 per cent recovery of deflection on removal of the load.

7. MATERIALS FOR CONCRETE MIXES

Quality of Water for Concrete

Water for concrete should be clean and free from oils, acids, alkalies, vegetable or other organic impurities. In general, water that is fit to drink is suitable for concrete (but the reverse is not always true). Excess of acidity or alkalinity can be tested by litmus paper ; rapid change of the litmus paper indicates dangerous amount of acid or alkali present. "Soft" waters may produce a weaker concrete than hard waters. Moorland or marsh waters are also harmful. Waters containing decayed vegetable matter should be particularly avoided as they may interfere with the setting of the cement. Use of sea water should be discouraged in reinforced works and it should not also be encouraged in plain concrete works, but may be used for mass concrete. Sea water will retard the setting and hardening and probably cause efflorescence but will not affect the ultimate strength of the concrete unless salt is present in excessive quantities. Salt in water corrodes the reinforcement. Brackish water although not always potable, is not usually harmful.

A practical field test for the suitability of a particular water, beyond a visual inspection for cleanlines, is to make two identical pats of size 75 mm dia. and 12 mm thick of neat cement paste, one with the water under test and the other with water of known suitablity. Place the pats on a clean non-absorbent surface and leave for 48 hours, and setting and hardening times observed for both the pats. Should the pat made with the water under test not be up to the standard of the other, then water should be deemed unsuitable.

Water in concrete has twofold purpose—to hydrate the cement and to lubricate the mix so as to aid compaction.

CEMENTS

Manufacture of Cement. The principal chemical constituent of cement are 60 to 67 per cent lime, 17 to 25 per cent silica and 3 to 8 per cent alumina, which are intimately mixed together with water to form into a slurry, which is subsequently heated, dried, calcined and ground to a very fine powder. A small proportion of gypsum is added before grinding in order to control rate of setting. Portland cement may be made quick or slow setting. If there are no special reasons which make quick setting desirable, normal setting cement should be employed as quick-setting cements require exceptional care and skill in handling.

Types of Cements. There are mainly five kinds of cements. Cements are classified by their properties and chemical composition.

Ordinary Portland Cement is the most commonly used cement for general engineering works. Other cements with different properties are used for particular purposes.

Rapid-hardening Cement is, in fact, a high early strength cement. This cement has the same composition as common cement but is ground more finally and is used where high early strength is required. It sets and hardens in a much shorter time than the common cement and develops higher strength in the early stages, but the ultimate strength is about the same as of the normal setting cement. The advantages of this cement over the common cement are that formwork can be removed earlier, and the structure can be loaded earlier. It has in 4 days the same compressive strength as common cement in 28 days. Rapid hardening cement is manufactured in India in limited quanties only and its use is also very limited. It is comparatively costlier than common cement. The setting time is about the same as of the ordinary cement. This cement is useful for repair work.

Rapid-hardening cement is also used for road work where it is imperative to open the road to traffic with the minimum of delay.

Quick-setting Cement. This type of cement sets initially after about 5 minutes and sets finally in about 30 minutes. Its uses are generally restricted to works in running water. The quick setting action of this cement allows very little time for mixing, placing and, compacting of the concrete, and its use therefore demands most careful site organization. Whilst quick setting, it hardens at approximately the same rate as ordinary cement. Concretes made with this cement should be kept moist with water as soon as after they are set.

"Quick-setting" cement should not be confused with "rapid hardening" cement ; quick-setting cement does not harden rapidly. The term "rapid-hardening" is synonymous with "high early strength". (Difference has been explained further.)

Blast-furnace Slag Cement is a mixture of Portland cement, clinker and granulated blast furnace slag. This cement is used for massive structures such as dams, retainning walls, foundations, and bridge abutments. Its resistance to chemical attack is slightly superior.

High Alumina Cement is used where it is required to impose loads on the concrete structure even earlier than that possible with rapid-hardening cement. It hardens much more rapidly than the common Portland cement and also develops strength very early—up to 75 per cent of its ultimate strength being attained during the first 24 hours after mixing. This cement gives out great heat during setting and cannot be used for massive structures. The speeding up of setting time can be brought about by mixing with it 1 to 2 per cent by weight of hydrated lime. No water-proofers are necessary with this cement. It is highly resistant to sea water but is not suitable for very hot climates. Aluminous cement must not be mixed with other cements or lime mortars, or placed against Portland cement concrete less than 7 days old. This cement is useful for emergency repair works.

Low Heat Cement liberates much less heat than the common cement upon hydration and setting. It is used for structures where it is necessary to restrict heat generation (due to hydration of cement) during concreting, to avoid cracks, in large masses of concrete such as dams, bridge abutments and retaining walls. The rate of development of strength is somewhat lower than common cement, but the ultimate strength is about the same.

Properties of Cements. Properties of interest to the engineer are: (*i*) Rate of setting ; (*ii*) Rate of hardening ; (*iii*) Heat evolution and (*iv*) Resistance to chemical action.

The terms "setting" and "hardening" should not be confused. Setting is the phenomenon which changes a cement paste mortar or a fluid concrete to a solid but in a weak state, while hardening is the process by which the weak set mortar or concrete attains strength. The term "initial set" relates to the start and "final set" to the completion of the setting. Hardening begins after the cement has set and proceeds rapidly during the first few days and continues to increase at a diminishing rate indefinitely. There is no necessary relationship between the time of setting and that of hardening or attaining the maximum strength ; a slow-setting cement may harden more rapidly than a quick-setting one and *vice versa*. The hardening of cement is actually a continuation of the chemical action which begins with setting. The setting time is determined by a Vicat's Needle. Temperature has a very great effect on the setting time of cement. Cements should have the following setting times :—

Type of cement	Initial set	Final set
Normal setting	Not less than 30 mins.	Not more than 10 hrs.
Rapid hardening	ditto.	ditto.
Quick-setting	Not less than 5 mins.	Not more-than 30 mins.
Low heat	Not less than 1 hour	Not more than 10 hrs.
High alumina	Not less than 2 hrs., nor more than 6 hrs.	Not more than 2 hrs. after the initial set.

(Times taken from that of adding water to the cement)

Approximate comparative compressive strengths at 28 days of 1:2:4 concretes prepared with different cements with water/cement ratio of 0.60 :—

Mix with common cement——1.00
" " aluminous " ——2.00
" " rapid-hardening" ——1.20
" " low heat " ——0.82

White and Coloured Cements have the same properties as the common grey cement, but are 4 to 6 times costlier.

Hydration of Cement and Evolution of Heat. When water is added to cement, the cement hydrates and during the chemical reactions which take place while the cement is setting an increase in temperature occurs and a considerable quantity of heat is generated. Hydration of cement is incomplete without an adequate quantity of water. Heat and humidity accelerate hydration. The amount of heat and the rate at which it is generated depends mainly on the type (chemical composition) of the cement and affects the rate of hardening. The greater the heat generated the more rapid the rate of hardening. Shrinkage occurs on subsequent cooling of the mortar or concrete resulting in cracks. The more rapid the rate of hardening the more susceptible is a concrete to shrinkage cracks.

Testing of Cement. Pats or briquettes made from ordinary Portland cement, when broken, exhibit a bluish grey colour in the fracture. Cement is tested for : (*i*) Fineness ; (*ii*) Chemical composition ; (*iii*) Tensile and Compressive strengths (cement and sand) : (*iv*) Setting time and (*v*) Soundness. Accurate testing of cement and concrete requires considerable practice and skill ; tests are standardized and described in various Codes. Cement to be used in important works should be tested in a laboratory. For less important works the following field tests may be done :—

Test for Fineness. The fineness of cement is a measure of its cementing value. A finer cement produces a stronger mortar, and it can be mixed with a large proportion of sand than a coarser one and yet attain the same strength. The residue of ordinary Portland cement left on a BS test sieve No. 170 or IS test sieve No. 9 (90 micron) should not exceed 10 per cent and with a rapid hardening cement the residue should not exceed 5 per cent.

Tests for Tensile and Compressive Strengths. It is usual to substitute a tensile test instead of a crushing test as tensile strength is roughly proportional to crushing strength and is easier to determine. Briquettes are made with 1:3 cement sand mortar in the prescribed manner and the average tensile breaking strength of six briquettes is taken. The tensile test alone shall not form the basis of acceptance or rejection of cement. No tensile strength test shall be required in the case of low heat cement. The test briquettes should give the following results :

Tensile Strength of 1:3 Cement-Sand Mortar

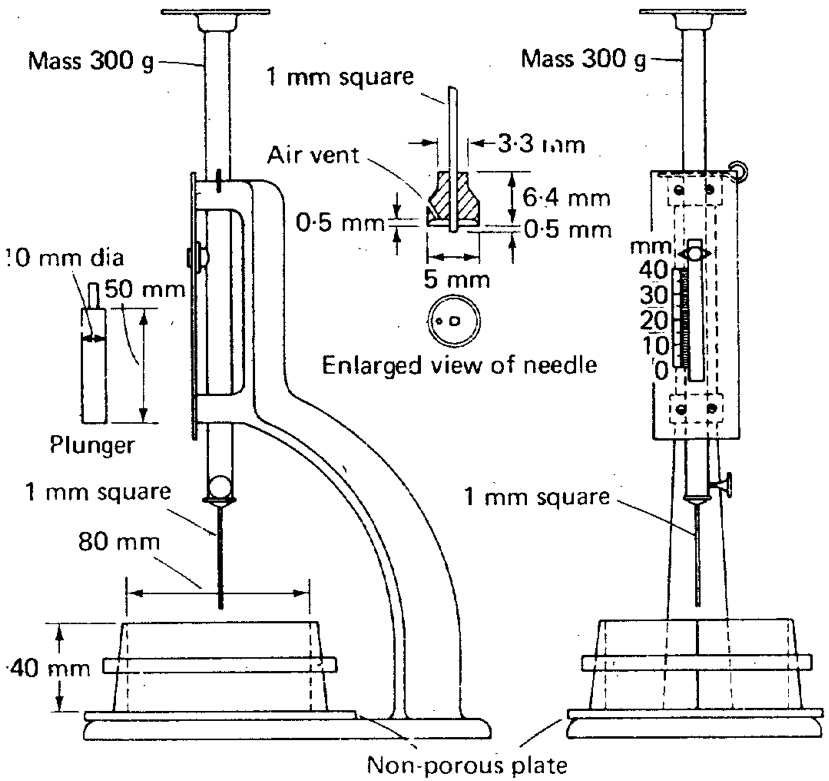
Common cement—Not less than 21 kg/sq. cm after 3 days and, greater than 26 kg/sq. cm at 7 days.

Rapid-hardening cement—Not less than 21 kg/sq. cm after 1 day and, not less than 31.5 kg/sq. cm after 3 days.

Compressive Strength of 1:3 Cement-Sand Mortar

Common cement—Not less than 115 kg/sq. cm after 3 days and not less than 175 kg/sq. cm after 7 days.

Rapid-hardening cement—Not less than 115 kg/sq. cm after 1 day and not less than 245 kg/sq. cm after 3 days.



The Vicat apparatus for determining the setting time of cement.

Test for Setting Time. Make a stiff paste of neat cement and water and form it into a pat about 75 mm diameter and 12 to 25 mm thick. The pat should commence to set in about 30 to 60 minutes. In 18 to 24 hours the pat should have hardened sufficiently so that it does not scratch the surface with a thumb-nail. It should be difficult to break the pat with the fingers after 48 hours and, it should be set fully hard in 7-8 days.

The commencement of setting of the cement can be roughly estimated by pressing the uncut end of a lead pencil into the mass ; it will be found that the resistance to piercing increase rather suddenly when setting begins. The time should be counted from when water is added to the cement. The effect of higher temperature is greatly to accelerate the setting time.

Cement keeps on hardening for at least one year and the strength of concrete at 28 days is considered only to be 60 per cent of the strength attained at the end of one year. Hardening of cement is a continuation of the chemical action which begins with setting. (This has been further explained under "Curing".)

Cement which has deteriorated in respect of quick setting can be made use of with lime in the proportions of 1:2 in place of pure lime and can be used for works where lime mortar will do.

Test for Soundness. Boil the set pat (as made for the setting-time test) in water for about 5 hours. The pat should remain sound and hard and should not swell, crack or disintegrate, but may show only hair cracks. The soundness test is in some respects the most important of all for if a sample passes all other tests satisfactorily yet fails in the soundness test; it is no good as a constructional material.

Deterioration of Cement with Storage. Cement has a great avidity for water and will readily absorb moisture from the atmosphere or from damp material in contact with it. Cement exposed to the atmosphere gets hydrated and loses strength. When cement is stored in sacks (made of jute) absorption takes place from the air and the strength of the cement is considerably reduced. The absorption by cement of 1 or 2 per cent of water has no appreciable effect, but further amounts of absorption retard the hardening of cement and reduce its strength. If the absorption of water exceeds 5 per cent the cement is for all ordinary purposes ruined.

Cement stored in bulk or in air-tight containers does not deteriorate. It can be stored in covered barrels or bins. Cement thus stored up to about 2 meters or more in depth can lie for longer than a year with no more damage than the formation of a crust on the surface about 5 cm thick, which is removed before cement is taken for use. Special care is necessary during the rainy season. Bags should not be opened until cement is to be emptied into the mixer.

The following figures show the average reduction of strength in a 1:2:4 mix. as a result of storage : —

Cement fresh				strength		100
Cement after 3 months storage, strength reduced by						20%
,,	6	,,	,,	,,	,,	30%
,,	12	,,	,,	,,	,,	40%
,,	24	,,	,,	,,	,,	50%

Ordinary Portland cement which has been stored for over six months, and rapid hardening cement which has been stored for over two months from the time of leaving the factory should always be tested before use. If deterioration is suspected the test for "setting time" described before should be applied.

Test for Freshness of Cement. Indications of a damaged cement are given by the presence of large lumps of set cement, and when this happens the lumps should be screened out unless they are soft enough to be powdered when pressed in the fingers. It needs to be considered if this cement is to be rejected.

When cement is rubbed between fingers and thumb it should feel like a smooth powder such as flour. Grittiness may indicate deterioration. Such a cement should be tested for setting time. If caked material is screened out and the balance used with 10 per cent extra cement, the concrete will be good for common small works, though it will be slow setting.

Storage of Cement. Cement can be safely stored in sacks for a few months if kept in dry and air-right room. If prolonged storage of cement is unvoidable, it is better to empty the bags and stock the cement in as deep a heap as possible in a damp-proof enclosed space. Paper sacks are better than jute sacks as regards deterioration by moisture. Cement stored for more than six months should be tested for soundness before use on all important works and which period may be three months when stored in jute bags. Concrete made with storage-deteriorated cement takes longer to harden.

Cement in bags should be stored in a dry room on a raised wooden platform 15 to 23 cm above the floor level and 30 cm away from walls. Bags to be stacked in not more than 10 layers high (max: 4.5 m) to prevent bursting of the bags in bottom layers. The bags should be placed close together to reduce circulation of air and all openings in the room should also be well closed. If the piles are to be more than seven or eight bags high, the bags should be placed in headers and stretchers, *i.e.*, alternatively lengthwise and crosswise.

Weight of Cement. Common grey (Portland) cement weighs 1300 to 1400 kg per cu. metre when loosely packed and 1700 kg when well compacted, and its weight varies between these two limits depending upon the degree of compaction. As such the method of measurement of cement by volume is inaccurate. Cement should be measured by weight. One 50-kg bag is taken to contain 34.72 or say 35 litres and to weigh

1440 kg/cu. metre (nett weight of cement). A sack of cement is 112 lbs. in UK and 94 lbs. in U.S.A.

Rapid-hardening cement is taken to weigh 1200 kg/cu. metre.

As cement manufactured at different works are not of a uniform quality and strength, use cement from one factory only for one job where possible. The large variation in cement strength causes a corresponding variation in concrete strength which ranges as high as 100 per cent between the max: and the min: values.

Other Cements

Portland-pozzolana cement : A general-purpose cement with special properties, particularly useful in marine and hydraulic constructions. Pozzolana cement has delayed setting properties.

Portland slag cement : An ideal cement for general construction jobs. Very useful in dams and large foundations.

Hydrophobic cement : A water-repelling cement which retains its properties in humid and wet weather conditions.

Silvicrete white cement : A cement base excellent for floorings, exterior and interior finishes, and cement-based paints.

Oil-well cement : Used for the cementation of oil wells.

Rediset : A rapid-setting instant-strength cement, used in the precast concrete industry and for rapid repairs on concrete jobs.

Shrinkkomp : Shrinkage compensated expansive grout which ensures a good hold on anchors, foundation bolts and accurate setting of heavy machinery on foundations.

Calundum : A high alumina refactory binder having hydraulic-setting and rapid-hardening properties.

Firecrete and Firecrete Super : Hydraulic-setting, rapid-hardening, high-temperature resisting readymix refractory castables for furnace linings.

Cal-Al-75 : A high purity refractory binder of high temperature castables and for high thermal and electrical resistivity.

Whytheat : An import substitute castable with a service temperature up to 1800°C, used as a lining for oil refineries, iron, steel and petrochemical industries.

Insulyte : A range of low-density, high-temperature insulating refactory linings, for temperatures ranging from 1100°C to 1350°C.

Accoset-50 and Accoset Super : High-temperature refractory mortars that withstand temperatures up to 1700°C and are used as mortar for monolithic lining.

Accocid : An acid-proof mortar, resists the corrosive action of acids and chemicals.

Accoproof : A waterproofing additive to safeguard against dripping roofs and damp walls.

STEEL REINFORCEMENT

The reinforcement may consist of any of the following types :

(*i*) Plain mild steel bars, (*ii*) Medium tensile steel bars, (*iii*) Mild steel and medium tensile steel deformed bars, (*iv*) Cold twisted plain rounded or deformed bars, (*v*) Hard-drawn steel wire fabric, (*vi*) High yield strength deformed bars ribbed-torsteel. The most commonly used reinforcement is plain mild steel bars and ribbed torsteel.

Permissible stresses given for ribbed torsteel at pages 8/11, 12 are about the same as for Tata's Tiscon 42 or Fe 415 steels.

Plain bars may be either round or square, the former being the more common. Common standard diameters are : 5, 6, 8, 10, 12, 16, 20, 22, 25, 28, 32, 36, 40, 45 and 50 mm. Sizes smaller than 5 mm are khown as wires and are manufactured in standard wire gauge sizes. Mild steel rods for reinforcement should be obtained from standard manufacturer. Re-rolled rods manufactured by small mills (generally known as commercial steel and sold at cheaper rates by petty dealers) are not generally of full strength.

Deformed and twisted bars have higher initial cost than those plain bars but lesser steel is required as these bars have higher tensile strength and yield point. Deforming the bars increases the bond between steel and concrete, and end hooks can be omitted. These bars are manufactured in a variety of proprietory shapes and names. The nominal size of a deformed bar is defined as that eqivalent to the diameter or side of a plain bar having the same weight per unit run as the deformed bar.

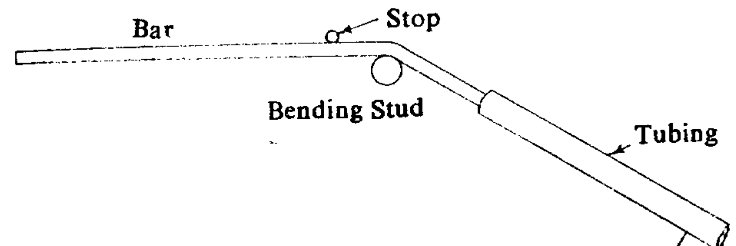
Deformed bars have some type of corrugations on their surface, and indented bars have dints on the surface.

Fabrics are used mainly for slab and wall reinforcements and also for road slabs where tensile stresses develop in both the faces of the concrete surface. Use of fabric is more convenient than placing individual steel rods in two layers at right angles to each other. There are a number of proprietory patterns with different strength properties. Expanded metal (XPM) is also used.

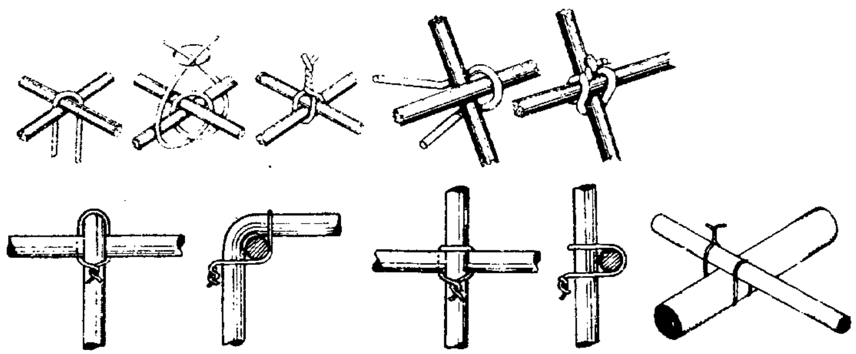
All reinforcement should be free from loose mill scale, loose or scaly rust, oil and grease, immediately before placing the concrete, otherwise adhesion between the steel and concrete is seriously affected. A thin discolouration or light rust which adheres firmly to the bars is not considered harmful and may be ignored. When steel rods are to be stored for sometime they can be given a cement wash to guard against rusting. Store the bars off the ground, and if they are to be in stock for long periods provide some covering to keep off the rain. Various sizes should be stacked seperately. Loose rust can be cleaned with a wire brush or hessian cloth. Oil, grease and paint can be easily removed by passing bars through fire or with the help of a blow-lamp.

Bending Bars. Wherever possible bars should be bent whilst cold. Bars bent hot should not be cooled by quenching in water or oil. Reinforcement should not be bent or straightened in a manner that will injure

the material. Twisted bar should not be heated. Welding may be allowed at the joints but such joints must be located at positions where the steel is not subject to more than 75 per cent of the maximum stresses and the welds should be so staggered that at any one section not more than 50 per cent of the rods are welded.

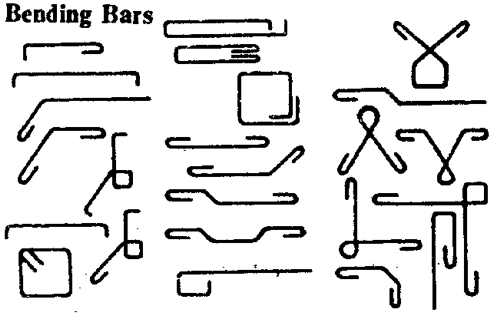


Bending bars by hand



Sketch Showing Binding together of bars with 1.25 mm soft iron wire

Bending Bars



1/8 in. bar	is taken as	3 mm
3/16 in.	,,	5 ,,
1/4 in.	,,	6 ,,
5/16 in.	,,	8 ,,
3/8 in.	,,	10 ,,
1/2 in.	,,	12 ,,
5/8 in.	,,	16 ,,
3/4 in.	,,	20 ,,
7/8 in.	,,	22 ,,
1 in.	,,	25 ,,
1-1/8 in.	,,	28 ,,
1-1/4 in.	,,	32 ,,
1-3/8 in.	,,	36 ,,
1-1/2 in.	,,	40 ,,
2 in.	,,	50 ,,

Placing Reinforcement. To ensure that reinforcement is correctly placed in position and is not moved during concreting, secured fixing is essential. If the bars get moved out of position, the member may lack strength and may fail under load. If reinforcing bars have too little cover they wlll rust, expand and eventually break up the concrete. Cover blocks or packing pieces can be made of concrete of size about 12×25×40 mm. The use of pieces of aggregate as packing pieces is a bad practice and should not be allowed. Never allow the reinforcement of a slab to be laid on the formwork and raised as the concrete is placed. After reinforcement has been fixed care should be taken that it is not damaged or displaced by men walking over it.

Bars should be cut with a tolerance of 25 mm or so in length. It is best to mark off the length from the centre of a bar towards each end if the bar is hooked or bent at both ends. Bars hooked at one end only should be marked off from the straight ends towards the hooked end. Use a steel tape for measuring bars. The diamention given should follow the convention measuring from "outside to outside" except where shown otherwise. Mark out the first bar according to the given diamensions and check it after bending. Base the diamensions of subsequent bars on this first one, making alterations where necessary. This is particularly important where a bar has a number of bends.

ROUND & SQUARE STEEL BARS

Areas & Weights

Dia. or Side		Round Bars			Square Bars
mm	in.	Weight per Metre Length kg	Cross Sectional Area cm^2	Circumference cm	Weight per Metre Length kg
5	0.20	0.154	0.200	1.57	0.20
6	0.24	0.222	0.283	1.88	0.28
8	0.31	0.395	0.503	2.51	0.50
10	0.39	0.620	0.785	3.14	0.78
12	0.47	0.888	1.131	3.77	1.13
14	0.55	1.208	1.539	4.40	1.54
16	0.63	1.578	2.010	5.03	2.01
18	0.71	2.000	2.545	5.65	2.54
20	0.79	2.465	3.142	6.28	3.14
22	0.87	2.983	3.801	6.91	3.80
25	0.98	3.852	4.909	7.85	4.91
28	1.10	4.832	6.158	8.80	6.15
32	1.26	6.311	8.042	10.05	8.04
36	1.42	7.990	10.180	11.31	10.17
40	1.57	9.860	12.566	12.57	12.56
45	1.77	12.49	15.904	14.14	15.90
50	1.97	15.41	19.635	15.71	19.62
56	2.20	19.34	24.630	17.59	24.62
63	2.48	24.47	31.172	19.79	31.16

The weight of steel is taken at 7.85 grams/cu. cm or 0.785 kg/sq. cm per metre length. Also at page 4M/37

AGGREGATES

Quality of aggregates. Since characteristics of concrete are directly related to those of its constituent aggregates, aggregates for load bearing concrete should be hard, strong non-porous, free from friable, elongated and laminated particles, and should be suitable for the purposed required. Stones absorbing more than 10 per cent of their weight of water after 24 hours immersion in water are considered porous. Porous materials corrode reinforcement. A friable aggregate will produce a concrete of similar nature. Elongated or laminated particles are weak in shear. Stones having mica inclusion should be avoided. Stones of the varities of granite, quartzite, trap and basalt, and those with rough non-glossy surface are considered best. All sand-stones tend to be porous. Soft varities of sand-stones make poor cancretes and also produce shrinkage cracks. Limestone is quite good provided it is hard, crystalline and entirely free from dust. Limestone should not be used in works subject to excessive heat. Both lime and sand-stones and other porous stones are not suitable for structures retaining water.

Aggregates must be clean and free from clay, loam, vegetable and other organic material. Clay or dirt coating on aggregates prevents adhesion of cement to aggregate, slows down the setting and hardening of the cement (concrete) and reduce the strength of the concrete.

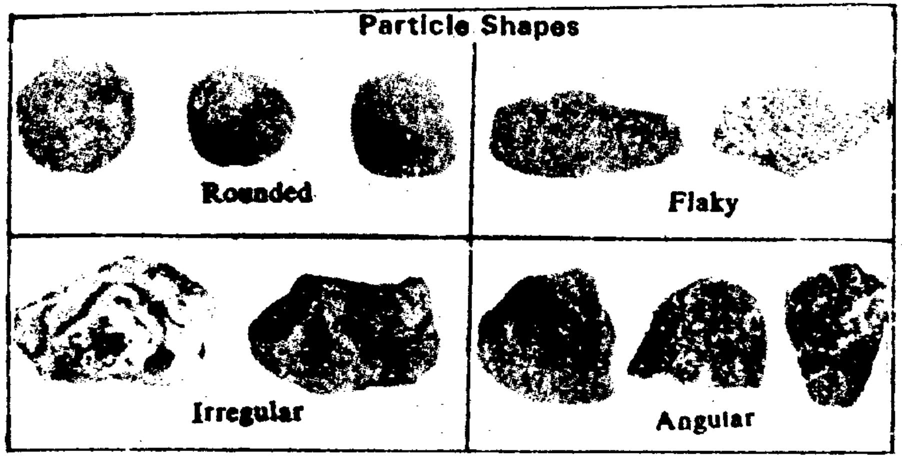
The material retained on a 4.75 mm IS sieve is classified as *coarse aggregate,* and below that size as *fine aggregate* or *sand.* The material passing a 75-micron IS sieve (No. 200 BS sieve) is generally considered to be clay, fine silt or fine dust in an aggregete.

Coarse aggregates should be ordered in separate sizes and recombined in the proper proportion while batching. A 40 mm nominal maximum size aggegate will be ordered in three different—40 mm to 20 mm, 20 mm to 10 mm, and 10 mm to 4.75 mm. A 20 mm nominal maximum size will be ordered in two sizes—20 mm to 10 mm, and 10 mm to 4.75 mm. Separate stockpiles should be maintained for the different sizes.

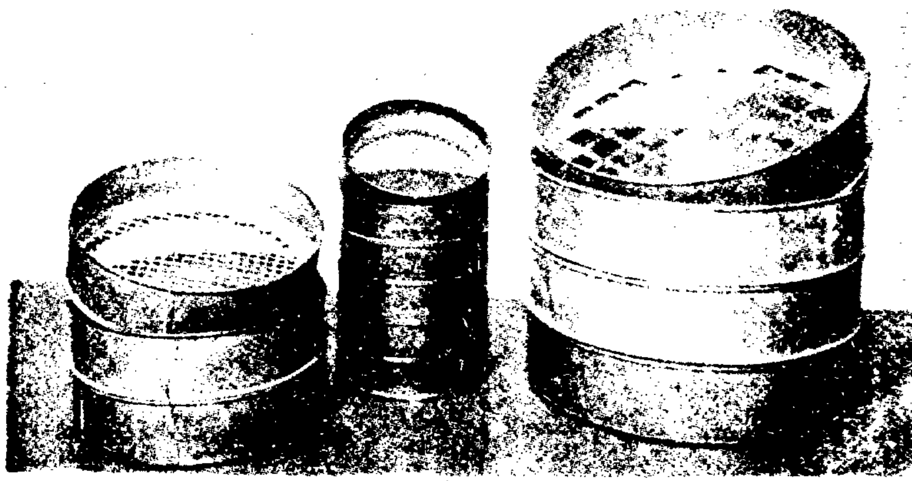
Angular and roughly cubicle particles are ideal. River gravel make the best coarse aggregate.

Shape and Surface Texture of Aggregate Particles

Shape is classified into four headings : rounded (or spherical), irregular, angular and flaky (or elongated). Rounded are fully water worn river or seashore, gravels. Irregular are partly shaped having rounded edges pit sands and gravels. Angular possess well defined edges and are crushed rocks of all types. Flaky is usually angular of which the thickness is small relative to the width and/or length. (See figure in Section 18 under "Selection of Stone Metal".) Surface texture is clasified under six headings as glassy, smooth, granular, rough or pitted, crystalline, and honeycombed and porous, but for most practical purposes these can be condensed under three headings : smooth, rough, and honeycombed. Rounded and irregular gravels are smooth, or relatively so, and crushed stones are rough.



BRITISH STANDARD SIEVES FOR TESTING CONCRETE AGGREGATES



Both shape and surface texture affect the workability and possibly density and strength of concrete ; shape is the more important factor. The workability increases as the aggregate particles become smoother and rounder. Roughly spherical (or rounded) aggregates produce the most workable concrete when the mix proportions and the water/cement ratio are unchanged. Concrete made with sharp angular aggregate (crushed rock or crushed gravel) is considerably less workable and also needs more sand and more cement ; but angular particles interlock better. Angular pieces have more voids than rounded ones. For the same degree of workability an angular aggregate may produce a concrete having a crushing strength some 50 per cent lower than a waterworn and relatively rounded aggregate.

Excess of thin, flat, elongated or flaky particles should be avoided as they produce harsh unworkable mix and are not suitable for strength

bearing concrete works. Aggregates with rough surface also produce weaker concrete.

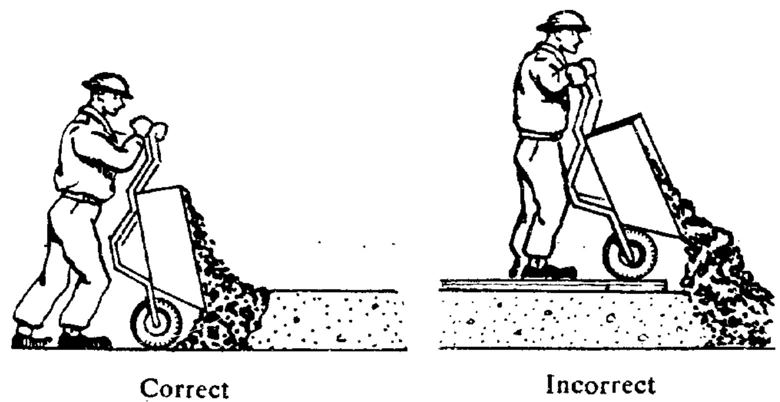
Rounded aggregate (shingle-bajre) require about one cu. m of less cement, about two cu. m of less sand per 100 cu. m of concrete and about 1.5 litres water per bag of cement than angular pieces to produce the same workability, and hence should be preferred where available. Gravel (rounded aggregate) is a very suitable aggregate for water retaining stuctures.

Brick Aggregate. This is often a good material suitable for plain concrete works but not for reinforced concrete due to its porous nature. Great care should be exercised in choosing bricks as some bricks contain sulpher and unslaked lime. Brick aggregate should be saturated with water before use to avoid absorption of the mixing water which is necessary for the hydration of cement and for the setting and hardening of the concrete.

Coarse aggregate of porous nature with a percentage increase of over 10 per cent for cement concrete and 25 per cent for lime concrete on dry weight, after immersion in water for 24 hours, should not be used. Brick aggregate is more fire resistant than broken stone but is not suitable for water-proof construction.

In locations where good quality stone is not available, hard-burnt or over-burnt varieties and of non-porous nature of brick ballast may be used for reinforced works where stresses are not very high. Laterite and jhamma agaregate (over-burnt) should be avoided in reinforced concrete constructions open to atmosphere and where there is chance of being open to alternate wet and dry conditions. A certain percentage of pozzolana used with brick aggregate concrete will 'kill' free lime in the cement which is the element which causes rusting of steel in alternate dry and wet conditions.

Storing or Stockpiling of Aggregate



Placing Concrete

During storing or handling of aggregate it is of utmost importance to see that there is no segregation *i.e.*, separation of the various sizes of particles. Stockpiling segregation does take place if successive consignments are dropped on the same place each time and it forms a pyramid like heap, as the coarser materials roll down the sides of the pile while the finer particles stay on in the centre of the pile on concentration at the top. Therefore, all the material should not be piled at the same place but should be placed in individual units side by side not larger than a truck load and should not be thrown from a height as this will also result in segregation by the winds. Each separate size of coarse aggregate and that of sand should be stacked separately, large in area and low in height—1 to 1.5 metres. All washed aggregate should be stacked for draining at least 12 hours before being batched.

Fine Aggregate or Sand

Fine aggregate or sand is the material most of which passes through 4.75 mm IS sieve. It is used as an ingredient of concrete that fills the voids in coarse aggregate to produce a dense concrete and to reduce the quantity of cement. Sand is usually obtained from the following sources :—

Sea sand—Particle sizes are often too fine and too uniform for good class work. Sea sand should not be used in its natural state. Salts will attack reinforcement ; if content is high it will retard setting and hardening of cement and may cause efflorescence but it may not have any deleterious effect on ultimate strength of the concrete. Thorough washing will remove most of the salt content. Sea sand must be tested for orgnic impurities. Presence of salt in sand can be detected by taste.

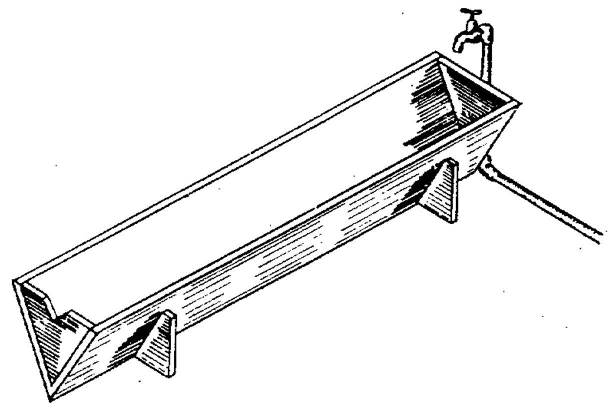
Pit sand—Sand obtained from old abandoned beds of rivers. Is usually considered to be the best. Pit sand has sharp angular grains while river sand is fine with rounded grains.

Fresh water, river or lake sand—Is usually quite good but may be contaminated with mud. Sands obtained from river beds or pits are often found mixed with clay, silt and mica.

Crushed stone—Screenings from crushed stone often contain a high percentage of dust and clay and may tend to be flaky. Flaky or angular particles may produce a harsh concrete.

Sand is either round or angular in grains and is often found mixed in various gradation of fineness. The sand used for mortars should consist of sharp (*i. e.*, angular) grains of various sizes. It is generally considered that rounded grains do not interlock sufficiently to produce a strong mortar. Recent tests however, have shown that as good a concrete can be made from sand consisting of rounded grains as from that in which the grains are angular. The grains of sand which are angular give the best results of tensile strength, those that are round give the highest compression strength. Experiments have shown that considerable variation in the strength of mortars may occur owing to the form and variety of the sand particles ; the strength of a mortar may differ by about 50 per cent of the average. Sand particles should, however be hard.

Colour of sands vary from deep brown to white and variations of colour may be found in the same quarry. Deep brown colour is due to the presence of traces of iron.



Trough for Washing Aggregates

Impurities in Sand. Clay, silt, salts, mica and organic matter are a source of weakness in any sand. All sands are generally found to contain some percentage of silt and clay. Mica is easily discernable from its shining surface. A certain percentage of impurities are inevitable in sand ; a maximum of 6 per cent of silt and 2 to 3 per cent of mica is usually allowed. Sand should also be free from particles of shell. Coal residues are particularly harmful as they may have a corrosive effect on reinforcement.

Test for presence of silt or clay in sand. A rough field test may be carried out by rubbing a sample of the sand between damp hands and noting the discolouration caused. Clean materials will leave the hands only slightly stained and such a sand is good for ordinary purposes. If the hands stay dirty after the sand has been thrown away, it indicates the presence of too much silt or clay.

(ii) Half fill a glass tumbler with sand and pour in clean water until the tumbler is three-quarters full. Shake up vigorously and leave it to settle for about an hour. Clean sand will settle immediately and presence of clay will show the water muddy. Any clay or silt will settle slowly on the top of the sand. If salt is added in water, one teaspoonful to a pint, it will quicken the process and silt will settle in a layer on top of the sand. Thickness of silt layer should not exceed one-seventeenth, or 6 per cent, of that of the sand below. If the thickness of the silt layer is more, sand needs washing. This is called *decantation test*. This test is not applicable to crushed stone sands.

A small percentage of silt or clay (not exceeding 1 to 2 per cent) is considered to improve the plasticity of a mortar to some extent, but an excess causes reduction in strength. In a very coarse sand, it may sometimes be considered desirable to introduce a small percentage of silt in order to improve its harshness. The material passing a No. 200 BS sieve is generally considered to be the clay, fine silt and fine dust in an aggregate. The permissible limits for silt, clay and fine dust as given in B. S. S. 882 are :—

Natural or crushed gravel sands . .	4	p.c.
Crushed stone sand	10	p.c.
Coarse aggregate of either type . .	1	p.c.

Clay forms a sort of film on the particles of sand and prevents or reduces the adhesion of cement to the sand particles ; retards the setting of cement, increases drying shrinkage. Clay having a greater surface area than sand increases the amount of water required for the mix, and thus reduces the ultimate strength of the concrete or mortar.

Test for Organic Impurities in Sand

A simple test for determining the presence of injurious organic matter in sands is made by shaking some of the sand in a plain glass bottle with an equal volume of a 3 per cent solution (100 gm in 4 litres of water) of caustic soda, and allowing the mixture to stand for 24 hours. The liquid above the sand should then not be darker than light straw (pale yellow) colour. If the colour is a marked yellow or brown the presence of an excessive amount of organic matter is indicated.

Such impurities can be removed by washing the sand. Washing has the additional advantage of removing any salts in the sand. Organic impurities in sand may be either due to decayed vegetation, humus, coal particles, or organic slimes and industrial wastes depending upon the source of the sand. It is generally considered that organic impurities retard the setting of cement and thus have deleterious effect on the strength of concrete or mortar. (Some laboratory tests have made this point doubtful.) Whatever be the effect of organic impurities on the behaviour of a sand, it is considered desirable to remove this impurity as much as possible. Organic impurities may also be present in the mixing water.

Test for presence of sand in cement. Dissolve the mixture in hydrochloric acid, sand remains undissolved.

Determination of Mix Proportions : Approximate Analysis of Fresh Concrete. Mix a small quantity of the concrete with water and then screen through a 4.75 mm mesh sieve ; deposits on the screen is coarse aggregate. Take the liquid in a glass jar and stir it vigorously with a rod and allow it to settle. Sand will settle at the bottom and cement will deposit on its top as a clearly defined sediment.

Bulking of Aggregates. Bulking in aggregates is caused by two factors. One is the method of filling the measurement boxes, the other is due to presence of water in the aggregate. In coarse aggregate the surface water does not cause any measurable bulking. Here the bulking is solely caused by the method of filling the boxes. Measurement of sand accurately is a little difficult. There is a great deal of difference in a unit

volume of dry compact sand (rodded) and dry loose sand. Allowance has to be made for this variation.

Bulking of Sand

Sand when damped bulks (expands) and occupies more space than it does when completely dry. This bulking increases with increasing water content until the sand is completely saturated when the bulking is practically nil and it has almost the same volume when dry. Sand has minimum volume when absolutely dry or absolutely wet. A moisture content of 2 to 5 per cent will increase the volume by 10 to 20 per cent or even 30 per cent. Fine sand bulks more than coarse sand. Even gravel of small size has been found to bulk as much as 5 to 10 per cent.

The amount of bulking can be readily determined at site. A suitable method is as follows : A parallel sided container is partly filled with the damp sand, levelled off but not pressed down and its depth is measured. The sand is then well mixed and stirred with plenty of water and allowed to settle. The volume occupied by the sand after settling is then roughly equal to that which would be occupied by the same weight of sand when dry. This new depth of sand is also measured.

If D be the depth of the sand when damp and D_1 the depth after settling under water ; then the percentage bulking $= \dfrac{D-D_1}{D_1} \times 100$, and this additional percentage of sand should be added to the mixture to give the required proportions. Thus a mix specified to be 1 cement, 2 sand, 4 coarse aggregate by volume (dry), will require 2.5 parts damp sand if sand bulked 25 per cent is used.

This increase of volume must be allowed for in the concrete mix otherwise the concrete will be under-sanded giving rise to excessive voids and weakness. The figures of proportions given (for cement : sand : aggregate) are nearly always for dry materials. Errors due to bulking of sand can be entirely eliminated by using "weigh-batching" in preference to volume batching as the weight of sand is very little affected by dampness and with weigh-batching the maximum error can be of about 5 per cent only. If volume batching must be used, the gauge boxes should be increased in volume over that required for dry sand so that an allowance is automatically made for bulking, and the gauge boxes should be deep and of small cross-section to avoid excessive errors arising from filling.

A correction must also be made in the quantity of mixing water to account for the moisture in the sand which otherwise would influence the water-cement ratio.

Water drains quickly through sand in a vertical direction but slowly on a gentle fall.

The free water in sands as delivered to the job site normally ranges from 2 to 6 per cent by weight, but may reach 8 per cent or more if the sand is extremely wet. Coarse aggregates seldom contain over 2 per cent of free water by weight ; traprock and granite may contain only half per cent and pebbles and crushed limestones 1 per cent, while porous sandstones may contain 7 per cent and very light and porous aggregate as high as 25 per cent. The coarser the aggregate the less water it will carry. If, on the other hand, the aggregates are air-dry, they will absorb up to 1 per cent of their weight of water before reaching a saturated, surface dry condition. Dry aggregates that are extremely porous will, of course, absorb serveral times this amount of water.

Control of Water Content. Measurement of water is a somewhat difficult task. Amount of moisture in aggregate varies as the day temperature and varies again with the different stack position of the aggregate. Slump test is the best guide in the field. Even with fixed quantity of water poured into the mixer, the slump varies from time to time. If it it stiff, just add a litte water for required consistency ; if it is sloppy, add some cement and fine sand mixture of the same mix as of the concrete.

Brick Aggregate

The following stresses may be adopted in brick aggregate reinforced concrete design calculations :

Compressive stress in 1 : 2 : 4 concrete	
Tensile stress in steel (untested)	40 kg/cm²
Shear	1200 kg/cm²
m—modular ratio	3 kg/cm²
kd—neutral axis	20
jd (lever arm of resisting moment)	0.400 d
p (reinforcement) %	0.867 d
Q (moment factor) = BM	0.0067 bd
	7.136 bd²

The proportion of voids can be estimated by filling a measure with the aggregate and then pouring in water until the water is level with the top of the aggregate. The ratio of the volume of water, added to the volume of aggregate, is that of the volume of voids to that of aggregate.

Test Sieves

A "sieve" has square apertures and the mesh of the sieve is indicated by the number of divisions per inch length. A "screen" has circular apertures and is described by the diameter of the circular openings. BS is British Standard used in England, ASTM and Tyler sieves are used in America.

The following Indian Standard Test Sieves are generally required for the grading of aggregates :

3.35-mm, 2.36-mm, 1.18-mm, 600-micron, 300-micron, 150-micron, 75-micron. (Fine Test Sieves made of mesh wire cloth).

80-mm, 63-mm, 50-mm, 40-mm, 31.5-mm, 25-mm, 20-mm, 16-mm, 12.5-mm, 10-mm, 6.3-mm, 4.75-mm. (Coarse Test Sieves made of square hole perforated plates).

COMPARISON OF DIFFERENT STANDARD SIEVES

Indian Standard Sieves Designation	Aperture mm	Equivalent (approx.) BS No.	ASTM No.	Tyler No.
4.75 mm	4.75	4—3/16″	4	4
4.00 ,,	4.00	—	5	5
—	3.35	5	6	6
2.80 ,,	2.80	6	7	7
2.36 ,,	2.36	7—1/10″	8	8
2.00 ,,	2.00	8	10	9
—	1.68	10	12	—
—	1.60	—	—	10—1/16″
1.40 ,,	1.40	12	14	—
1.18 ,,	1.18	14—1/21″	16	14
1.00 ,,	1.00	16	18	16
—	0.85	18	20	20
710 micron	0.710	22	25	24
600 ,,	0.600	25—1/42″	30	28
500 ,,	0.500	30	35	32
425 ,,	0.425	36	40	35
355 ,,	0.355	44	45	42
300 ,,	0.300	52—1/84″	50	48
250 ,,	0.250	60	60	60
212 ,,	0.212	72	70	65
180 ,,	0.180	85	80	80
150 ,,	0.150	100	100	100
125 ,,	0.125	120	120	115
100 ,,	0.100	150	140	150
90 ,,	0.090	170	170	170
*75 ,,	0.075	200	200	—
*63 ,,	0.063	240	230	200
*45 ,,	0.045	300	270	—
—	0.044	—	325	—

The dimensional ratings specify the width of the apertures while the numbered figures indicate the number of apertures per linear inch (BS sieves).

*These sieves pass only dust.

Maximum Aggregate Size. The maximum size of aggregate is governed by the type of the work to be built. The bigger the maximum size of the aggregate, less the voids when the aggregate is graded. A coarse aggregate which has less void content is economical and will give higher strength for the same amount of cement in the concrete.

The maximum size of aggregate may be up to 160 mm for mass concrete, but size up to 225 mm has been used in dams. Aggregate of this size require careful mix design to avoid segregation and it is probably wise to limit maximum size to 80 mm. Large stones which are embedded in mass concrete works are called "plumbs". Plumbs should be sound and hard and should not be placed nearer than 160 mm to one another or to an exposed surface.

Estimated relation between the minimum and average crushing strengths of works cubes for different works conditions

Conditions	*Minimum strength as a percentage of average strength*
Excellent control with servo-operated weigh batching use of graded aggregates, moisture content determinations on aggregates, etc. Constant supervision.	80
Very good control with weigh-batching, use of graded aggregates, moisture determinations on aggregates, etc. Constant supervision.	75
Fair control with weigh-batching. Use of two sizes of aggregate only. Moisture content determinations on aggregates. Constant supervision.	70
Fair control with weigh-batching. Use of two sizes of aggregate only. Water content left to mixer-driver's judgement. Occasional supervision.	60
Poor control; inaccurate volume batching of all-in aggregates. No supervision.	40

The size and proportion of coarse aggregate should be as large as possible (subject to the limitations mentioned herein) for affecting the greatest possible economy and crushing strength, but it should not normally be greater than one-fourth in plain concrete and one-fifth in reinforced concrete, of the smallest dimension in the structure.. For heavily reinforced members the nominal maximum size of aggregate should be 5 mm less than the minimum distance between the reinforce ent bars for the minimum cover of concrete over the reinforcement whichever is less, provided that the concrete can be placed without difficulty so as to surround all reinforcement thoroughly and to fill corners of the formwork.

For most of the common reinforced concrete works, a maximum size of 20 mm for coarse aggregates is generally suitable. For heavy sections a maximum size of 40 mm and for mass concrete works size up to 80 mm may be used. For thin members such as ribs and top slabs, the largest size of aggregate is generally 10 mm.

Max: size of aggregate in mm for various types of works :

Min: size of section mm	65 to 125	150 to 280	300 to 740	760 or over
Reinforced walls, beams and columns	12 to 20	20 to 40	40 to 80	40 to 150
Unreinforced walls or mass concrete	20 to 25	40 to 50	80 to 150	150
Heavily reinforced slabs	20 to 25	40	40 to 80	80 to 150
Lightly reinforced or unreinforced slabs	20 to 40	40 to 80	80	80 to 150

Cinder Concrete

1 part of cement and 10 parts of cinders. Is light and porous. Has good heat insulating properties and can be used on top of roofs, laid about 6 cm thick with a slope of not less than 1 in 48 ; should have another layer of about 40 mm waterproof cement concrete over the top (as cinder concrete is not waterproof) with a layer of waterproof paper in-between.

Percentage of Voids in Sand and Aggregate

Aggregate	Void % (av.)
Sand, moist, fine	43
Sand, moist, coarse	35
Sand, moist, coarse and fine mixed, ordinary	38
Sand, dry, coarse and fine mixed	30
Gravel	27–37
Gravel and sand mixed	22–25
Ballast 20 mm and under, 6% coarse sand	33
Broken stone 25 mm and under	46
Broken stone 40 mm and under, dust only screened	41
Broken stone 50 mm and under, most small stones screened out	45
Ditto. 63 mm	41
Brick ballast	35–40

Approximate quantity of surface water (or free water) carried by average aggregate (IS: 456) :

Condition of aggregate	Litres per cu. metre
Very wet sand	120
Moderately wet sand	80
Moist sand	40
Moist gravel or crushed stone	20 to 40

Wet sand has less voids as water fills in some space.

As sand is seldom available in absolutely dry condition, 5 to 10 per cent extra sand may be added for common works.

Grading of Aggregates

A graded aggregate is one which is made up of stones of different sizes, ranging from large to small (inclusive of sand) so as to have minimum of air voids (and that will have maximum density) when mixed together. The voids in the mixed aggregate would be minimum when the sand is just sufficient to fill the voids in the coarse aggregate. Voids in the coarse aggregate are filled in by sand and voids in the sand are filled in by cement. Mix that occupies the least volume is the densest and will produce the best results. The volume of the coarse aggregate is generally taken as twice that of the fine aggregate, but variations of its proportions may be made within the limits of 1.5 to 3 times the volume of the fine aggregate to suit the size and grading to secure dense and workable concrete. Grading is of great importance since it affects workability of concrete and hense density and strength.

The proportion of voids to volume of well graded sand is 30 to 35 per cent and that of coarse aggregate between 30 and 45 per cent. This should represent the amount of cement required to fill the interstices in the sand and the amount of sand required to fill the interstices in the coarse aggregate. But it has been realized that when sand is added to coarse aggregate the particles of latter are separated by grains of sand so increasing the original volume of voids. To allow for this and to obtain workability, ten per cent extra sand and fifteen per cent extra cement to the percentage of voids in aggregate and sand are usually provided to arrive at the proportions of different materials for a particular mix.

Well graded sand from coarse to fine has less voids than fine sand. The lesser the voids the better is the quality of sand for use in cement concrete, provided there is no silt in sand.

The combined aggregate when mixed with the required quantiy of cement and water, should give a good workable concrete which can be readily placed in position without segregation. The proportion of fine to coarse aggregate should be such as will give maximum workability with minimum of water. Mixtures with a deficiency of fine materials will be harsh, hard to work and difficult to finish. A little more sand makes the concrete more fluid without extra water. Too much of sand will increase porosity of the concrete and need more cement. Mixes having a larger coarse aggregate require less water and less cement.

For concrete works to be water-roof, a dense mix should be aimed at with small size of aggregate. As little of cement as possible consistant with the required stength of the resulting concrete has to be used for purposes of economy.

While a drying water evaporates and leaves air voids, cement expands in the process and occupies some of the air voids.

Excess of fine materials need more cement and more water. A small amount of very fine material (silt and clay) passing 75-micron (No. 200 BS) sieve may improve the workability of the concrete, but an axcess causes reduction in strength. The material passing through 150-micron (No. 100 BS) seive must not exceed 10 per cent. Crusher dust in broken stone is

injurious when present more than 10 per cent. Clay particles although less injurious, but should not be more than 5 per cent.

Nominal mix is the proportion of cement, sand and broken stone ; all the three measured separately by volume (dry materials).

Real mix is the proportion of cement to a mixture of sand and stones by volume. Sand and stones when mixed together will occupy less volume than when measured separately.

Field mix. The proportions of wet sand and stones taken to make particular nominal mix is called a "field mix." (See under "Bulking of Sand.")

Faults in a Concrete Mix and Remedial Measures

Fault	Remedy
Mix too dry	Slightly reduce quantities of coarse and fine aggregate.
Mix too wet	Slightly increase quantities of coarse and fine aggregate.
Mix harsh and lacking plasticity	Slightly increase quantity of fine and slightly decrease quantity of coarse aggregate.
Mix excessively plastic and "fat"	Slightly decrease quantity of fine and slightly increase quantity of coarse aggregate.

Real mix proportions depends on the size of aggregate and the proportion of voids. Field mix proportion depends on the sand and aggregate.

Batching and Mixing of Concrete

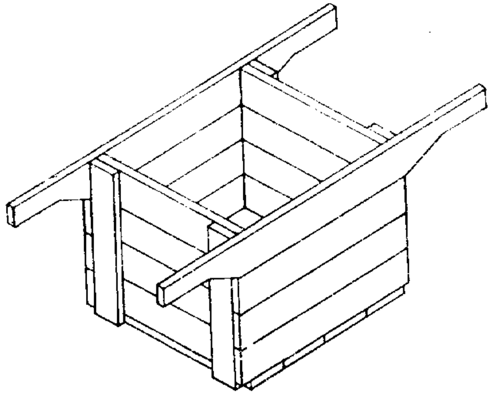
After fixing the proportions of the different ingredients of concrete for a particular work, the materials, viz., aggregates, cement, and water are measured out in batches for mixing. This process is called "batching". The batching may be done either by volume or by weight. When concrete is batched by volume there is always the possibility of variation between one batch and another.

For all important concrete works batching should be done by weight and mixing by machine mixers. Machine mixing ensures better and more uniform concrete. If volume baching is done, care must be taken to allow for the bulking of aggregate due to unequal filling of measuring boxes and moisture in the sand. (See under Bulking of Aggregate") otherwise more cement will be consumed and the mix will become undersanded and harsh.

For volume batching, cement is measured by bag, and water in litres. A 50-kg bag of cement is taken as 35 litres.

For measuring aggregates wooden boxes are made in units of one or more whole bags of cement, i.e., in unit of 35 litres. For example, a convenient size of box would be 40 cm long, 35 cm wide and 25 cm deep (inside diamensions). The boxes should be provided with handles for ease of lifting and loading the mixers. Separate measuring boxes should be provided for the different aggregates.

Where cement is measured by bag, it would be necessary to sample check the weights of the bags occasionally as quite a lot of cement oozes out through the gunny bags every time a bag is moved or handled. Sometimes loss is as high as 10 per cent. (The weight of a gunny bag is about 570 grams and that of a paper bag about 370 grams.)



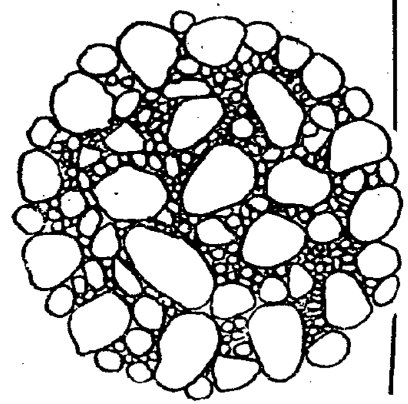
Typical gauge box

Density of water is 1 kg per litre. The quantity of water required is a product of water/cement ratio and the weight of cement. For example, if a water/cement ratio of 0.55 is specified, the quantity of mixing water required ber bag of cement is 0.55 x 50=27.5 litres.

The limiting proportions of particles of various sizes may be derived by any of the following common methods.

Table III Coarse Aggregates by Volume

Mix:	Nominal size of aggregate	Parts of aggregate of size				
		55 mm	40 mm	20 mm	12.5 mm	10 mm
1:6:12	63 mm	9	—	3	—	—
1:6:12	40 mm	—	9	3	—	—
1:5:10	63 mm	7.5	—	2.5	—	—
1:5:10	40 mm	—	7.5	2.5	—	—
1:4:8	63 mm	6	—	2	—	—
1:4:8	40 mm	—	6	2	—	—
1:3:6	63 mm	4.5	—	1.5	—	—
1:3:6	40 mm	—	4.5	1.5	—	—
1:3:6	20 mm	—	—	4.5	—	1.5
1:2:4	40 mm	—	2.5	1	—	0.5
1:2:4	20 mm	—	—	3	—	1
1:2:4	12.5 mm	—	—	—	3	1
1:1.5:3	20 mm	—	—	2	—	—



GRADING OF AGGREGATES

Grading Limits for Coarse Aggregates (Based on IS : 383)

Table I / Table II

IS Sieve	Table I: Percentage passing for graded aggregate of nominal size				Table II: Percentage passing for single-sized aggregate of nominal size					
	40 mm	20 mm	16 mm	12.5 mm	63 mm	40 mm	20 mm	16 mm	12.5 mm	10 mm
80 mm	100	—	—	—	100	—	—	—		
63 mm	—	—	—	—	85-100	100	—	—		
40 mm	95-100	100	—	—	0-30	85-100	100	—		
20 mm	30-70	95-100	100	100	0-5	0-20	85-100	100	—	—
16 mm	—	—	90-100	—	—	—	—	85-100	100	—
12.5 mm	—	—	—	90-100	—	—	—	—	85-100	100
10 mm	10-35	25-35	30-70	40-85	0-5	0-5	0-20	0-30	0-45	85-100
4.75 mm	0-5	0-10	0-10	0-10	—	—	0-5	0-5	0-10	0-20
2.36 mm	—	—	—	—	—	—	—	—	—	0-5

When coarse aggregate brought to the site is ungraded, single size coarse aggregates of different nominal sizes conforming to the requirements vide Table II, shall be mixed with the other ingredients of concrete in the proportion indicated in Table III.

Grading Limits for All-in Aggregates

IS Sieve	Percentage passing of nominal size 40 mm	20 mm	
80 mm	100	—	Aggregates should first be seprated into fine and coarse aggregate with 4.75 mm sieve, and then necessary abjustments made by the addition of single sized aggregates.
40 mm	95-100	100	
20 mm	45-75	95-100	
4.75 mm	25-45	30-50	
600-micron	8-30	10-35	
150-micron	0-6	0-6	

Grading Limits for Fine Aggregates

IS Sieve	Percentage passing for grading zone I	II	III	IV
10 mm	100	100	100	100
4.75 mm	90—100	90—100	90—100	95—100
2.36 mm	60—95	75—100	85—100	95—100
1.18 mm	30—70	55—90	75—100	90—100
600-micron	15—34	35—59	60—79	80—100
300-micron	5—20	8—30	12—40	15—50
150 micron	0—10	0—10	0—10	0—15

(i) For crushed stone sands, the permissible limit on 150-micron IS sieve is increased to 20 per cent.

(ii) As the fine aggregate grading becomes progressively finer from grading zone I to IV, the ratio of fine aggregate to coarse aggregate should be progressively reduced.

Aggregate Gradings based on British Standard Specifications

(*a*) Coarse Aggregates :

6 parts	63 mm
4 ,,	40 mm
2 ,,	20 mm
1 part	63 mm

For small quantities :

3 parts	63 mm
1.5 ,,	10 mm

(*b*) Medium Aggregates :

5 parts	40 mm
2 ,,	12.5 mm
1 part	3.35 mm

For small quantities :

3 parts	40 mm
1.5 ,,	10 mm

(*c*) Medium Fine Aggregates :

5 parts	20 mm
1 part	10 mm
1 ,,	3.35 mm

(*d*) Fine aggregate :

3 parts	10 mm
1 part	3.35 mm

Sand with above gradings :

Whole passing 3.35 mm screen, not more than 30% passing 850-micron screen not more than 60% passing 4.5 mm screen.

All-in aggregate for 20 mm maximum size, small quantities :

Passing 20 mm mesh	100%	Coarse aggregate
Prssing 10 mm mesh	55-65%	
Passing 4.75 mm mesh	35-42%	Fine aggregate
Passing 150-micron (No. 100 BS) sieve	3%	

The term "all-in aggregate" or "combined aggregate" is applied to a graded aggregate containing both coarse and fine materials in varying proportions.

When combined material graded from 4.75 mm to 40 mm is specified, this should be furnished in at least two separate sizes with separation at 20 mm sieve, and at 25 mm sieve when combined material graded from 20 to 50 mm is specified.

A nominal mix of 1:2:4 is not the same as a 1:6 mix using "all-in". aggregate. The equivalent of the 1:2:4 proportions would be approximately 1:5 when using "all-in" aggregate. Similarly, for a 1:3:6 mix it will be aproximately 1:7.5, and for a 1:4:8—1:10.5.

Except when otherwise provided for, all percentages specified are by weight.

Most gradings within the above range would be satisfactory for many purposes although they should not be considered as ideal gradings. The proportions specified are for dry materials.

A well graded aggregate of sand and stones should have voids between 20 and 30 per cent, and if more, the mix needs re-grading.

Different Methods of Measuring and Proportioning Mixes

The process of measuring materials for a mix is known as "batching". There are three methods :—(a) by volume, (b) by weight, (c) by a combination of (a) and (b).

Weight of a concrete depends on various factors—Cement has a specific gravity of 3.10, water-1, sand and coarse aggregate may have specific gravities varying from 2.5 to 3.20. Voids content of the aggregate also vary from 28 to 50 per cent. The weight of a particular concrete depends on all these factors.

(i) Proportioning by specific mixes (or arbitrary standards) called the mix method, in which the proportions of cement, sand and aggregate are specified, *viz.*, 1:1.5:3, 1:2:4, 1:3:6, etc. This is the simplest and usual method for common works. The strength of the concrete produced by this method is variable and uncertain and it is not thus suitable for important works where high strength is required. This is also an uneconomical method for large works. The following mix proportions are generally specified for various types of works :—

Mix	Type of construction
1:1:2	Heavily loaded columns and long span arches.
1:1.5:3	Water tanks, top of road surface, piles, fence posts, pre-cast RC works and other works where dense concrete for impermeability or exta strength is required.
1:2:4	Normal RC works and for ordinary uses of concrete such as in beams, columns, walls, arches, road slabs.
1:2.5:5 1:3:6	Mass concrete in superstructure, massive RC members, bases of machinery, walls below ground, footings, cement concrete blocks.
1:4:8 1:5:10 1:6:12	Mass foundations. Lean mixes are prescribed for foundations as controlling factor in founds is usually the bearing area and not the allowable stress in concrete.

(ii) Proportioning by minimum voids method. The voids of coarse aggregate are ascertained and a quantity of fine aggregate is used so as to be equal to the voids of coarse aggregate plus 5 to 10 per cent extra to allow for the fine aggregate to wedge in-between the coarse aggregate. The quantity of cement paste used is similarly made equal to the voides in the combined aggregate plus 10 per cent. One 50-kg bag of cement is assumed to produce 0.03 cu. m of cement paste.

(iii) Proportioning by trial mixes. It has been stated earlier that to produce densest concrete the mixture should give the smallest volume for the same weight or the heaviest weight for the same volume for a particular kind of aggregate. A box is filled with the varying proportions of the coarse and fine aggregate and slightly shaken. The proportion which gives the heaviest weight will produce the densest concrete. Cement paste is then prepared, the quantity of water being in accordance with the water-cement ratio for the required strength or the particular mix. Mixed aggregates giving the heaviest weight (determined as stated above) are then added until the consistency is suitable for the work.

(iv) By sieve analysis or Fineness Modulus. The aggregates are separated by various sieves into different sizes of particles and mixed to the required proportions. Certain values of Fineness Modulus for mixed aggregates and varying with the maximum size of aggregate have been accepted as giving the best results. (Also see under "Glossary of Terms").

Sieve analysis takes a lot of time, checking void content is a good guide and takes much less time.

Roughly, 100 units of coarse aggregate mixed with requisite quantity of sand, cement and water, produces 110 to 120 units of concrete with normal proportions of mixes, and 108 to 116 units of concrete for foundation works (consolidated) with lean mixes.

8. METHODS OF CONSTRUCTION

Mixing and Placing

Whether the concrete is mixed by hand or in a mechanical mixer, it should be thoroughly mixed and the concrete placed in its final position with the minimum of delay. After placing it should be well compacted by rodding, tamping or vibrating to remove all air pockets (voids). Failure to do this can have very serious effects on the finished concrete. Presence of 5 per cent of air voids may reduce the strength of concrete by 30 per cent and presence of 10 per cent of voids by as much as 50 per cent. Badly compacted concrete is also likely to contain unsightly patches of honey-combing or porous concrete which in turn may well lead to the corrosion of any reinforcement used and to spalling of the faces. For a concrete of high quality good compaction is essential. This may often mean extra work during placing but on no account should more water be added in order to reduce the work of compacting.

Machine Mixing

Types of Mixers. There are two main types of machine mixers, continuous and batch. Continuous mixers are used for works involving large masses of concrete, and produce a continuous flow of concrete. Batch mixers are the most commonly used and are of two main types : tilting, in which the drum tilts to discharge its contents ; non-tilting, which is empetied by means of a chute. With these mixers the required quantities of materials are placed in a revolving drum which is completely discharged after each mix ; the drum is fitted with blades which turn over the materials. For normal works there is little to choose between the two above-said type of batch mixers. However, more uniform mixing with less time can be obtained from non-tilting types.

Most of the mixers are satisfactory with wetter mixes but difficulty arise for mixing dry concrete and there is incomplete discharge and the concrete produced is also non-uniform and less workable as some mortar remains sticking with the blades of the mixer, this is especially so with non-tilting mixers. Lean dry mixes with large size of coarse aggregate are more difficult to mix uniformly than rich and more workable concretes. Sticking has a serious effect on the quality of the concrete. Some improvement with dry mixes can be obtained by introducing all the water into the mixer before the dry materials, or by hammering the mixer drum during the mixing and discharging operations.

Sizes of batch mixers were hitherto denoted by two numbers, *e.g.*, 5/3.5, 7/5. The first indicates the drum capacity in cubic ft. for dry materials (measured separately) and the second the approximate feet cube of concrete produced by that quantity of unmixed materials. With the adoption of metric units, the size of a mixer is designated by a number representing its nominal mixed batch capacity in litres. The equivalents in cubic feet and litres are as follows :

Cubic ft.	5/3.5	7/5	10/7	14/10	18/12	22/14	45/28
Litres	100	140	200	280	340	400	800

Letter T is fixed with a tilting type and letter NT with a non-tilting type.

The batch mixers that are in general use are easily portable.

Where aggregates are to be measured by volume the mixer should be sufficiently large to take a whole bag mix so as to avoid the necessasity of measuring the cement which would make errors. Thus the minimum size of a mixer for a 1:2·4 mix will be 10/7 (200 litres) and for a 1:3:6 mix a 14/10 (280 litres) will be required. The quantity of material mixed should not exceed the rated capacity of the machine.

The coarse aggregate should be placed in the hopper first followed by sand and then cement. The drum should be revolving when it is charged. A small proportion of the water should be placed in the drum before the dry materials are put in. This will prevent the accumulation of cement paste around the blade roots. The rest of the water may be poured in simultaneously with dry materials. There is no need for any dry mixing before the water is added.

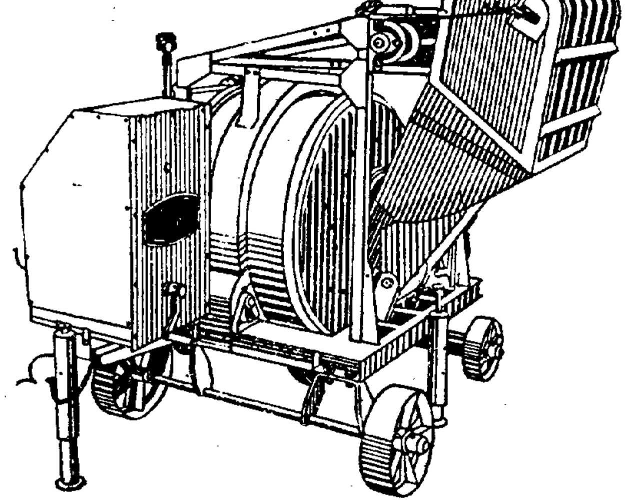
The concrete should be mixed until it is uniform in colour and consistence, but for not less than 2 minutes after all the ingredients, including the water, have been added to the mixer. Where the mixed concrete is transported in trucks the mixing time is increased. No more concrete should be mixed than can be used before the initial set of the cement commences. At each mixer discharge the drum is washed with clean water which is completely removed along with any concrete if remaining, before putting in the new aggregate.

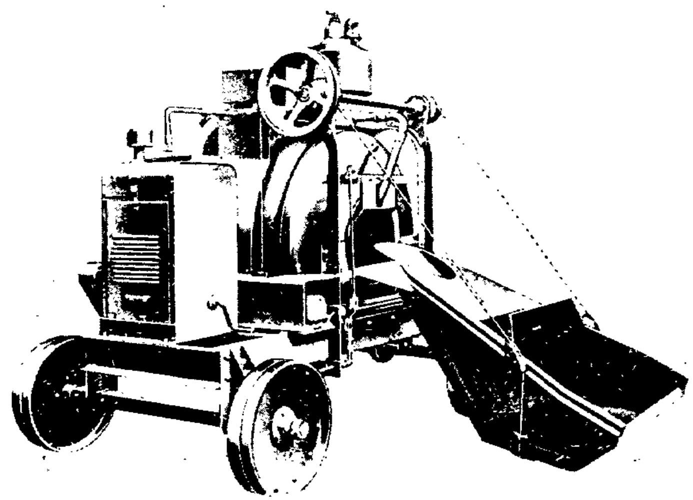
Hand-mixing should be done on a clean paved area or a water-tight platform at least 3.5 m × 2.5 m or 3 m square with strips or kerbs fastened along three sides to prevent the materials being washed or shovelled off during mixing. (An additional 10 per cent cement over and above the specified proportion should be used when hand mixing is resorted to.)

The materials should be mixed for at least two minutes. A rough test for good consistency is to take handful of the concrete and squeeze it tightly in the hand, if the mixture is good the sample will retain its shape when the pressure is released and the surface of the sample will be moist but not dripping.

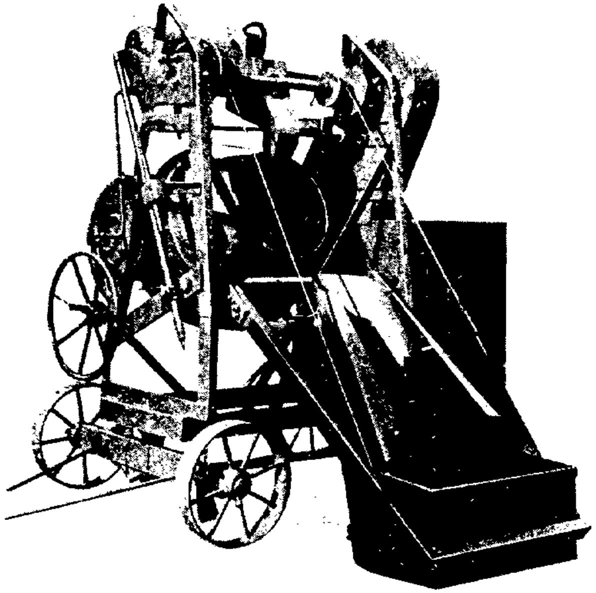
Cement and sand should be first mixed dry then aggregate added and whole turned over 3 times dry and then turned over 5 times wet and thoroughly mixed until the concrete is of a uniform colour. Measured quantity of water should be added from a can fitted with a rose.

Non-tilting Mobile Concrete Mixer

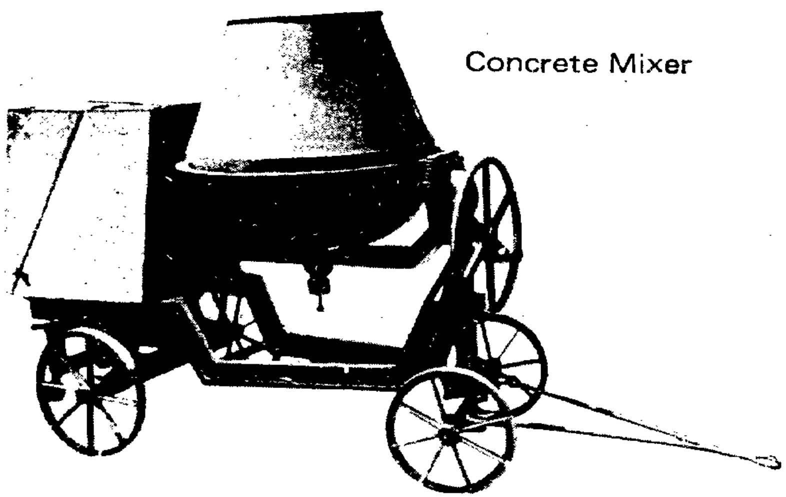




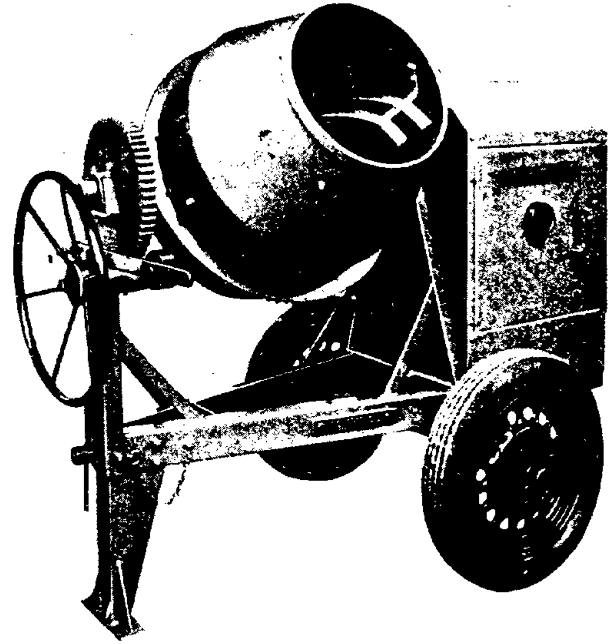
NON-TILTING TYPE



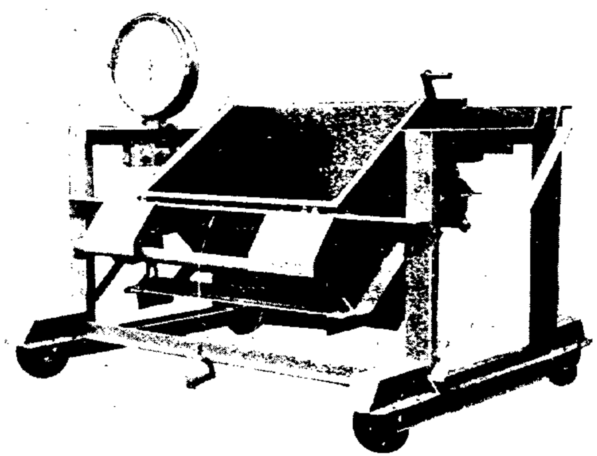
Tilting Drum Mixer



Concrete Mixer



TILTING MIXER



PORTABLE BATCH WEIGHER

Placing of Concrete and Preparation of Formwork

Concrete should be placed and compacted as soon as possible after it has been mixed with water and befor initial set of cement ; a maximum time-limit of 1.5 hours is allowed, and it should not be disturbed once the setting of cement has commenced. The concrete which has been left over may be used if it can be re-mixed to a workable consitency without the addition of more water, but if the concrete stiffens whilst being mixed and placed it should be discarded.

Mortar or concrete should not be re-mixed with water after it has partly set, except for patch repairs for which retempered mortar or concrete is better than fresh materials as it adheres better.

All debris, saw-dust, etc., should be removed from the shuttering before any concrete is placed. Care should be taken to see that the shuttering is not likely to absorb water from the concrete mixture. On common works, wet the form immediately before placing the concrete. The forms can also be given a coat of crude oil or grease thinned with paraffin, or soft soap and water, or whitewash, in order to prevent adhesion of concrete. Oiling should be done sometime before the reinforcement is placed in order to prevent greasing the steel. Where it is intended to plaster the concrete, oil or grease should not be used as it prevents the adhesion of plaster.

If ground or sub-grade is dry and absorbent where concrete is to be deposited, it should be covered with a layer of water-proof paper or similar material to prevent loss of water from the concrete.

Concrete should be placed in even layers each of which should be compacted before the next is placed. The thickness of layers is generally 15 cm to 30 cm for reinforced work and up to 45 cm for mass concrete.

The layers are to follow in quick succesions to prevent any distinct joint between them and each layer should be placed before the previous one has set. There should not be a gap of more than half-an-hour in placing of concrete between any two adjacent layers. As each successive layer is laid, the "laitance" film and the layers of porous concrete immediately below it must be removed before placing the new concrete. Care is to be taken not to disturb the partially set concrete.

Conveying Concrete. When handling, transporting or depositing concrete, care should be taken that the particles do not segregate. If concrete segregates during transit it should be re-mixed before being placed. Concrete should not be thrown from a height when brought in baskets and when dumped or dropped from a chute, the direction of its fall should be vertical.

When concrete has to be lowered any depth below 1.5 metres it should be conveyed in suitable receptacles or by chuting. Where chutes are used the slope of the chute should be so adjusted that the concrete moves without segregation of the materials ; and mixture has also to be sufficiently plastic which can travel at a speed that will keep the chute clean. A slope not flatter than 1 : 3 nor steeper than 1 : 2 is generally considered suitable. The troughs of the chutes should be flushed with water before and after each working period. The delivery end of the chute should be as close as possible to the point of deposit. Where the concrete has to be deposited at a higher level, this is done under air pressure by mechanical means.

The ribs of L-beams and T-beams shall normally be concreted together with the floor slab of which they form a part. Where however, the shear reinforcement provided is sufficient to prevent any risk of shear failure at the joint, the slab may be cast within a period of two days after the casting of the rib. Two hours shall elapse after depositing of concrete in columns of walls before the depositing of concrete in beams, or girders of slabs supported thereon.

Placing of concrete in columns, beams and slabs

Concrete in columns should be deposited in the centre to elimimate possibility of coarse aggregate getting between the formwork and stirrups and thereby obstructing flow and compacting of concrete. Concrete should be poured into the moulds in about 7 or 8 cm layers and constantly tamped and puddled with a rod to expel air bubbles. Concreting of columns should be stopped about 6 or 7 cm below the bottom of the beam concrete. Where it is necessary to cast the column, beam and slab togther, sufficient time should be given for the column concrete to sink—not less than two hours before the beam concreting is started. Access doors should be kept in the column form to have access water and laitance removed from top of column before beam concreting is started with a layer of grouting as usual.

In all vertical concretings it will be found that concrete in the forms is getting sloppier and sloppier, this is due to the excess water from the bottom layers trying to rise to the top. The concrete has thus to be made drier for the top layers.

In compacting concrete in columns, 10 to 15 cm timber posts from the top may be used. A vibrator shaft inserted within the core area may get scratched or cut by the hooks or stirrup steel and there is always fear of over vibration.

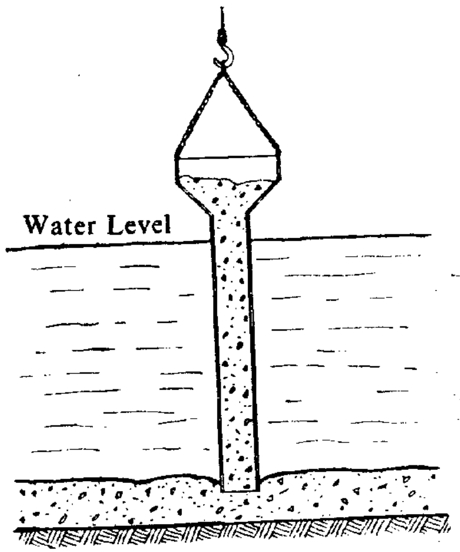
Concreting in walls and footings should be in horizontal layers. Concreting in walls can be started from both ends meeting at the centre.

Depositing Concrete Under Water

Under water placing of concrete is confined generally to mass unreinforced works. Whilst concrete will set and harden under water its placing presents several problems, the most difficult is the prevention of segregation and loss of cement. Consideration should always be given to the possibility of using pre-cast blocks for the whole of the work or as permanent formwork ; formwork is difficult to place accurately.

Where the concrete has to be deposited under water, one of the two courses may be adopted, either the space where the concrete is to be deposited be enclosed and water excluded temporarily, or the concrete may be placed directly in the water using one of the following methods :-

Tremie is a water-tight steel pipe strong enough to withstand the water pressure of 15 to 30 cm diameter and of sufficient length to permit the lower end reaching the bottom of the space while the upper end is above the water level. The upper end of the tremie is fitted with a hopper large enough to hold one entire batch of the mix poured in at one time. The lower end of the tremie is either equipped with an automatic check valve or the upper end of the pipe is plugged with a wadding of gunny sacking or some such material. When the concrete is poured in through the hopper the plug is forced down displacing air and water from the pipe. As concreting proceeds the tremie is gradually raised, the lower end, however, is kept submerged in the plastic concrete all the time while the concrete is being poured. It is essential that the bottom seal be kept unbroken whilst concrete is being placed, if it is broken there is a danger of cement being washed out of the concrete.



Placing Concrete Under Water by Means of a Tremie

Drop Bottom or Bottom Opening Bucket or Skip. This is a specially made bucket with top open end of which the bottom opens downward and outward when it is lowered down and tipped. The bottom doors are filled completely, covered with a canvas cloth and lowered slowly to avoid backwash until it rests on the bottom surface where the concrete is to be deposited.

Instead of buckets gunny bags can also be used which are filled with concrete and lowered down in water and opened at the bottom without disturbing the water. The laying should be started from one side and proceeded till all the surface is uniformly covered.

Grouting method is also useful for under-water works. Cement-sand grout is heavier than water having specific gravity of just over 2. The grout has a colloidal form repugnant to water and it displaces water without getting mixed with it. Packing of the dry aggregates within the shuttering is done first and a rich cement-sand grout (1:1 or 1:1.5) is poured through pipes already placed in the aggregate. As the grout is poured at the bottom and rises up, the water gets displaced. (See also under "Colcrete").

Another method is—A series of round cages made from 50 mm mesh of 6 mm steel wire, and extending over full height to be concreted, are prepared and laid vertically over the area to be concreted so

that the distance between centres of cages and also to the faces of concrete does not exceed one metre. Stone aggregates of not less than 50 mm or more than 200 mm size are deposited outside the steel cages over the full area and height to be concreted with due care to prevent displacement of cages.

A stable 1:2 cement-sand grout with a water-cement ratio of not less than 0.6 and not more than 0.8 is poured down under pressure through 40 to 50 mm dia. pipes terminating into steel cages, about 90 mm above the bottom of the concrete. As the grouting proceeds the pipe is raised gradually up to a height of not more than 600 mm above its starting level after which it may be withdrawn and placed into next cage, and so on.

Dry ingredients should not be dumped into water nor concrete should be let to fall through water from any height. The water under which concrete is laid should be quite still to get best results. Cofferdams or forms should be sufficiently tight to reduce the velocity of flow within them to 3 metres per minute to prevent loss of mortar through the wall. Pumping or bailing shall not be conducted while the concrete is being deposited, and within 24 hours after placing of the concrete. No tamping or ramming shall be done until the concrete surface rises above the water level.

The concrete mixture should have 10 to 30 per cent extra cement (not less than 390 kg/cu. m of concrete), and a slump of not less than 10 mm and more than 180 mm. The ingredients should first be mixed dry, and just sufficient quantity of water added so as to enable the mixture to be made into a ball without giving out water on squeezing. Aggregates should be properly graded to reduce the number of voids to the minimum. Aggregate of size 40 mm down to 3 mm will produce a concrete mass of excellent plasticity and strength.

Concrete should not be placed in very cold water. For cold water, rapid hardening cement should be used. Alternatively, an accelerator such as calcium chloride or silicate of soda should be added to the mix. See under "Cold Weather Concreting". For producing increased plasticity in concrete chemical constituents like powdered silica and extra cement can be used, the maximum quanitity of silica not to exceed 3 to 4 per cent by weight of cement.

No construction joints shall be allowed within 600 mm below water level or within 600 mm of the upper and lower planes of wave action.

By bags. Old cement jute bags are filled about two-thirds with concrete : open ends are securely tied, or better sewn, to make the bags square ended. These bags are deposited under water in alternate headers and stretchers courses so that all the bags are interlocked to form into one solid mass. Bags should be built in bond with the mouth of the bags away from the outside surface. Courses of bags may be held together by driving steel spikes through them after placing. In deep waters services of a diver may be necessary for a more satisfactory work.

FORMWORK

Shuttering, centering and falsework are synonymous terms in common use. The cost of formwork is estimated as between 1/3 to 1/4 of the total cost of a concrete structure. It is made of wood or steel, each of which has its own advantages.

The formwork must be strong and rigidly braced which will not bulge or sag when concrete is placed and it must be constructed which can be easily dismantled without causing damage to the concrete or disturbing the remainder parts. If mechanical vibrators are to be used then bolts must be used in place of wire ties or nails and members strengthened to resist additional stress. The joints of the formwork must be sufficiently tight to prevent loss of liquid from the concrete. As far as possible, formwork should be standardized which will permit the re-use of sections without alteration.

Half-seasoned soft-wood is considered best for formwork. Very dry timber will absorb moisture from wet concrete and swell while green timber will shrink. Hardwood is expensive, heavy and difficult to work and nail. Where appearance of the finished concrete is of no importance, clean sawn timber may be used, and if a smooth finished face is desired, wrought boarding should be specified. Where a fair finish face is desired under a roof slab, the upper surface of the supports may be covered with oiled soft building board or other water repellant packing material ; oiled paper is not suitable.

For laying roof slabs, kachha centerings should not be permitted where vibrators are used otherwise the bottom of the slab will be honeycombed. Centering should be either of steel or of strong wood with joints made watertight. Steel forms have many advantages over timber forms : They can be easily and rapidly assembled and dismantled, are non-absorbent, there is no shrinkage or distortion due to change in moisture content. Steel forms may be of 16 gauge (1.6 mm thick) or, perferably, 14 gauge (2.0 mm thick) thickness, stiffened with angles etc. Steel forms can be used up to about 50 times whereas timber formwork cannot normally be reused more than 4 or 5 times.

The surface of timber shuttering that would come in contact with concrete shall be well wetted and a coat of limewash, raw linseed oil or soap-water may be given inside the formwork, to prevent adhesion of concrete to the formwork. Soap solution can be prepared by dissolving yellow soap in water to get the consistency of paint. Steel shuttering must be well oiled. Light greasing is sometimes adopted. Oiling shall not be done immediately before the concrete is to be poured in.

Suitable *camber* shall be provided in the formwork for horizontal member to counteract the effects of deflection. The cambers for beams and slabs shall be 4 mm per metre (1 in 250), and for cantilevers, the camber at free end shall be 1/50 th of the projected length.

Loads for design of formwork : (a) weight of the wet concrete which is taken at 2300 kg/cu. m ; and (b) live load due to men working, impact effect of ramming or vibrating, which may be taken at 400 kg/sq.m. Allowable bending stress (flexural tensile stress) in soft timbers may be taken 80 kg/sq. cm. Posts over 2.4 metres in height should be braced both-ways at centres. Spans of beam bottoms should be supported at 1 to 1.2 metres according to depth of the beam.

Sizes of Timber for Formwork

Members	Size—mm
Flat sheetings for slab bottoms, column and beam sides	25 to 50
Beam bottoms	50
Vertical posts	75 × 100 to 150 × 150
Ballies, dia.	not less than 100 at mid-length and 80 at thin end.
Joists and ledgers supporting sheetings of slabs	50 × 100 to 75 × 200
Studs and wailings supporting vertical wall sheetings	50 × 100 to 150 × 150
Column yokes —horizontal cross pieces supporting vertical sheeting	50 × 100 to 100 × 100

All dimension are in mm. The sizes of formwork stated above are applicable for spans up to 5 m and height up to 4 m.

Roughly, in slabs, thickness of shuttering should be one-third the depth of the slab.

Compacting Concrete

The purpose of compaction is to expel voids and air bubbles in the concrete mass entrapped during mixing. Full compaction is essential, incomplete compaction will not achieve this aim. It has been shown earlier how the strength of a concrete is reduced by the presence of air-voids. Air-voids are not necessarily large and obvious but may consists of many small holes. Concrete will develop its full strength only if it is throughly compacted. Thorough compaction can only be achieved by the correct proportioning of the mix ingredients and water in relation to the method of compaction to be adopted (whether hand compaction or vibrations compaction).

Over-compaction is also bad as it will cause segregation. In all cases compaction should cease when cement paste (scum) starts to appear on the upper surface of the concrete ; all scum or laitance formed should be removed.

Compaction may be carried out by hand or by mechanical vibrations.

Hand compaction is done either by rodding, tamping, hammering or ramming on the outside of the formwork. Rodding consists of inserting a bar (may be a piece of reinforcing rod) vertically into the concrete an moving it up and down until the concrete is throughly worked into place. Rodding is used only for thin vertical sections for awkward corners, or to work the material around reinforcement. Special care should be taken to see that concrete is worked well into all corners, cavaties and around reinforcement. Care is also required while ramming around reinforcing bars to prevent their distortion.

Tamping is the best method for concrete slabs. It is done with a small wooden mallet or a tamping rule which will also serve for finishing of the concrete. Ramming is used for heavy masses of plain concrete. The rammer should have a face at least 15 cm square. Cement concrete is not rammed heavily but is thumped slowly, especially the thin sections. For compacting dry concrete the surface is rammed with a heavy flat bottomed rammer until a thin film of mortar or paste appears at the surface showing that the voids of the aggregate have been filled. The concrete should be placed in layers of such thickness as will enable proper consolidation to be done, and it shall not be dropped from such a height as to cause segregation.

The steel reinforcement should be properly braced, supported or otherwise held in position so that the placing of the concrete will not change it. All bars protruding from piers, columns, beams, slab, etc., to which other bars and concrete are to be added later, should be protected with a coat of thin neat cement grout if the bars are not likely to be incorporated into the succeeding mass of concrete within the following 10 days.

Trowelling the surface will improve water-tightness of dry mixes but will have little effect on wet mixes. Too much trowelling while the concrete is still plastic should be avoided as the cement and other fine material will be brought to the surface and dusting and cracking will result. Only sufficient trowelling should be given to provide an even surface. When the concrete has taken its initial set it may be trowelled again to produce a hard, dense, smooth surface. Sprinkling dry cement on newly laid concrete to take up surface water should be discouraged. Such cement forms a rich layer of fine material on the surface and may result in the formation of cracks and dust.

Compaction by Mechanical Vibrations

Mechanical campaction by vibrations permits the use of less water and less cement than is required with hand compaction (lesser water gives higher strength). Mechanical compaction gives a saving of about 15 per cent of cement, to produce a concrete of given strength. Concretes which are so wet and plastic as can be compacted by hand (125 mm or more slump) should not be vibrated as it will result in segregation. But vibrations can be used to make a harsh, stiff mix, with a slump of 40 mm or less, easily workable. (A harsh stiff mix contains less sand and less water.) The concrete should only be just wet enough to ball in the hand when squeezed. If a wet concrete mix is vibrated, the heavier particles sink to the bottom and liquid comes on the top, concrete should be as

dry as is practicable without segregation. Over-vibration causes segregation and vibration should be stopped when scum appears on the surface.

With rich mixes like 1:1:2 or 1:1.5:3 with coarse sand, use a mechanical vibrator very sparingly and very carefully to avoid segregation. Hand tamping and rodding will give better results.

There are various forms of vibrating equipment to meet the needs of different types of work. Vibrators are classified into four groups : (i) Internal ; (ii) External ; (iii) Table ; and (iv) Surface. Internal or immersion type vibrators are of rigid or flexible type and are the most efficient for general work and should be used whenever conditions permit. Immersion type vibrators are also used for thick floor slabs or for massive work where they can be moved about by men standing on the concrete or on a platform over the concrete.

The flexible handled type are suitable for general building works and the rigid type for massive structures. Vibrating heads are available in many sizes, those of small diameter are used for closely reinforced work and thin sections, and those of large diameter for mass concrete and heavy sections with open reinforcement. The vibrator should be immersed in concrete slowly and withdrawn more slowly otherwise a gap may be left in a stiff concrete.

Vibrators may be used at not more than 60 cm apart horizontally to cause the flow in the concrete. It has been found from tests that vibration of partially hardened concrete does not reduce its strength and that vibration is even beneficial.

External vibrators are clamped to the forms and are useful for awkward corners or very thin sections where an internal vibrator connot be used. As cosiderable energy is absorbed in the formwork itself in transmitting the vibratory action to the concrete, they are not much used.

Table vibrators are used for pre-cast units, their use is generally restricted to factory manufactured articles.

Surface vibrators are used for road work or for mass concrete. Small vibrators are also attached to wooden tamping rules.

Vibrators are driven by either petrol, compressed air, or electric machines. The capacity of a vibrator varies from 4 to 30 cu. metres of concrete handled per hour, according to its size. The minimum frequency of vibrations for effective compaction is about 3,500 impulses per minute ; the higher the frequency of vibrations the shorter the time required for full compaction. **see page 8/187**

Surfaces Finishes

To obtain an even surface on walls after the form-work has been removed a cement wash can be brushed into the surface in two coats. The cement wash may consists of equal parts of cement and fine sand made into the consistency of a stiff oil paint. The area to be coated should be throughly wetted before the wash is brushed on. This coat should be rubbed in with a wooden float. A better appearance is obtained if the wash is rubbed in with a carborundum stone. The finished surface should be sprinkled with water two or three times a day for three days, for if the wash dries before it has attained its set it may dust off.

Rubbed finish. As soon as the forms have been removed the surface should be throughly wetted and then rubbed with No. 20 carborundum stone using plenty of water. The rubbing will remove board marks. Rubbed surface should be washed clean and small voids filled with cement mortar 1:2. The first rub should be applied while the concrete is still green, preferably 24 hours after placement.

A plain smooth surface may be obtained by the use of planed boarding of good quality with tight fitting and preferably with tongued and grooved joints or by lining the formwork with thin sheet metal or plywood. Where necessary, joints may be filled with plaster or clay.

Exposed concrete surfaces which are to be plastered should be roughened with wire brushes and hacked out closely with chisels after the formwork is struck off.

Cement wash. The simplest form of concrete paint is a cement wash which is a slurry made up of simply cement and water (about 14 kg/100 sq. m) and applied with a white-wash brush. This does not stick on for long though it may be improved by the addition of fine sand. The following methods will, however, improve the adhering qualities of a cement wash :—

(*i*) Cement and slaked lime mixed into equal proportions.

(*ii*) I part cement, I part sand, and 5 per cent of hydrated lime by weight of cement.

(*iii*) 1 kg of calcium chloride in about 30 liters of mixing water.

Calcium chloride absorbs moisture from the air and keeps the cement damp.

(*iv*) 1 kg of common salt per bag of cement in the mixing water will also give some improvement.

There are a variety of proprietory concrete (or cement) paints available in various range of colours, and which are also waterproof, and give excellent results. Before applying any form of cement wash or paint, the concrete surface must be clean and free from any oil, grease, dust or loose material and be well wetted. Curing is essential to ensure complete hydration of the cement base. Manufacturer's instructions should be followed.

Architectural Surface Finishes

Very pleasing effects may be obtained by the use of linings of wall-board, plywood or hessian cloth. Mouldings and decorative features may be produced by fixing appropriate insets in the shuttering. Where hessian is used it should be of a very coarse texture as otherwise the weave pattern will not be reproduced. It should be well stretched and turned over the edges of shuttering.

Exposed aggregate surface finish can be obtained by removing the outer film of cement from the concrete to expose the aggregate by one of the following methods. Best results are obtained by the use of rounded aggregate of size 10 mm to 24 mm.

In the case of roof slabs the top surface should be finished even and smooth with a wooden trowel before the concrete begins to set. The surface of RC slab on which cement concrete or mosaic floor is to be laid should be roughened with brushes while the concrete is green. This should be done carefully without disturbing the concrete.

Surface defects of minor nature can be rectified as follows :

Shallow patches are first treated with a coat thin grout composed of 1:1 cement and sand then filled with mortar similar to that used in the concrete. The mortar is placed in layers not more than 10 mm thick and each layer is given a scratch finish to secure bond with the succeeding layer.

The exposed surface of RC work should be plastered with cement mortar 1:3 of thickness not exceeding 6 mm to give a smooth and even surface.

The surface which is to receive plaster, or where it is to be joined with masony, should be properly roughened immediately after the shuttering is removed, taking care to remove laitance completely without disturbing the concrete. The roughening can be done by hacking.

To avoid patching appearing darker than the surrounding concrete, white cement in small quantity can be added to the mortar. Many earths and loamy sands will cause discolouration of concrete.

(*i*) Where the shuttering can be removed within 48 hours of concreting, the surface may be scrubbed with stiff fibre or wire brushes and water. Care must be taken that the concrete is sufficiently hard to prevent the dislodging of whole pieces of aggregate.

(*ii*) Applying copious washings of a solution of 1 part hydrochloric acid to 6 parts of water (or a stronger solution if this is not effective). The work is then scrubbed with stiff brushes and well washed down with clean water. Precautions must be taken against the acid solution. This treatment should not be applied to concrete made with a limestone aggregate. (See also under "Concrete Roads".)

(*iii*) Retarding agents are available which are liquid or jelly-like compounds which have the effect of retarding the setting of cement with which they are in contact. The formwork is coated with a retarding agent, and while the bulk of concrete hardens normally, the outer skin which is in contact with the coated formwork does not. On removal of the forms, the outer cement film can be brushed off and aggregate exposed.

A wide variety of finishes may be obtained on concrete surfaces with cement mortar which may consist of either 1 part cement and 3 parts sand, or 1 part cement, 1.5 parts lime and 6 parts sand. After application it can be figured by the use of combs, trowels or special tools to produce ornamental finishes. If mortar is forcibly dabbed on the concrete with a hand brush it with produce a rough pleasing appearance.

A wet plastic mix of 3 part cement, 1 part lime, 6 parts sand, and 4 parts of 6 mm to 12 mm shingle or crushed stone which is thrown on to the wall by means of a scoop or plaster's trowel. This is called *rough cast*.

A 10 mm coat of 1 part cement, 1 part lime and 5 parts sand upon which, while it is still soft, is thrown 6 mm to 12 mm selected shingle which has been well washed. This is called *pebble-dash*.

Water-proofing Concrete

(See also under "Damp-Proofing and Water-Proofing in section 7).

A badly made concrete cannot be made impermeable to the penetration of water by any admixtures. The first requisite therefore, is to obtain a dense concrete with well proportioned non-porous aggregates, and with low water/cement ratio (0.54 or less) so as to have a minimum of air voids. Normally all concretes are porous, and these pores have to be reduced to make the concrete, as for as possible, impermeable to water.

Methods of making good concrete as detailed in the preceding paragraphs must invariably be followed. It is often beneficial to use a slightly excessive proportion of fines. A small increase in cement content over that used for ordinary concrete is also advantageous as with more of cement less of water is required for the same workability. The following methods can be used for further water-proofing :—

(*a*) Concrete and mansonry surfaces can be made water-proof by giving three alternate coats of alum and soap solutions. 10 grams of alum is dissolved in one litre of hot water, and 50 grams soap is dissolved in one litre of hot water. The hot alum solution is applied first and worked in with a stiff brush immediately followed by hot soap solution. The solutions are applied with an interval of about 24 hours between alternate coats.

Recent experiments have indicated that a cement plaster (even 1:6) can be made water-proof by mixing the cement mortar in a 1 per cent soap solution instead or ordinary water. "Sunlight" soap was used in the experiment.

Soap solutions act as lubricants and also form insoluble fillers by reaction with cement and may be applied while the concrete is still green.

Walls can be effectively treated against moisture penetration by these methods.

(*b*) Addition of fully slaked (hydrated) white lime in the following proportions will also make the concrete water-proof. Lime paste occupies about twice the bulk of paste made with equal weight of cement and is therefore very efficient in void filling ; but the mixture must be of dense concrete.

1:2:4 concrete — 10% of the weight of dry cement

1:2.5:5 „ —15% „ „ „

The addition of hydrated lime increases workability but it is nevertheless an adulterant and where strength is a primary consideration the use of higher cement content should be preferred for increasing workability and achieving impermeability. (Some of the experiments have shown that addition of a small quantity of hydrated lime slightly increases the strength of a concrete, but there are conflicting views about this point.) Increase of workability permits a slight reduction in water content, which in turn reduces permeability.

Make concrete rich to have at least 15 to 20 per cent excess cement over sand and 20 per cent excess mortar over coarse aggregate. A 1:1.5:3 mix with water/cement ratio of about 0.40 will make the concrete practically water-proof.

Rendering with mortar consisting of cement, hydrated lime and sand in the proportions 1:3:10, 12 mm thick will also make concrete waterproof. (See also Section 12 under "Mortars".)

(*c*) The form-work should be removed as soon as practicable and the concrete surface rubbed smooth and washed. A mixture of cement and sand of proportions 1:1.5 with some waterproofing compound should be worked into the pores and over the whole surface in such a manner that no more material is left on the concrete face than is necessary to fill the pores completely.

(*d*) Concrete floors may be treated during concreting operation with dry cement sprinkled over the surface and worked in with a steel trowel on the initial set of the concrete. (See under "Concrete Floors".)

(*e*) As regards surface application of a water-proofer the method depends on the quality of the conerete. If pores are very small, silt or fine clay may fill them. Boiled linseed oil, paraffin, or varnish, can be brushed on the surface when the concrete has been well cured and has dried. Two or three coats may be applied allowing each to dry before the next application. A coat or two of bitumen or coal tar makes the surface impermeable to water ; concrete must be perfectly dry and dust free ; a thin priming coat (of bituminous material) should be given to ensure bond. 50 to 60 litres per 10 sq. m of bitumen are requried.

(*f*) Bituminous Mastics are generally laid on horizontal surfaces and also towelled on vertical surfaces. They are used either hot or cold.

(An asphalt lining has the disadvantage that it insulates the floor from the beneficial effects of saturation, thus increasing the tendency to develop cracks, the asphalt is liable to fail ultimately over such cracks and construction joints—remarks for water reservoir floors.)

(*g*) Proprietary compounds such as Pudlo, Medusa, Ceresit or Ironite are used according to the manufacturers' instructions, not exceeding 1.5 kg/50 kg (one bag) of cement.

Inert materials used include finely divided chalk, Fuller's earth and talc, all of which consist of very fine particles. They assist in making the concrete dense, especially if the aggregate is deficient in fines.

(*h*) Treatment with silicate of soda. (Described under "Treatment of Porous Concrete").

(*i*) One kg of washing soda dissolved in 30 litres of mixing water will make a cement mortar water-proof.

Concreting in Hot Weather. In tropical countries, such as India, air temperatures may rise up to 40-50 deg. C. during summer months. Such temperatures combined with high wind velocity and/or low humidity enhance the rate of evaporation of the water, increase the rate of hydration of cement, which factors reduce the setting time of the concrete considerably and this is apt to effect adversely the properties of hardened concrete. Concrete placed at high temperatures is prone to excessive contraction due to rapid evaporation of water ; there is increased drying shrinkage, which tend to develop cracks soon after placing, even before hardening is complete.

Even though *water* forms about 1/6 to 1/8 the total weight of concrete, the role it plays, especially during summer months, is of much significance—the quantity of water used and its temperature. The quantity of mixing water in a unit of concrete is decided primarily by the workability desired, besides the role of the maximum size of aggregate. To obtain higher workability more water is required. Due to accelerated hydration of cement and loss of water by evaporation, the concrete is prone to lose workability and to maintain the required consistency water content will have to be increased. But merely increasing the water content without increasing the cement content will result in decreased strength and durability. Drying shrinkage of concrete is directly proportional to water content. Rapid loss of workability makes compaction difficult. As such, mixing and placing of concrete at high temperatures may not develop its full strength. As for as possible cold water should be used for concreting in hot weather. Rapid hardening cement is not suitable for hot weather concreting.

When high temperatures prevail during the summer, it is best to abandon concrete work. No concreting shall be done at temperatures exceeding 38 deg. C. Aggregates shall be stacked under shade and sprinkled with cold water immediately before use. As far as possible all mixing should be done under shade, and mixing time should be the minimum that will ensure quality. Every effort should be made to lessen the time elapsing between mixing and placing. The mixer may be painted white on the outer side to lessen absorption of heat from sun and air.

Protect the concrete from exposure to sun and drying hot winds with wet gunny bags or hessian cloth as early as possible. In the case of flat surfaces, it may be convenient to use a gabled framework supported across the side-forms and covered with hessain, tarpaulins, or strawmats (which can be made travelling stands). Water curing should be started as soon as possible by ponding.

Setting-time Retarding Admixtures. Addition of 0.05 per cent of sugar, by weight of cement, has been found to retard the setting time of concrete by about two hours in outdoor hot weather condition of 44 deg. C. temperatures. For the same water-cement ratio, workability and strength are also improved. For equal strength, the sugar admixed concrete would enable a leaner mix to be used, resulting in about four per cent saving in cement. Overdosage of sugar is harmful. Sugar is added to the mixing water. (IRC : 61-1976).

Concreting in Cold Water and Cold Weather. The rate of hardening and setting of concrete is very much retarded when the temperature falls below 21 deg. C. (70 deg. F.). At about 10 deg. C. (50 deg. F.) the action of setting slows down to about one-half of what it is at 21 deg. C. In addition to the slowing down or stopping of hydration and hardening there is also danger of disintegration of unset concrete due to the disruptive effect set up by the expansion of the mixing water as it freezes.

During cold weather concreting shall be abandoned when the temperature falls below 4.5 deg. C. (40 deg. F.). (Use immersion thermometer inserted in concrete near forms or surface for recording temperature.) If however, the work is of urgency or importance that must be continued, it can be carried out with complete success provided certain remedial measures and precautions are taken.

The most convenient method is to heat the mixing water and, for very low temperatures to heat the aggregate as well. Heat the mixing water to 66 deg. C. (150 deg. F.). On no account shall the hot water be added to cement alone. Aggregates may be heated to 21 deg. C. Mixer drum may also be warmed. Cement must not be heated.

Temperatures of fresh concrete exceeding 21 deg. C. (70 deg. F.) are undesirable due to the higher water requirement, and likelihood of cracking when the concrete contracts on cooling, and relatively low strength. For most constructions, the right temperature of concrete at placement is somewhat below 21 deg. C. Concrete with a low water/cement ratio is less liable to damage by frost, and for good resistance to frost it is considered that the average water/cement ratio should not exceed 6.60 (30 litres per 50 kg of cement).

Fresh concrete must not be allowed to freeze. If concrete is frozen, setting and hardening ceases. Avoid the use of forzen aggregate. The concrete placed shall be protected against frost by suitable covering. Concrete damaged by frost shall be removed and work redone.

Provide layers of straw of other insulating material on the freshly laid concrete surface as soon as the concrete is hard enough to sustain it without detriment. An insulating layer for covering concrete may be composed of waterproof paper overlaid with a layer of straw and finally with second layer of waterproof paper.

In frosty or other adverse weather conditions, use of colloidal concrete may be considered.

An increase of cement content of the mix by about 20 to 25 per cent use of rapid hardening cement with an admixture of calcium chloride or, high alumina cement are usually recommended. With high alumina cement concreting can proceed without any further precautions provided that the temperature is not at freezing point or below and the materials are not frozen.

"Accelerators" are used in cold weather to increase the rate of hardening and thereby reduce the likelihood of failure. They accelerate the hydration of the cement and increase the rate of evolution of heat; thus the temperature of the concrete is raised and the freezing point of the mixing water is lowered, enabling concreting to be carried out when the air temperature is near or slightly below freezing point.

As far as practicable, the use of accelerators or admixtures should be avoided.

Calcium chloride is the most commonly used material for accelerating hardening of the concrete and is perhaps the most reliable, which may be used up to 2 per cent max : (prefer 1.5 per cent) of the weight of cement. Quantity in excess of this proportion is harmful.

In no circumstances should this chemical be added to high alumina cement. Calcium chloride is a white deliquescent and hygroscopic salt commercially available at low cost in flakes or granular form and delivered in moisture proof bags or airtight drums, and should be stored in a dry place. It is disolved in the mixing water to which cement is added afterwards. Calcium chloride should not be placed in contact with water or mixed dry with aggregate. Calcium chloride shall not be used where reinforcement is provided in the concrete.

The use of calcium chloride approximately halves the setting time; the concrete must be placed in position and finished with the minimum of delay because of the rapid setting.

Common salt (*sodium chloride*) lowers the freezing point of water. For temperatures below 0 deg. C. dissolve 1 kg of salt in 170 liters of water and which may be slightly more for lower temperatures. Larger percentages of salt appear to weaken the concrete. Salt over 5 per cent by weight of cement is injurious as it not only affects the strength of the concrete but may also cause rusting of the reinforcement and efflorescence. Much dependence should not be placed on salt for prevention of freezing. Salt should be thoroughly dissolved or the results will not be satisfactory.

Timber formwork is a valuable insulating agent and should be used in cold weather. The concrete must be kept warm and protected from frost after it has been placed and until it has hardened. Heat losses from concrete are greater in the first few hours, therefore, protective methods must be applied as soon as possible after placing. Suitable methods of protection are wrapping or covering the concrete with dry hessian or backing, straw blankets, old paper cement bags, tarpaulins or a 15 cm layer of dry straw.

If timber formwork is used it should be left in position as long as possible. Since the rate of hardening of concrete will be slower in cold weather, formwork will have to be left in position somewhat longer.

Before any formwork is stripped it must be made certain that concrete has hardened sufficiently. Precaution must be taken against coverings being displaced by wind.

Reinforcement that is left protruding from the concrete constitutes a danger spot since it offers an easy path for heat losses. It should therefore be wrapped.

Steam is sometimes used for heating the concrete, which is introduced between the coverings and the concrete.

CURING OF CONCRETE

What is Curing. When water is added to cement chemical reactions take place (hydration of cement) which result in the setting and hardening of cement. Mixing water is usually sufficient for the initial hydration of cement. If however, there is insufficient water in the concrete during the setting period for the complete hydration of the cement, the concrete does not develop its full strength.

The strength of concrete increases with age provided moisture is present though at a much lower rate after a certain period. It is generally believed that cement keeps on hardening for at least one year. When concrete is laid, its water content is rapidly lost if sufficient precautions are not taken, by evaporation occasioned by the action of sun and wind and the heat generated during setting cement. The prevention of such loss of water from concrete during its early life is known as curing. If water in the concrete is allowed to evaporate, the cement will not have sufficient water for hydration and will not set properly. The cement sets very rapidly in the intial stages, if it is not kept damp, the shrinkage effects will be very marked and there will be loss in strength of the concrete. After about a fortnight the curing can be stopped as changes afterwards are very slow and the shrinkage that occurs is not very harmful.

The strength of a concrete is only 50 per cent if it is not damp cured, of the strength if it is damp cured for 14 days, whilst the strength is 25 per cent more if continued for about a month.

Comparative Strength of Ordinary (Portland) *Cement Concrete at Various Ages* :—(Approx.)

3 days	40%	3 months	115%
7 days	65%	6 months	120%
28 days	100%	1 year	130%

Strength of Rapid-Hardening Cement Concrete :—(1:2:4 mix.)

Age		
	24 hours	140 kg/sq. cm
	2 days	200 ,,
	3 days	260 ,,
	7 days	370 ,,
	28 days	480 ,,
	12 months	530 ,,

Strength of High Alumina Cement Concrete :—(1:2:4 mix.)

Age	7 hours	260 kg/sq. cm
	12 hours	490 ,,
	24 hours	520 ,,
	3 days	530 ,,

Climatic conditions and the type of cement used will affect the curing practice. Concrete gains strength more slowly at low temperatures than at high; higher temperatures are more favourable for curing. During hot weather or high winds there is rapid drying of water and it needs more care. Rapid drying will also produce shrinkage cracks.

Curing should be begun as soon as possible after concrete is placed and when initial set has occured and before it has hardened, and should be continued for a minimum period of 7 to 12 days when normal (Portland) cement is used, 4 to 7 days when rapid hardening cement is used, and should be kept thoroughly wet for 24 hours when high alumina cement is used. The cold weather reduces the rate of hardening even if the concrete does not actually freeze; the minimum curing period should be increased to 14 days with normal (Portland) cement and 7 days with rapid hardening cement. After placing the concrete should be protected during the first stages of hardening from harmful effects of sunshine, drying by winds and cold, and from running water on its surface.

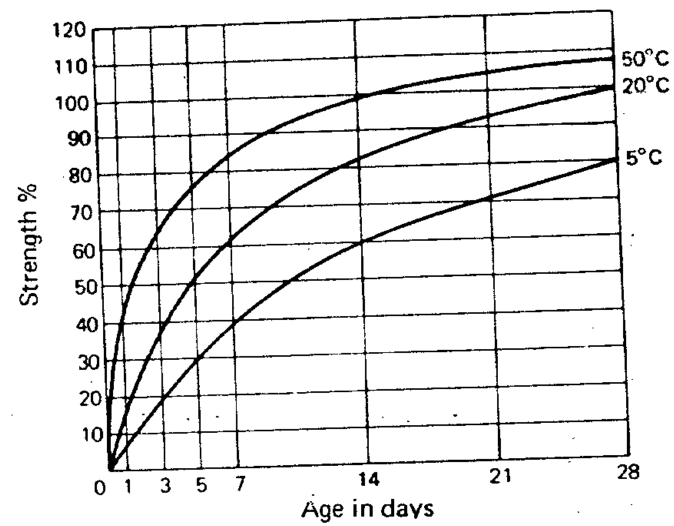
Methods of Curing Concrete. There are two methods of curing, one in which the moisture is supplied to the concrete, and the other in which concrete surfaces are sealed to prevent loss of moisture from freshly laid concrete. Evaporation of water from the concrete is largely prevented by the forms which should therefore be left in place as long as possible, and in hot weather should be periodically sprayed with water. Where timber formwork is used care has to be taken that water is not lost by absorption from the concrete itself. This can be done by either painting the inside of the forms with oil or by well wetting them before concreting. Subgrades of roads and foundations should also be well wetted.

Soon after concrete is placed it should be protected from direct rays of the sun by covering with hanging curtains on vertical surfaces or by screens mounted on light frames on flat surfaces. After the concrete has begun to harden, i.e., 1 to 2 hours after laying, it shall be protected from quick drying with moist gunny bags, coir matting, bundles of straw, 50 to 80 mm layer of saw-dust, sand or earth, or other moist retaining materials and keeping such materials constantly wet by watering. (If a gunny bag is put on wet concrete, it will stick to its surface.) After 24 hours of laying of concrete, the surface shall be cured by flooding with water with minimum 25 mm depth, or by covering with wet absorbent material. Small earthen mounds can also be made in squares over the entire area and water is kept in them all the time to a depth of about 30 to 50 mm, called *Ponding*. A flat concrete surface is considered ready for ponding when a small boy can walk over it without leaving impressions. Curing can also be done by spraying the surface periodically with water.

Vertical surfaces may be covered with hanging curtains of tarpaulin or wetted hessian. Columns and small members are best cured by wrapping round them wet sacks or hessian and sprinkling water continuously from a perforated pipe placed on the top horizontal position, or keeping pots full of water on their tops with a pin-hole at the bottom. On vertical surfaces see that the wet fabric is kept in contact with concrete. Don't forget to sprinkle water on the undersides of slabs or sides of beams. When curing is done in a tank (of precast members) water should be changed frequently.

There are a number of proprietary membrane curing compounds in the market, or bituminous emulsions, which are sprayed on the concrete surface. Since the purpose of these coatings is to prevent evaporation, it is essential that they be applied as soon as the concrete is finished. It is preferable to apply two coats, the second being applied immediately after the first. Although these coatings prevent evaporation of water during the early life of the concrete, their full curing action is not very certain and it is advisable to carry out normal moist curing as a safeguard as soon as concrete is sufficiently hard. Bituminous emulsion is not recommended for road work as it makes the road slippery for a time and also produces an unsightly appearance as it becomes worn away in patches. When bituminous or any other black compound is used it should, in hot weather, be sprayed with whitewash to reduce heat absorption.

Curing time can be reduced by the use of calcium chloride as has been explained earlier under "Cold Weather Concreting" as it has the



Ratio: $\frac{\text{Strength of concrete cured at different temperatures}}{\text{Strength of concrete cured at 20°C}}$

Water/cement ratio: 0·5.

properties of absorbing moisture from the air. Concrete can be cured under steam within 24 hours or with hot water, but these methods are not for common use.

Curing with sea water should be deprecated as far as possible, especially the reinforced works. If this water has to be used, alternate wetting and drying must be avoided to prevent crystallization of salts on evaporation as crystallization is very harmful to green concrete.

Concrete in Sea Water and Protection of Marine Structures

Sea water is regarded as an active attacking agent of concrete structures unless special precautions are taken. Two types of action appear to take place, one involving the cement and the other the reinforcement. The region attacked is the portion of the structures above mean tide which is subjected to alternate wetting and drying. Portions of the structures continually submerged have been found to remain in perfect condition after many years of service. It is well-known that a mixture of air and sea water rapidly corrodes steel. In the region of alternate wetting and drying, steel reinforcement is apt to be affected unless the surrounding concrete is highly impervious. Since the oxides formed by corrosion occupy 2.2 times the volume of the original metal reinforcement, the process of corroding expands and bursts the concrete member along the line of the reinforcement. Therefore, it is important to provide 65 mm to 75 mm (and 100 mm at corners) or more of covering of concrete.

A rich and well graded mix with low water/cement ratio to ensure dense and water-tight concrete should be used. The mix should be at least of 1:2:4 in the case of plain concrete and of 1:1.5:3 in the case of reinforced concrete. Slag, broken brick, soft limestone, soft sandstone, or other porous or weak aggregates should not be used. Well made pre-cast members have given satisfactory results. Therefore, as far as possible preference should be given to pre-cast members, unreinforced, well cured and hardened, without sharp, corners, and having trowel-smooth finished surfaces free from crazing, cracks or other defects ; plastering should be avoided. Formation of hair cracks is very dangerous. In some localities where excessive scour from sand or ice takes place wood sheathing has been found to offer satisfactory protection. Where unusually severe conditions or abrasion are anticipated, such parts of the work can be protected by bituminous coatings or hard stone facings bedded with bitumen.

The sulphate-resisting cements and high alumina cements have a greater resistance to sea water than ordinary Portland cement. Blast-furnace slag cement is considered highly resistant to attack by sea water and has been used at places abroad.

Special care has to be given to construction joints ; no construction joints should be allowed within 60 cm below low-water level or of the upper and lower planes of wave action. Care should also be taken to protect the reinforcements from exposure to salty atmosphere during storage and fabrication.

Resistance to attack can be increased by the addition of *surkhi* or puzzolana (ground fine as cement) about 10 to 25 per cent of cement (cement will be proportionately less) in a concrete. "Trass" is used in England : 1 part trass, 2 parts cement and 2.5 or 3 parts of well graded aggregate.

Use of Fly-Ash in Cement Concrete Mixes

Experiments carried out at Basin Bridge Construction Works have shown that the recommended minimum design strengths for M 150 (1:2:4) and M 200 (1:1.5:3) grade concretes can be achieved when 20 per cent of cement is replaced by 27.5 per cent of fly-ash by weight. This requires modification in the proportions of fine and coarse aggregates and in the water-cement ratio of the mix. Fine aggregates is reduced by 7.5 per cent of weight of cement and coarse aggregate is increased by this amount.

Development of strength of fly-ash concrete at early ages is relatively slow but its strength at 28 days is the same as that of a mix without fly-ash. As such early removal of shuttering may prove dangerous where fly-ash (or pozzulana) has beem used in the concrete.
(Also see pages 8/185 and 12/7, 15)

When pre-cast units, such as concrete piles, are to be used, these should be stored in air for as long as possible, since by so doing, their resistance to chemical attack is greatly increased. The impregnation of concrete piles with hot asphalt under low pressure has been tried in America and resistance to sea water has been found to be considerably increased.

Sewers and Drains of Concrete

Sewer acids are harmful to concrete. Lining of vitreous clay tiles or stoneware pipes may be immune from attack of sewer acids and jointing materials can be of acid-resisting cement. Bituminous paints or emulsions applied over concrete or mortar joints bacome emulsified and disappear. Coal-tar products or bitumen-rubber emulsions applied in thick coats appear to offer more resistance. High alumina cement—either for a jointing material or for concrete, has proved more resistant than Portland cement but it is not immune from attack and deterioration occurs though at a slower rate.

Concrete in Alkaline Soils

Ground waters with concentrations of alkaline salts (soluble sulphates) are harmful to Portland cement concrete. Concrete sewer pipes laid in sulphate-bearing grounds have been completely disintegrated. Where structures are only partially immersed or are in contact with alkali soils or waters on one side only, evaporation may cause serious concenstrations of sulphate salts with subsequent deterioration even where the original sulphate content of the soil or water is not high. Increased resistance to attack by sulphates can be obtained by using a fully compacted concrete (1:1.5:3 mix.) having a low water/cement ratio and low permeability. This does not provide complete immunity and the use of "sulphate-resisting cements" or high-alumina cement is recommended where the structure is directly exposed to waters of moderate alkali concentrations.

The presence of sulpate salts in clay or soil may be indicated by the presence of colourless crystals or formation of a white scum or efflorescence on the surface of the excavated clay as it dries. The absence of the above indications cannot be taken as proof that sulphate salts are not present. A chemical laboratoy examination should be conducted for doubtful cases.

Kerosene oil 10 per cent by weight of cement if used in cement mortar or concrete is known to retard greatly the disintegration of concrete due to alkalis. Protection of concrete coverings over reinforcement should be increased to 50 mm, and for foundations and footings, a covering of 100 mm may be given. Construction joints should also be reduced to a minimum. Additional protection may be obtained by the use of a chemically resistant stone facing, or a layer of plaster of Paris covered with suitable fabric such as jute, thoroughly impregnated with tar.

Petrol, oil and horse droppings have no effect on hardened concrete. Strong acids and alkalis cause disintegration of concrete.

Causes of Deterioration of Concrete in Structures

Corrosion of Reinforcement. Penetration of moisture through porous concrete causes rusting of reinforcement. A frequent cause being segregation at corners, edges, and faces or at constructions joints. A 0.25 mm gap is necessary before the reinforcement steel will be corroded by moisture.

Excessive fine sands lead to general weakness in a concrete on account of the high water cement ratio which their use entails. A coarse sand gives a stronger concrete than fine sand for the former permits cement to fill in the interstices between the sand particles and thus bind them together. In order that the cement may exercise the maximum binding action, the sand should be coarse enough for cement and water to get through its pores and surround each particle of sand. On the other hand, absence of sufficient fine sand below 300-micron (No. 52 BS) mesh sieve tends to give harsh-working mixes prone to segregation and to local defects.

Frost also has a destructive effect on a weak and porous concrete. The severest conditions for frost action arise when concrete has more than one face exposed to the weather and is in such a position that it remains wet for long periods. Alternate wetting and drying has also an adverse effect on the concrete.

When reinforcing rods corrode they in turn swell up in volume and throw off the covering concrete and become absolutely bare. The rusting goes on slowly through the porous concrete. A covering of cement plaster about 13 to 20 mm thick not weaker than 1:2.5, applied on all doubtful faces will give good protection to the rods. In the case of reinforced brickwork roof slabs, the rod must not touch the bricks which are porous, but should have a covering of 20 mm thick cement mortar.

Extracts from report of an experts committee on the premature collapse of certain important RC structures in Bombay. (Also see page 8/154)

"Although the normal life of RC structures is at least 80 to 100 years, several multi-storeyed buildings, bridges and other structures built less than 20 years ago in Bombay are showing signs of distress.

Leaking terraces, tanks and bath-rooms in several newly-built buildings, including multi-storeyed ones are proof of a poorly made concrete mix.

"Honeycombed" concrete because of segregation of stones from cement in the concrete, and air voids due to excess water in the mix, is a familiar sight in buildings under construction. If a column or beam of concrete has honeycombs or voids in it, moisture, chemicals and salt (in a coastal area like Bombay) penetrate the structure and corrode the steel rods.

Since rust occupies thrice the original steel volume, the concrete gradually splits and weakens the whole structure. Five per cent voids in concrete could reduce the strength of a structure by 30 per cent and ten per cent of voids by 60 per cent.

Since honeycombed concrete is generally superficially plastered over, it is not noticed till the rod has corroded and the concrete has split, causing cracks in the plaster.

If minor cracks by honeycombing and voids not repaired propertly, the disintegration would continue like cancer and result in a collapse.

It is not only supervision during concrete mixing that is important, but also the quality of the building materials used. Most Indian cements are not of adequate quality for the high strength concrete required in high-rise buildings or concrete roads surfacing. The locally available sand is screened through 20 mm square mesh instead of the specified five mm square mesh, thereby allowing 25 per cent gravel in the sand. This results in "undersanded harsh concrete" which causes honeycombing. A structure could not be strong if the right proportion of cement, sand, stones and water did not go into the mix and if it was not compacted properly with the help of a vibrator. Failure of RC structures after the centering has been removed, are due not only to the use of substandard quality of cement, but even with good quality cement there is a great loss of strength due to voids (honeycombing) in the concrete.

Voids in concrete are caused mainly due to wrong blending of stone and sand which has less fine particles. This causes lack of cohesion in the freshly made concrete and therefore it segregates during mixing, transporting and placing, and results in honeycombed concrete.

Another cause of honeycombed concrete is the tendency for the workers not to use vibrators to fully compact the concrete. Besides causing loss of strength in concrete and leakage in roofs, bothrooms and tanks, voids permit the entry of moisture which eventually corrodes the reinforcing steel bars and the resulting rust further splits the concrete and the structure disintegrates within a few years."

More Water added

However, at most sites, the mixer driver and other untrained workers who were unaware of the consequences of improper implementation of the basic principles. Often, more than the required amount of water is added to the mixture to make it more compactible, weakening the meterial in the process. This kind of a problem is said to be unheard of in advanced countries where concrete of different strengths was mixed at the site. In India, architects and consulting engineers have abandoned their responsibility of site supervision and are concentrating only on sophisticated designs on paper.

Most architects and consulting engineers are also alleged to be designing slender, economic multistoreyed structures to increase the saleable carpet area. These slender structures are difficult without modern sophisticated vibrators as done abroad. Columns and beams which are too slender for concreting, are seen to develop cracks before the completion of the building.

No amount of supervision would help unless the workers themselves were trained in concrete mixing.

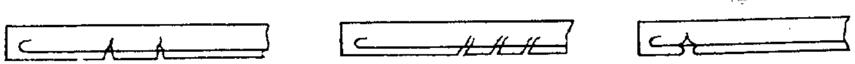
Inadequate provision for testing the strength of concrete during and after construction is also a major reason for the premature collapse of building.

Unless non-destructive testing methods were made obligatory, testing of concrete would never be sound. The only major concrete test so far has been the cube test in which freshly made concrete is moulded in the shape of a cube and subjected to heavy pressure. It was, however not a foolproof method because the quality of concrete in the control cube need not necessarily be the same as in the structure. Moreover, interested parties might manipulate results of cubes tests because the cubes were generally broken later. Although non-destructive tests did not give high precision data about the strength of concrete, it gave a proper idea about the quality of material in a built structure.

Among the simplest of such tests was the rebound hammer test in which the concrete was hit by a spring loaded hammer. The extent of rebound suggested the strength of the meterial.

In the ultrasonic concrete tester, sound waves were sent from one end of the concrete to the other and their velocity measured. The higher the velocity, the better the concrete."

Cracking of Concrete : Causes and Remedial Measures



Tension Cracks **Shear Cracks** **Bond Cracks**

Cracks in concrete are caused due to several factors :

(*i*) One of the chief causes of cracks in concrete is the cooling and contraction (which follows the evolution of heat due to hydration of cement due to setting of concrete. Volume change and stresses due to shrinkage are independent of any external load or stress applied.

(*ii*) Cracks may develop in a smaller section attached to a larger section due to differential expansion and contraction—a joint should be provided at the change of section. There is more possibility of cracking of fixed (restrained) members than those which are free to expand and contract, as simply supported beams. Reduce restraints on free expansion or contraction of the structure.

(*iii*) Repeated expansion and contraction or alternate wetting and drying, which may result in gradual disintegration of poor concrete.

(*iv*) Rapid drying due to hot weather and high winds or absorption of water from the concrete by the wooden forms. The formwork on which fresh concrete is placed must be damped, or it should otherwise be waterproof so that it does not absorb water from the concrete. Ensure adequate moist curing.

(*v*) Formwork should be of adequate strength to bear the pressure of the wet concrete without swelling, spreading or any movement.

(*vi*) Concentration of tensile reinforcements at square openings or re-entrant angles (as in corners of door and window openings) cause cracks. These can be avoided by suitably placed reinforcements having adequate coverings. Sufficient thickness of concrete should be given at the points where bars are bent up and anchored.

(*vii*) Minute cracks on the tension side of a reinforced concrete member are unavoidable due to the poor tensile strength of concrete as compared to steel and which must crack when the steel reinforcement takes its load. These cracks, however, should be fine enough for moisture penetration to prevent corrosion of the reinforcement.

Hair cracks are partly due to the unequal shrinkage of the surface concrete and the mass behind it. Delayed finishing and final floating of concrete, up to a certain limit, avoids surface cracks. Surface cracks are also caused by surface dressing with a mortar too rich in cement, too much water, insufficient curing, or from over-trowelling. Such hair cracks do not affect the strength of the concrete. One method of avoiding such hair cracks is to remove the surface skin of the concrete by brushing it with a stiff brush soon after setting. Fine superficial cracking is known as "crazing".

Contraction of concrete is more harmful than expansion as it sets up tensile stresses in the structure, particularly those with a large surface area and thus form cracks. Such contraction cracks may be prevented by inserting reinforcement near the surface. Closely spaced reinforcement of small diameter and near the surface is more effective than large diameter bars further apart and further from the surface. Contraction or shrinkage of concrete in hot dry areas is much greater than that in damp and cool climates.

Patching and Repairing of Old Concrete. Patching is necessary when some of the concrete at the surface has disintegrated or become honey-combed or where spalling or crazing has occurred, or there are defective construction joints. In patch repairs, all deteriorated and loose materials should be removed and the surface prepared as explained under "Methods of Jointing New Concrete to the Old" in the pages following, to establish a satisfactary bond between the old and the new concrete. For patches with depths more than 10 cm, anchors or dowels may have to be used consisting of 6 mm bars embedded about 40 mm in sound concrete by drilling holes and grouting. These anchors may be spaced 40 to 60 cm apart. In addition use wire mesh, which may be 75 x 75 mm, tied to the anchors and kept a distence of about 35 mm from the finished top surface and also above the old concrete surface. In case of vertical and overhead surfaces, the filler concrete should be placed in small layers and each layer thoroughly compacted before the next layer is applied. After about 12 hours setting, gently hammer the new concrete to test if there has been a proper bond. Remove any portion which produces a hollow sound.

Repairs to Corroded RC Structures

The basic problem is corrosion of reinforcement. Reinforcement corrosion is indicated by a peculiar cracking of concrete structures. At times, reinforcement is exposed and the extent of corrosion is visible as rust. Often it is local. Externally located concrete members corrode more as compared to the internally located ones. Therefore, for inspection it is necessary to expose the corroded locations to the extent visible, clean the rust with wire brush, tamp lightly with 5 lb (2 27 kg) hammer, and rub the surface with hessain ropes or wire brush. In the market rust removing chemicals and allied solutions are available. In important members, sand blast the surface, but at times only air blast may be sufficient. This cleaned surface, if found heavily corroded, is coated with black ship paint, that is, marine paint. This seals off the steel surface and prevents further corrosion. Provide additional reinforcement required locally by providing suitable links 'hooked' or 'anchored' in columns or beams. For slabs 'U' hooks and expanded metal or wire-mesh *jali* for beams and columns, depending on extent of repairs; provision of *jali* helps in bond.

Repair work in concrete is difficult, as the fresh concrete does not easily stick to the old set concrete. The most effective way of repairing is by *guniting* Cement and sand are mixed in proportion of 1:3 and shot on to the surface of repair by compressed air. Experiments show that gunite of maximum strength and density is obtained with a certain velocity, insufficient velocities producing reduced strength while excess velocities increase the amount of rebound.

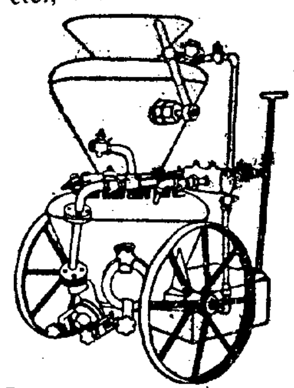
A feel with hand would indicate concrete quality; loose concrete portions falling down with little or no efforts.

Guniting or Shot Concrete (or Shotcrete)

An intimate mixture of cement, sand (or fine aggregate) and water is forced or ejected through a cement-gun and shot into place by means of compressed air. The usual equipment consists of a compressor, spray nozzle and flexible hose pipe. At the end of the hose there is a nozzle to which water under pressure, by a separate connection, is added to form a slurry. Uniformly graded, thoroughly mixed dry materials, with low water/cement ratio, are charged into the gun and shot under pressure by compressed air. Slightly moist sand works better. The usual proportions of cement and fine aggregate are 1:3 or 1:4. The fine aggregate should be well graded up to a maximum size of 10 mm—usual size is 4.75 mm downwards. Hard-stones sand should be used. Just only sufficient water necessary for the hydration of cement is used.

There is usually 20 to 30 per cent "rebound" depending upon the wetness of the mixture. A very wet mixture will not stick. While shooting, a nozzle under normal conditions is held at a distance of about 75 to 90 cm from the working face. The surface to be treated must be thoroughly cleaned of any dirt, grease or loose particles and should be fully wetted. The correct No. of gun should be obtained for the maximnm size of aggregate or sand to be used. Reinforcement, usually of 80 mm sq. mesh, may be incorporated to withstand structural or temperature stress.

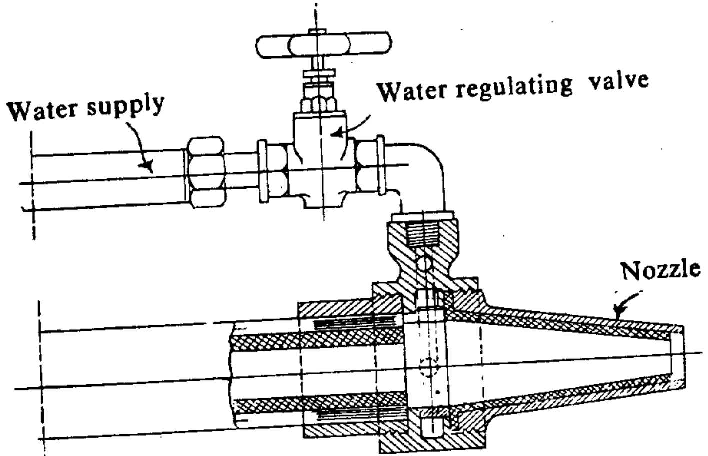
This method is very useful for rehabilitating or reconditoning old concrete, brick or masonry works which have deteriorated either due to climatic condition or inferior work. It is also used for water-proofing exposed concrete surface or for resisting water pressure on pipes, cisterns, etc., where it forms a very impervious layer



The bond between old bad concrete with corroded reinforcement and guniting is not assured very effective always. It is sometimes advisable to replace total concrete. Grouting or pressure grouting has to penetrate to some extent into the porous concrete surface exposed. Effectiveness of their surface is considered to last for about five years only.

Guniting or Shotcreting Machine

For bad concrete and corroded reinforcement, clean all the exposed concrete surface with water (in addition to the reinforcement as explained above). Various chemicals with different trade names are now available in the market which can be applied on the (old) concrete surface for bonding of the old and the gunite. If surface to be concreted is more than 25 mm, expanded metal or wire-mesh can be used.



Cement Gun

Cementation or Pressure Grouting

Cementation is injecting under pressure cement grout into cracks, voids or fissures in structures or ground (road subgrades etc.). This process is useful for repairing structures, consolidating ground and forming water cut-offs, etc. This normally restores stability in a structure which has otherwise become unstable due to cracks or voids. Holes are drilled at selected points and cement grout, which is sufficiently fluid to ensure complete penetration, is pumped in.

Holes may be drilled about 40 mm diameter and about 3 metres horizontal and 1 metre vertical distances apart. Cracks, joints and voids should be tapped by such holes. Holes must not be drilled right through the thickness of the mass under repair. Cracks should be cleaned by forcing water under pressure through the grout holes.

Ground of fairly hard nature but loose texture, *e.g.* certain types of made-up grounds, may be consolidated and have its bearing capacity increased by cementation. Pipes are driven into the ground and the cores within the pipes removed by means of an earth auger. Grout is then pumped into the ground through the pieces and penetrates into and fills the voids in the ground. Below and around dams and deep excavations, water cut-offs are made, by making deep bore-holes, to prevent seepage or ingress of water.

Treatment of Cracks. The strength of reinforced members is in general little affected by the presence of fine cracks of such a width that can be closed by "stopping" and painting. Oil paints, cement paints and water paints (distempers) can be applied. Wider cracks should be filled in with grout consisting of 1 part of cement and 1/2 to 1 part of fine sand. The crack is sealed at the surface with cement paint or grout of stiff consistency applied with a brush, leaving opening at selected points for the liquid grout to be injected into the crack. Should the crack extend through the member it is sealed at the rear face in a similar manner, the

openings spaced along the crack here acting as vents to allow any entrapped air to escape the filling operation. Such sealing prevents further deterioration through penetration of moisture.

Cracks in water retaining structures can be filled with hemp or jute fibers impregnated with bitumen which are pressed into the cracks and then sealed with bitumen.

Bulges and ridges due to forms can be removed by carefully chipping and then rubbing with a grinding stone. Small patches can be filled in with cement mortar similar to that used in the concrete.

Surface Treatments of Concrete

Treatment of Porous Concrete with sodium silicate (waterglass). Surfaces of porous concrete or concrete of poor quality produce harmful dust due to abrading action. Treatment with silicate of soda hardens the surface by forming a glassy substance and increases wear resisting properties. It is the cheapest, simplest, and most effective method. Sodium silicate is a colourless liquid. It is diluted with four times its own volume of water, well stirred, and the solution sprayed over the surface with a watering can and brushed evenly with a soft broom. The solution may be applied in three or more coats, each coat is allowed to dry for 24 hours before the next coat is applied. Scrubbing each coat with water after it had hardened provides a better condition for the application of succeeding coats. If the surface is very porous, or for water retaining structures where greater protection is required, stronger solutions may by used with 1 to 3 (silicate to water) for the second coat and 1 to 2 for the third coat. For average concrete, 25 litres of the diluted solution will be sufficient for about 100 sq. metres. The surface of concrete to be treated must be thoroughly cleaned of any grease or dirt and dried after completion of the curing period before the solution is applied.

In all cases where surface treatment is to be used the concrete must be hard, dense, and water-tight since the destructive effect will be much greater if corrosive liquids are able to penetrate the concrete. The following materials may also be used :—

Boiled linseed oil—Best results are obtained when the oil is applied hot. Two or three coats should be given and each coat must be dry before the next is applied. Raw linseed oil should not be used.

Varnishes—Any varnish can be applied to dry concrete. Two or more coats should be applied.

Bituminous or coal tar paints, tar and pitches—Various proprietary brands are available for cold application like ordinary paint. The concrete must be absolutely dry, clean and dust free. Such paints are usually applied in two coats, a thin priming coat to ensure bond and a thicker finish coat. (See also under "Cement Paints" in Section 12.)

Colloidal Concrete (Cement Grouting)

This method may in some circumstance be used as an alternative to normal concrete construction (especially for pavement). It has the advantage of comparative speed and simplicity, but the finished quality is relatively poor.

The system essentially consists of placing coarse aggregate in position, levelling it off slightly above finished height and filling the interstices with grout. It is a cheap method of concreting "in place" and very simple. Cement and water are first mixed together in the proportion of about 30 litres of water to 50 kg of cement, to which sand is added afterwards to form a colloidal grout. The water content should be the minimum that is required to porduce a grout capable of penetrating all interstices in the placed aggregate. The proportions of cement and sand can vary from 1:1 to 1:4 according to the type of work desired. The grout can be mixed in an ordinary tilting drum mixer. This grout is poured in or pumped into the coarse aggregate already packed in forms until complete penetration is achieved after which the concrete is compacted in the normal way.

The main advantage of this method is that large size of aggregate can be used in their natural from, saving cost of crushing and also expediting the work. The largest size of aggregate can be a little less than the finished thickness of the work. After large stones are packed, voids are filled with smaller stones which should be of sizes not less than 25 mm. The coarse aggrgate should not contain any particles of less than 25 mm gauge since otherwise complete penetration can be obtained only by using a very watery grout producing a weak and porous concrete. It is all hand-packing of aggregates.

Proper grading will minimize the voids and so reduce the consumption of grout. When grouting is to be done for depths of over 90 cm, 50 mm diameter pipes should be embedded at intervals and the grout poured through them. Compaction should be delayed as long as the hardening of the grout will permit. A dispersing agent such as "Cheecol" is generally added. The function of a dispersing agent is the same as that of a "wetting" agent. (See also under "Cement Grouted Macadam" in Section 18). A dispersing agent when mixed with cement grout, it makes the grout flow more freely around the aggregates so that all the voids are fully filled up.

9. JOINTS IN CONCRETE STRUCTURES

There are three types of joints : Construction joints ; Contraction joints and Expansion joints.

Construction joints are those which occur at points where, work having been stopped for any period, concrete already placed has started to harden or has hardened thus necessitating some form of jointing before any fresh concrete is placed. The concrete on either side of the joint, both old and new, should be quite dense. The position of construction joints should be such that the strength of the member is not affected and may be as follows :—

(*a*) **Beams and slabs.** Joints in beams and girders should be ocated at points of minimum shear, that is mid-way between supports, with a vertical plane at right angles to the direction of the main reinforcement. Where a beam intercepts a girder, the joint in the girder should be offset a distance equal to twice the width of the beam. If this cannot be done the joint must in any case be within the middle-third (but not near the support, column or wall) of the beam or slab. Joint can also be made over the centre of the column allowing one-half of the beam to become the bearing surface of the future adjoining beam. Construction joint may be made in the smaller beam at a short distance from the junction of intersecting beams, adequate shear reinforcement being provided at the joint.

Ordinary slabs supported on two sides may be left after finishing any layer of reinforcement, or at the middle of span making the perpendicular and at right angles to the direction of the span. In the case of slabs continuous over beams, concreting can be stopped directly over the centres of the beams making a vertical joint and allowing for the future adjoining slab. In slabs having reinforcement in two directions, concreting may be left somewhere near the middle, *i.e.*, when half the slab from one side is laid, or within the middle-third on either span.

The ribs of L-beams and T-beams should normally be concreted together with the floor slab of which they form a part. Where however, a joint cannot be avoided, the following method can be adopted :—

In the case of T-beams with continuous slabs in which all the shearing action has been provided for in the shape of stirrups, may be left after completing the rib portion provided the stirrups project from the rib almost to the top of the slab, otherwise the concreting may be taken within 3 cm of the uuderside of the slab. The slab should be built over the rib not later than two days after completion of the rib.

In no case shall work be terminated in beams or slabs where shearing action will be great, as for example, near the ends or directly under a concentrated load.

As a general rule the position and arrangement of construction joints should be settled before concreting begins and should be planned so that the days's work ends at a joint required for structural purposes.

Layers of concrete at the joint should be finished against a properly fixed stop-board to ensure a clean vertical face. The stop-boards must be rigidly fixed and slotted to allow for the passage of reinforcement. Sufficient number of continuity bars must be left to tie the beam or slab to the opposite side. Concrete should be well rammed against the stop-board.

Battens may be nailed to formwork to ensure an horizontal line and if desired may be used to form a grooved joint. The width and depth of the rebate should be about 1/3 and 1/4 respectively of the thickness of the member. If thickness of concrete permits, some engineers prefer to provide a key of the tongue and groove type at the construction joints. The width of the groove is about 1/3rd thickness of the concrete. It is sometimes desirable to put in extra loop steel.

(*b*) **Columns** : These should be finished with a level surface 10 to 15 cm below junction with beams, this is particularly important in the case of the column heads in flat slab construction. Two hours should elapse after depositing of concrete in columns or walls before depositing of the concrete in beams, girders or slabs supported thereon to allow for settlement or shrinkage in the column concrete.

Methods of Jointing New Concrete to Old. The surface of the concrete already placed should be prepared in the following manner and joint made :—

(*a*) If the stoppage in the work is of short duration, say a matter of a few hours, then a mortar grout of similar composition to that contained in the new concrete should be applied to the old surface.

(*b*) When the concrete is partially hardend, the surface should be scrubbed with wire or hard bristle brush, removing any laitance, care being taken that pieces of aggregate are not dislodged, moistened with water and cement slurry consisting of 1 cement and 1 sand (of the consistency of cream) is applied with a brush followed by the application of a 15 cm thick layer of cement mortar 1:2.

(*c*) Where the concrete has hardened, similar treatment to the above is given. The concrete is hacked or chiselled and roughened and thoroughly wetted for about an hour before placing the new concrete.

Fresh concrete should be well rammed against old, particular attention being paid to edges and corners.

Cement surfaces can be roughened to improve adhesion by etching with muriatic acid. The acid should be washed off with plenty of water. Method has been explaned in Section 18 under Cement Concrete Roads—"Correcting Slippery Surfaces."

Contraction and Expansion Joints are necessary due to changes in volume of concrete caused by (*i*) Shrinkage due to hydration of cement during setting (hardening and drying) ; (*ii*) Temperture changes, and (*iii*) Changes in moisture content. The reduction of water due to the gradual drying of the concrete results in shrinkage or contraction of the concrete. In the initial stages large stresses are built up in the concrete and these stresses must be relieved by incorporation of joints. Subsequently, when the concrete has hardened, expansion and contraction occurs due to the seasonal variations of temperature and moisture content. Unless a reinforced concrete structure is free to expand and contract, stresses are set up in the structure. The intensity of such stresses depends on the range of temperature and variation in the moisture. Joints are therefore provided in long buildings to relieve the stresses.

Contraction joints are essentially breaks in the structural continuity of the concrete, and are intended to open when the concrete contracts during setting or when the temperature falls below the temperature of laying. Expansion joints permit the concrete to expand and contract. As concrete is very much weaker in tension than in compression, contraction joints normally have to be spaced at closer intervals than expansion joints.

Contraction is more dangerous than expansion because shrinkage leads to tension cracks through which water seeps in. The magnitude of expansion is 1/2 to 1/3rd that of shrinkage or contraction.

The amount of shrinkage during setting of concrete is largely dependent upon the quantities of cement and water in a mix, the greater the cement and water content, the greater the shrinkage. Water content

is particularly important. Shrinkage results from the loss of excess water due to evaporation. Shrinkage per 10 meter length of 1:2:4 concrete, based on water/cement ratio is approximately :—

(*i*)	With water/cement ratio of	0.3	1.5 mm
(*ii*)	—ditto.—	0.5	4.2 mm
(*iii*)	—ditto.—	0.7	8.0 mm

Experimental results point to a very wide range of changes in shrinkage. In a normal concrete structure the amount of shrinkage may be 2 to 4 mm per 10 meter length. Contraction of neat cement is roughly three times as great as an ordinary mortar 1:3 or 1:2:4 concrete.

Concrete generally shrinks more intensely during the initial period of hardening, and during the first year. Under load with continous compression shrinkage is accelerated, with continous tension, on the contrary, it is retarded.

The combined effect of shrinkage and temperature change can cause stresses which may be greater than the tensile strength of the concrete and lead to the formation of cracks. For this reason the surface of large continous areas of plain and reinforced concrete should be sub-divided by means of break-in continuity joints.

Differential nature of variations due to the temperature gradient between the upper and lower surfaces of a slab may cause the slab to warp, thus producing flexural stresses at the ends and particularly at the corners of the slab.

Most buildings do not require expansion joints. For the most part buildings of ordinary size and regular in plan can resist the stresses caused by volume changes without recourse to expansion joints.

If concrete is cast in long lengths or is restrained from free movement and cracking and drying is uncontrolled, then shrinkage cracks will in all probability be concentrated at a few points and so be severe. Warping of the slabs may also occure. Cracking of concrete due to shrinkage may be prevented by (*i*) proper curing, (*ii*) provision of reinforcement, and (*iii*) restricting length of concrete cast in any one operation.

Expansion and Contraction of RC Structures

Concrete and steel both expand with rise in temperature, and the expansion of both these materials is about the same.

Co-efficient of Linear Expansion for 1 deg. C/F per unit length :

Steel	0.0000120/0.0000067
Plain concrete	0.0000110/0.0000060
Reinforced concrete	0.0000117/0.0000065

The co-efficient of linear expansion is the change in length per unit of length for a change of one deg. of temperature.

Total expansion of a structure = co-efficient of linear expansion × length of the structure × change of temperature in deg. If expansion is not allowed to occur, the stress produced=co-efficient of expansion × change of temperature × modulus of elasticity of the structure metal. A structure 30 metres long will expand about 10 mm through a temperature change of 50 deg. F/10 deg. C.

A temperature variation (difference between the max. rise and fall) of 34 deg. C is considered in moderate climates and 50 deg. C in extreme climates.

Shrinkage co-efficient for RC members is taken as 0.0002 per unit length, and this produces a stress of 35 kg/sq.cm.

Spacing of expansion joints is determined in relation to the movement which will occur due to temperature changes. Expansion joints should be provided depending upon the range of temperature of the surface and the season during which the work is carried out, and may be at intervals of 24 metres to 36 metres. If the concrete is laid in summer, the expansion joints may be made at greater spacings than when the concrete is laid in winter. IS:456 recommends for general guidance that structures exceeding 45 metres in length should be divided by one or more expansion joints. (Some engineers recommend that expansion joints must be provided for all concrete structures of 10 metres length and more.)

It is considered that maximum movement of joints located 10 metres apart will not exceed 4 mm under most unfavourable conditions. For summer constructions the width of a joint need not be more than 12 mm and for winter constructions the width may be larger up to 20 mm. Minimum space left for a joint should be 6 mm and maximum 25 mm. It is better to provide larger number of joints than providing wider joints.

In order to be effective, expansion joints should extend entirely through the building, forming independent units. Column footings that may come at expansion joints need not be cut through unless the columns are very short and stiff. Joints should extend through foundation walls otherwise the restraining influence of the wall below grade, which is without a joint, may cause the wall above ro crack in spite of the joint in it. Wall and roof joints must be made continuous over parapets.

Although there is no definite indication that joints should be provided where a building changes direction, as for intance in L, T or U-shaped structures, unless the adjoining parts are quite dissimilar in size, some designers locate joints at such places as a precautionary measure. Joints should be provided at change of secton. There should be no fear in providing joints so long as care is taken in their design and adequate supervision given to their construction.

Expansion joints should always be made as simple as possible without sacrificing effectiveness. Wind and water tightness are essential ; so some form of seal is necessary. Usually a crimped copper or stainless steel strip of 1.5 mm thick metal is used for this purpose. 12 mm diameter holes at 20 cm centres should be punched near the edges of the copper strip to securely anchor it in the concrete. A copper strip is generally preferable to filling the joint with mastic (or bitumen) as it is liable to be extruded

when the joint is closed, making an unsightly appearance. A copper stip can be painted at the surface to match the wall.

The figures shown hereunder illustrate general principles and should be modified according to the situation.

JOINTS FOR RCC WORKS

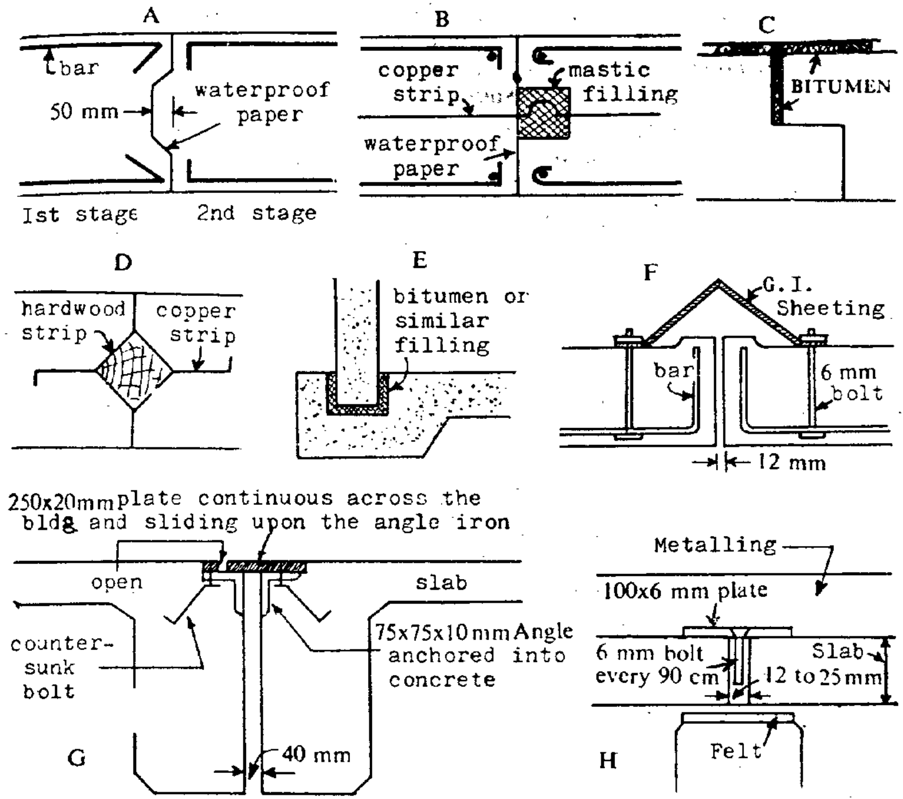


Fig. A : Vertical joints in walls. Where large allowance is desirable for expansion and shrinkage, a space of 6 mm can be left in-between. Where dealing with water leakage, a copper or stainless steel strip, 1.5 mm thick metal can be put in.

Fig. B : Vertical joints in reservior wall.

Fig. C : Joint in roof slab.

Fig. D : Joint for external wall of buildings.

Fig. E : Base of wall of cylindrical tank, where wall and floor are not monolithic.

Fig. F : Roof expansion joint.

Fig. G : Bridge slab without metalling.

Fig. H : Bridge slab with metalling over it.

In reinforced concrete walls, expansion joints may be at 20 to 25 metre intervals. In unreinforced walls, joints at intervals of about 4.5 metres may be necessary for walls exposed to weather and 9 metres for walls protected from weather. If the walls are panel walls between columns at not more than 9-metre centers, no joints are necessary.

In ordinary roof slabs of RC protected by layers of mud or other insulating materials is unframed construction, joints may be at 20 to 30 metres intervals. But for unprotected thin roof slabs, expansion joints should be provided at not more than 15 metres intervals. Where slabs are built over walls or pillars, there should be no joint between the wall and pillars and the slab should be free to move and expand.

For tank walls and similar works all joints should be formed with a strip of rust proof sheet metal about 1.5 mm thick inserted half-way into the concrete so as to form a continuous water barrier.

Suitable materials for joint-fillers are bitumen impregnated fibre board, soft-wood free from knots, cork, etc.

Self sealing synthetic rubber packing for expansion joints are now available in the market.

Propping for Centerings. The props may consist of ballies or brick masonry pillars laid in mud mortar. Ballies should have 100 mm minimum diameter, measured at mid-length and 80 mm at thin end and should be placed at 1 to 1.2 metre spacings. These should rest squarely on wooden sole plates of 40 mm thickness and minimum bearing area of 10 cm square laid either on ground or on 40×40 cm brick masonry pillars in mud mortar of height not exceeding 40 cm. Double wedges should be provided between the sole plate and the wooden prop so as to facilitate tightening and easing of the formwork without jarring the concrete. In case brick masonry pillars of adequate section are used instead of props, wooden sole plates should be provided at the top of pillars and double wedges inserted between the sole plate and the bottom of the formwork.

Removal of Centerings

All formwork should be stripped or removed with care so as not to damage the concrete due to shock or vibrations. Wedges, etc. should be slackened gradually and forms eased carefully in order to prevent the load being transferred suddenly to the concrete. A rough guide is to strike the concrete with a light hammer—a hard metallic sound indicates that the concrete is hardened sufficiently for the forms to be removed. In a slab and beam construction, sides of the beam should be stripped first, then the undersides of the slab, and lastly the undersides of the beam. In normal circumstances (generally where temperatures are above 20 deg. C) and where ordinary cement has been used, forms may be struck after expiry of the followings periods :

(*a*) Structures not carrying loads such as—

Vertical sides of foundations, colums, beams and walls — 48 hours

(*b*) Undersides of slabs up to 4.5 metres span — 7 days

(*c*) Undersides of slabs above 4.5 metres span and undersides of beams and arches up to 6 metres span — 14 days

(*d*) Undersides of beams and arches over 6 metres span up to 9 metres span — 21 days

(*e*) Undersides of beams and arches over 9 metres span — 28 days

(While removing centerings from under slabs and beams some props have to be put or left under at intervals.)

With high temperatures the period may be less and with low temperatures more as cold weather will slow down the rate of hardening.

Removal of forms gives an opportunity to wet the undersides of slabs and beams.

With rapid hardening cement 3/7 of the above time with minimum of 24 hours should be taken.

For structures carrying loads, side timbers should not be removed before 10 days and supporting timbers 28 days. No loads should be allowed on the works before 28 days.

10. SPECIAL TYPES OF CONCRETES

Prestressed Concrete

A concrete (reinforced in the special method described below with cold drawn steel wires of high tensile strength) which is subjected to compression (at the time of manufacture) in those parts which under load will be subjected to tensile forces so that the concrete will be nowhere in a state of tension under the working load. Prestressing induces compression on the lower or tension side of a beam and when the design load is applied, tension is produced on the lower side which neutralizes the compression already set up by prestressing. The aim in prestressing thus is to completely neutralize the stresses due to the design load. It has been stated earlier that it is not economical to use high tensile steel in ordinary reinforced concrete works.

The prestress is set up in a concrete beam by stretching several wires of high tensile strength in the concrete. There are two methods in general use : The wires are stretched before the concrete is cast (called "pre-tensioning") and the stretching force subsequently released. After the concrete has set, the wires are cut and prestress is created in the concrete due to the prevention of steel from contracting to its original length. The prestress in the concrete is maintained and the stress is transferred to the concrete through the bond between wires and concrete. The steel is prevented from returning to its original condition by the concrete and this induces compression in the concrete. A good bond between the steel and concrete is ensured because several wires of small section (as compared with the common mild steel rods) are used, and due to the slight lateral expansion of the wires in the surrounding concrete on release of the wires from stretching.

In the second method of prestressing, which is called "post-tensioning" the wires are stretched after the concrete has hardened ; which are eithe encased in pipes or sheathes, or holes are left in the concrete througl which wires are subsequently threaded. The wires in this method have to be held stretched permanently by mechanical means, *i.e.*, anchors. There is no bond between wires and concrete. The reinforcement in the post-tensioning method consists of a few large or several small cables made of high tensile steel wires laid in one or more rings round a core. Pretensioned bonded type is more suitable for small structures particularly of the precast variety and the post-tension bondless type for heavy structural members of long lengths such as in bridges.

For the same design load the weight of concrete in a prestressed concrete is about 50 per cent less than in ordinary reinforced concrete and the weight of the high tensile steel used is about 50 to 75 per cent less than the quantity of mild steel. The working stresses adopted for prestressed concrete are much higher than the common reinforced concrete, thus permitting the use of much smaller and lighter section for the same load.

Lightweight Concrete

Concretes weighing less than 1600 kg/cu. m are generally termed as lightweight concrete. Such concretes are usually produced by using lightweight aggregates such as breeze or clinker, foamed slag, pumice. These concretes have good insulating qualities but are porous and absorbtive and corrosive to steel, and are not used for reinforced works. Blocks made with this concrete are used for non-load bearing partition walls and penels, flooring and for fixing bricks for joinery.

No-Fines Concrete

No-fines concrete is made with coarse aggregate and cement only without using fine agggregate (sand). The coarse aggregate is graded to pass 20 mm sieve with not more than 5 per cent passing through a 10 mm sieve. Any normal aggregate may be used though a natural gravel is best. Mix proportions are 1:8 by volume, or 50 kg of cement to 0.28 cu. m of aggregate ; and the water/cement ratio should be the minimum necessary which will be about 0.4. An excess of water content will tend to cause cement paste to run off aggregate surfaces and to fill the interstices.

This type of concrete is light in weight, about 1600 kg/cu.m ; thermal conductivity is less than normal concrete ; crushing strength at 28 days is 35 to 55 kg/sq. cm ; it offers very little resistance to the passage of water and is comparatively weak in strength. Can be used for walling plastered both sides, but is not suitable for foundations or works below ground for its cellular structure.

Mixing and laying of no-fines concrete is carried out in the same manner as for normal concrete but the aggregate should be wetted before mixing. This concrete will not segregate but neither will it flow into position in the moulds or forms. Rodding for compaction is therefore necessary but should be carried out carefully so as not to destroy the cellular nature of the concrete ; ramming or vibrations for compaction must not be used. Construction joints will perforce be weak ; horizontal

joints only should be used. Cutting and making holes are difficult. Reinforcement (which is not a normal system with this type of concrete) where required (for lintels over small openings) should be bedded in cement mortar.

11. CEMENT CONCRETE FLOORS

In residential quarters concrete floors may be laid in 30 to 50 mm thick layers over 80 to 100 mm thick base of lean cement concrete 1:6:12 or lime concrete, according to the wear and load expected on the floor. The top cement concrete should be laid before the lime concrete has completely set (within 7 days). The surface of the lime concrete must be moistened before laying the cement concrete. The hardest procurable aggregate should be used well graded from 20 mm down 1:2:4 mix., (graded from 3 to 20 mm), the matrix being 1 part of cement to 2.5 parts of aggregate, sand being added as necessary to make a workable mix. Minimum amount of water should be used so that no scum is formed on the surface when the concrete is beaten ; the mix should have a slump of not more than 40 mm. Solid floors may be laid with 1:2:5 of medium aggregate.

The cement concrete, which should not be too dry, should be spread evenly immediately after it has been mixed, using straight edge ; it should be atonce well beaten and consolidated with 2.5 kg wooden rammers. The concrete is to be beaten until the mortar comes to the surface, which should be in less than 15 minutes ; the floor should then be trowelled and finished.

Concrete floors under heavy loads or those exposed to heat may be reinforced with fabric (light reinforcement of 150 × 150 mm mesh) placed 20 mm below the finished surface.

Granolithic finish is used to provide a hard wearing, abrasion resistant and dustless surface to concrete floors, stair treads, etc. and also where smooth polished and rich surface is wanted. Granolithic concrete is composed of cement and specially selected aggregate of hard rock and graded from 10 mm down to 150 micron (No. 100 BS) sieve. Mix proportions are normally 1:2 or 1:3 by volume (all-in grit aggregate) ; or may consist of 2 parts cement, 1 part of fine aggregate and 4 parts of coarse aggregate. Coarse aggregate is graded from 10 mm or 6 mm down to 2.36 mm (No. 7 BS) sieve and fine aggregate from 4.75 mm down. The all-in aggregate specified above may consist of 3 parts of 10 mm screenings and 1 part of 2.36 mm screenings, with sufficient sand added to make a workable mix. Particular care should be taken to sift out all dust or fine material and to use minimum amount of mixing water which will give adequate workability and will permit of satisfactory finishing.

Best results are obtained where granolithic is laid before the base concrete has set. Many specifications require this to be done within 30 minutes after placing of base concrete, and in any case it should be done within two hours. The granolithic layer should be of 12 mm minimum thickness. The base concrete need not be smooth finished. The granolithic should be well tamped into place, screeded and lightly floated to required levels. Finishing, as described later, should be left for at least an hour after laying.

Where the base concrete has hardened, its surface should be roughened by hacking, laitance removed and well saturated with water and cleaned. Immediately before laying granolithic, any excess water should be removed and the surface of the concrete covered with a thin layer of neat cement grout well brushed in. The granolithic should then be laid as described above but in this case minimum thickness must be increased to at least 25 mm and preferably 40 mm. An adequate key between the base concrete and the top finish is essential where the base concrete has set.

Joints in Concrete Floors. Floors (under layers) should be laid in a series of panels not exceeding 4.5 metres in length or 18 sq. metres in area, and which may by reduced in exposed positions. The top layer should be divided into panels not exceeding 2 sq. metres, and the longer dimension of any panel not exceeding 2 metres. In exposed positions the length of any side of a panel shall not be more than 1.25 metres. This will avoid cracking due to contraction during setting.

Thin 1.5 mm jointing strips of iron, teakwood, brass, ebonite or plastic should be introduced between the panels and which should be oiled or white washed to prevent adhesion of concrete. Strips of oiled paper can also be used. The effect of initial shrinkage is reduced to a minimum by constructing the floor panels either in alternate bays or in chequer-board pattern in such a manner that no bay is cast in contact with one already concreted until the initial contraction of the latter has taken place. Where concrete in a panel has set, the new panel may be laid by simply butting against the old, the surface of old panel being given a coat of whitewash.

Mosaic, Terrazo Marble Chips Flooring. Flooring surfaces can be made into a large variety of patterns and colours and are generally cast into situ, over a hard concrete base which must be sound with clean rough surface. Methods commonly adopted for laying the terrazo finish surface:—

(*i*) 40 mm to 20 mm layer of concrete is laid wherein crushed marble chips are used as aggregate.

(*ii*) The top course is made with a mixture of 1 cement and 2 marble chips 3 mm size, laid 6 mm thick.

(*iii*) A layer of 1:3 cement mortar 12 mm thick is laid over the concrete base next day the base has been laid. A terrazo topping 12 mm to 20 mm thick consisting of 1 part cement and 2 to 2.5 parts marble chips 3 mm to 12 mm mixed, is laid and surface rammed to consolidate and finally trowelled light.

(*iv*) A stiff mortar of 1 cement and 2 sand is laid over the base course to a depth of 20 to 25 mm. Small approximately cubes of various colours or pieces of terracotta are pressed into the mortar which may be arranged into various petterns. Or alternatively—small irregularly shaped chips of marble are sprinkled over the floating coat of cement, and pressed into the surface with a hand float, and whole consolidated by rolling.

Coloured floors. The colouring pigment should not be more than 1:3 and not less than 1:12 (colouring pigment to cement). Weight of colouring pigment used in excess of 12 per cent of the weight of coment reduces the strength of the mortar. Colours are better mixed with white cement than ordinary Portland cement. Coloured cements should be used in preference to mixing pigments in the cement as with the latter method it is difficult to produce a uniform colour, and resulting concrete will look patchy. Also sufficient coloured cement should be obtained in one batch to complete a whole job. Coloured floors without marble chips can be made with 50 kg of cement and 8.5 kg of red oxide or ivory black laid 6 mm thick. An iron float must on no account be used in finishing with a coloured floor as it will cause crazing. (Crazing is fine hair cracks produced at the surface.)

For *Skirting* : The underlayer should consist of 12 mm thick cement plaster 1:3. Top layer to be same as for the flooring.

Trowelling Floor Surface. Granolithic floor surfaces are trowelled to a smooth hard finish with a wooden float and a steel trowel. Over trowelling or finishing should be avoided. Two separate trowellings are required, the first being done as soon as surface has hardened sufficiently, between an hour or two after placing and when excess water has disappeared from the surface : As surface hardens trowelling should be repeated at intervals until the required degree of finish is obtained. The final trowelling should be finished before the initial set takes place.

A wooden float has advantage of not exerting much suction bu a steel trowel is necessary to form a hard and smooth metallic surface. Exterior surface which are not to have a very smooth finish should be finished by means of wooden floats and not steel trowels. All high and low spots should be corrected during trowelling with the wooden float.

The practice of sprinkling dry cement on the freshly laid concrete surfaces to absorb excess water is not favoured as it tends to form a skin coat which will later be subject to dusting and crazing, and will peel off. Thick cement slurry with 2.2 kg of neat cement per sq. metre of flooring, mixed with water, is spread over the surface while the concrete is still green. The slurry is spread twice by means of iron floats, once when the slurry is applied and the second time when cement starts setting, and finished floated smooth.

Grinding and Polishing. When concrete surface is 3 to 4 days old, it can be surfaced by hand or with a surface grinding machine. This grinding removes laitance or loose material and produces a smooth finish. The first grinding should be done with a coarse carborundum stone, (or sandstone block) 10 to 13 cm in dia. and 50 to 65 mm thick and fine sand sprinkled over the surface, using water freely. All pores and holes are filled with cement mortar of the same material as the floor surface. Second grinding should be done after a further five days using a finer grained carborundum stone and, patches if any, similarly filled in and third grinding carried out. The floor is washed thoroughly after each grinding and in the final grinding washing should be done with hot water and pure soft soap. Final grinding should generally be done after 10 days of rest.

In the case of grinding with a machine the first cut should not be made till the coloured surfacing layer has been down 14 days.

After the final grinding (or cut) oxalic acid is sometimes dusted over the surface which must be sprinkled with water and rubbed hard with numdah blocks. (This operation may be repeated till the surface has acquired the required gloss.) The following day the floor is wiped with a moist rag and dried with a soft cloth. A hot mixture of turpentine and beeswax (4:1 or 3:1) is then applied to the surface and throughly rubbed in with hand and later again rubbed with clean cotton waste for 4 hours. The rubbing must be continued until the floor ceases to be sticky. Best result is obtained with a minimum of beeswax and a maximum of rubbing.

Oils used on dusting concrete floors have not been very successful.

Floor paints in various colours are available and give a hard water-resisting surface and withstand the action of water. They form a hard film on the top. Cement paints have been described elsewhere.

Cement rendering 12 mm thick 1:3 can be done over a lime concrete surface or over brick floors with open joints for key, (dry bricks laid flat or on edge), which makes a cheap quality of floor.

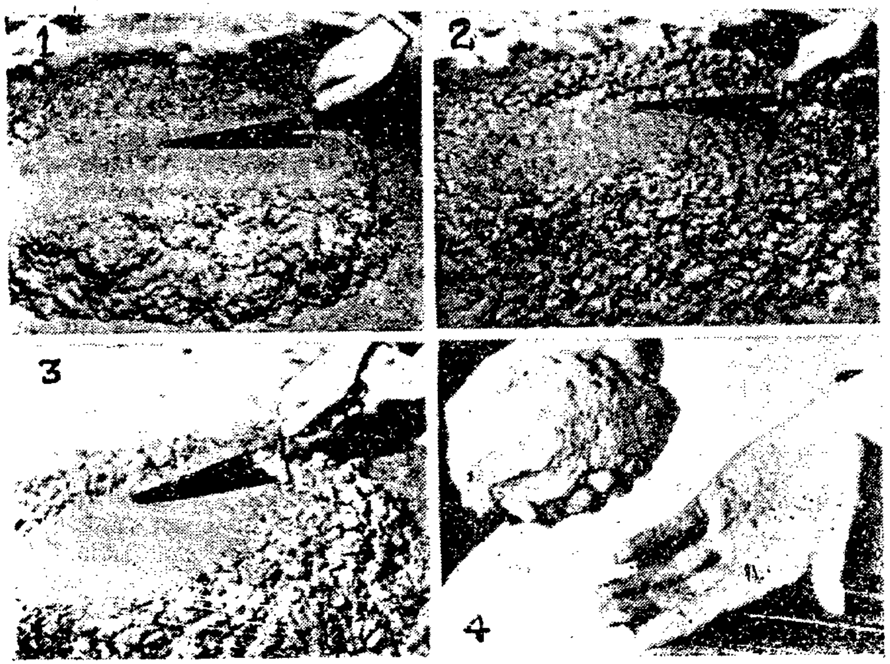


Fig. 1 A concrete mixture with too much cement-sand mortar.

Fig. 2 A concrete mixture without sufficient cement-stand mortar to fill the space between the particles of large aggregate.

Fig. 3 A concrete mixture which contains the correct amount of cement-sand mortar.

Fig. 4 A handful of good mixture should retain its shape when squeezed and become moist on the surface without dripping.

Encasing Rolled Steel Sections

The steel sections to be encased in concrete shall not be painted and shall be wire brushed to remove any loose rust/scales. Ungalvanised wire mesh or light weight expanded metal, having meshes or perforations large enough to permit the free passage of 12.5 mm nominal size aggregate through them shall be wrapped round the section to be encased. The surface and edges of the steel section shall have a concrete cover of not less than 50 mm, and the wrapping shall be so arranged as to pass through the centre of the concrete covering.

12. REINFORCED BRICKWORK

The design of reinforced brickwork structures follows the same general principles of design and analysis as are adopted for the design of similar RC structures.

Roof Slabs

Reinforced brickwork is not very reliable and should not be used for important structures without having ascertained the exact strength of the bricks by laboratory tests although this type of construction has been extensively used for common roofs for its cheapness. Stength of bricks is so variable that we cannot have a universal rule like the concrete work. Strength of bricks is given in Sections 12 and 7. A combination of cement concrete and brick for roof slabs as shown in the illustrations is a better way without much of extra cost. Concrete on the top takes the compression. Best bricks should be selected for the work. Care should be taken to ensure that the bricks do not contain an injurious amount of salt or other deleterious material (efflorescence). The concrete should consist of well graded fine aggregate all passing 4.75 mm sieve. Use the Table for reinforced concrete slabs (page 8/38) and place reinforcement rods in joints. Concrete at the top of the bricks should not be less than 1/3rd the effective depth of the slab.

Renforcement rods of greater diameter than 12 mm should not be used. Use small bars (less than 10 mm) hooked on all ends and placed in every joint for the main reinforcement. Overlapping should be avoided as far as possible and when it has to be done a lap of 45 diameters should be given with proper hooks at the ends and the two rods bound with wire along the lap. All bars should be straight and free from kinks, and should be in one plane only. The thickness of the joint should not be less than 40 mm and not less than three times the diameter of the reinforcement rods. A line of bricks should be laid first in each direction to act as a guide and to ensure that cutting of bricks is avoided as far as possible. If a part-brick has to be introduced this should be done about the middle of the length. When the reinforcement is to be placed only in one direction the bricks should be laid in rows parallel to the reinforcement, their ends being properly jointed with mortar. Negative reinforcement and distribution bars should be provided as for RC slabs. Temperature bars are required at top where concrete is used on the top and where roof is exposed to sun, which may not be less than 0.2 per cent of the area of the cross-section of the concrete.

Co-efficients of expansion and contraction of brickwork is about half that of RC.

The following working stresses may be taken generally for good class bricks in reinforced brickwork : —

Design procedure as given at pages 8/25, 8/27, etc. are to be followed :

Safe compressive stress (in bending) for bricks in slabs when compression is limited to the thickness of one brick only	25 kg/sq.cm
Ditto, for beams or slabs when compression is not limited to one brick	18 kg/sq. cm
Direct compression	14 kg/sq. cm
Ditto, with hoop binders	20 kg/sq. cm
Brickwork in shear or diagonal tension	1.0 kg/sq. cm
Lever arm factor (j)	0.86
Neutral axis ratio (k)	0.42
Modular ratio (m)	40
Moment of Resistance (MR or BM)—Q	4.5

Reinforced brickwork is very weak against diagonal tension and if the diagonal tensile stress exceeds 1 kg/sq. cm, shear reinforcement shall be provided. Even with shear reinforcement the diagonal tension shall not exceed 3 kg /sq. cm. Average bond stress may be taken 6 kg/sq. cm and local bond stress 10 kg/sq. cm.

(If bricks are tested, those giving a breaking stress of 84 kg/sq. cm in compression and 1.4 kg/sq. cm in tension may be accepted.)

Thickness of slabs should not be less than 1/30 of the span.

Another method is to make slabs like "Ribbed Floors" as described earlier. The concrete ribs or flanges can be made 80 mm wide and 300 mm centre to centre to accommodate 9-in. bricks. 40 mm thick concrete is on the top. The ribs are designed as T-beams. Distribution bars 10 mm dia. are provided 300 mm apart on the top slab portion or flanges of the tees, which are at right angles to the main reinforcement in the stem of tees. Negative reinforcement is provided in the top of the stem of the tees extending up to about one-fourth of the span. Stirrups for shear can be provided where required for thick slabs, 150 mm apart and up to a length of one-quarter of span on either side from the supports. Bricks may be laid in 1:6 cement mortar.

DATA FOR REINFORCED BRICKWORK SLABS

Simply Supported at Both Ends

Total depth of slab	Safe Distributed Superimposed Loads in kg/sq. metre (excluding weight of slab and other dead loads) on Various Spans in Metres										Steel area per 30 cm width	Depth below centre of steel
Inches	1.22	1.52	1.83	2.13	2.44	2.74	3.0	3.66	4.27	4.88	sq. mm	mm
3	600	350	200	100	50						70	20
4.5	1875	1125	725	475	325	175	125				120	20
6	3450	2100	1350	950	800	450	300	150			160	25
7.5	5975	3675	2225	1675	1225	875	625	325	125		200	25
9	8550	5300	3550	2500	1800	1350	1000	550	300	100	250	32

Compressive stress in bricks is taken as 25 kg/sq. cm.

Where Grade II steel is used, add 10 per cent extra tensile reinforcement.

Where ribbed—torsteel is used—See notes at pages 8/12, 8/38 etc.

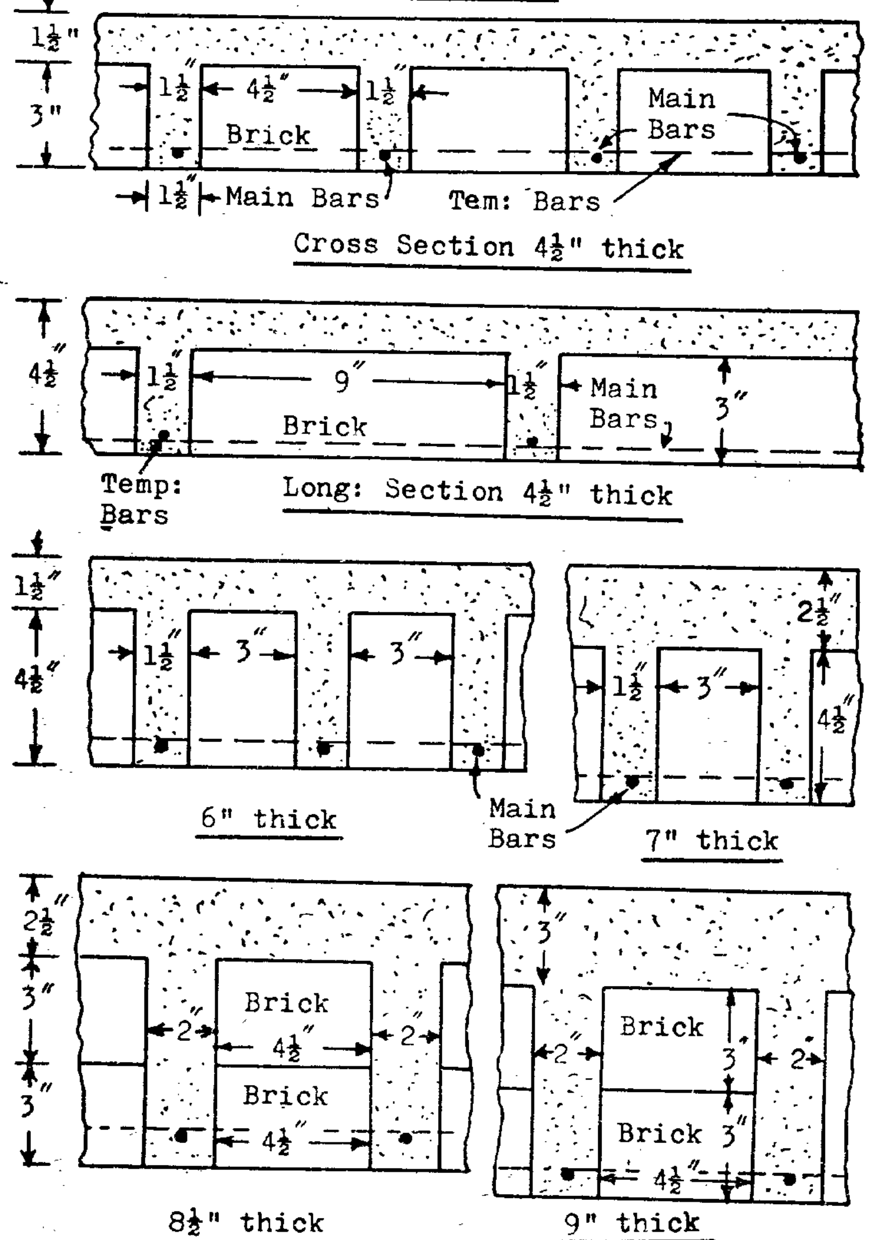
First class brickwork in cement mortar 1:3 to be used.

The table is worked out for simply supported slabs with B.M.=WL/8. For other conditions of support, the span should be multiplied by the following factors, for the same loads :—

WL/10 by 1.12 ; WL/12 by 1.25 ; WL/16 by 1.41.

This table is applicable where the joints are filled with cement mortar or fine cement concrete and rods laid in the joints with no cement concrete at the top of bricks. Fine cement concrete should be preferred to cement mortar in the joints. The cement mortar should be 1:3.

R. B. ROOF SLABS



Where Indian standard modular bricks are used of size 20 cm × 10 cm × 10 cm or 20 cm × 10 cm × 5 cm tiles, the diamension given in the illustration should be adjusted accordingly.

All bricks should be well soaked in water for one hour before use. All joints should be well filled in and reinforcement rods well surrounded by mortar ; the rods should in no place touch the bricks. Similarly there should be a sufficient cover of mortar at the bottom. Mortar filled joints should be "topped up" with additional grout, as and where necessary, after the first pour has been allowed 10 to 15 minutes to settle. Where mortar joints are filled first and top concrete laid afterwards, the mortar in the joints should only reach about 20 mm from the top in order to give good key to the surface concrete. All concreting should be done at the same time as far as possible. The end of the bricks required for filling should be slightly chamfered to form into a dovetail shape for bonding with the concrete.

Reinforced Brick Panelled Walls. (See also Panelled Walls and Partition Walls in Section 7.) Reinforcement may be of wire netting or hoop iron in every second or third joint fully embedded in cement mortar 1:3, and which may consist of two strips (of hoop iron) placed near to either edge of the wall. The reinforcement must be continuous between supports. Panels 4.5 in. (9 cm) thick should not be more than 6 metres long, and 9 in. (19 cm) thick not more than 9 metres long (which may not be more than 5 metres long for 9 cm panels, and for more than 8 metres long for 19 cm panels). Where the panels have no foundations, the bottom course should be reinforced like a reinforced brick slab, and designed for load similar to a lintel. The bricks will all be laid as stretchers, no half bricks or bats being used in 4.5 inches or 9 cm panelled walls.

Where panels are not reinforced, pillars may be provided 6 ft. (1.83 m) apart 13.5 inches thick by 9 inches broad (which may be 1.5 m for 30 cm thick by 20 cm broad) for half brick thick panels.

13. GLOSSARY OF TERMS

Accelerator : An admixture which increases the rate of hardening of concrete (in cold weather) by accelerating the hydration of the cement. Calcium chloride is most commonly used. (See page 8/152.)

Aggregate : Strictly speaking this means all particles of sand, broken stone or gravel, etc., used in making concrete. The term is often loosely used to denote all particles larger than 4.75 mm in which case "coarse aggregate" is more correct.

Bar : The term is applied to simple sections both round or square. (Also defined under "Steel Structures" in Section 21.)

Batch : The quantity of concrete mixed at one time.

Bleeding The discharge of freeing of water from freshly placed concrete. The formation of a thin layer of water on the exposed surface of concrete after compaction.

Bulking : The increase in volume of sand or aggregate caused by the absorption of water.

Colloidal concrete : Concrete in which a wet mixture of cement and sand is injected under pressure in coarse aggregate.

Consistency : Is a general and not a very definite term relating to the state of fluidity of a concrete mix obtained according to the proportion of water in the mix and is usually measured by the slump test.

Controlled concrete : Concrete in which the proportions of cement, aggregate and water are determined by a laboratory test for concrete of a specified strength, and the same are used.

Creep or Plastic flow : Is the gradual and continous yielding of the concrete, which is a permanent deformation, when an applied load is maintained for some time. This deformation or flow becomes appreciable only when the stress in concrete exceeds half the ultimate breaking stress. The strength of the creep is proportional to the stress applied, and the tendency of concrete to creep generally decreases as the strength increases. Creep may enable a structure to support loads much greater than those which consideration of strength and elasticity alone would indicate.

A material is said to possess plastic properties, if, when loaded, it yields and deforms without breaking. The difference between elastic and plastic deformation is that in elastic deformation the material returns to its original shape when the load is removed whereas plastic deformation is permanent. A material is in a plastic state beyond the elastic limit. Steel has a fairly well defined elastic limit and plastic flow in steel takes place only when it is stressed beyond the yield point. But with concrete the elastic and plastic conditions cannot be separated. When the applied load is removed from concrete a certain amount of deformation remains.

Cribbing : Same as forwork or shuttering.

Curing : Keeping the concrete damp after it has been placed in its position to complete the chemical combination of cement and water.

Density of concrete: Is the ratio of the solid volume to the total volume of a specified mass of concrete ; it is the percentage of solid mass in a given volume and may be taken as about 80 per cent for ordinary concrete.

Drop panel : The structural portion of a flat slab above a supporting column, which is thickened in the area surrounding the column capital.

Dumper : A vehicle for transporting materials, so designed as to be capable of discharging its load by forward tipping.

Effective grain size . Is that size in which 90 per cent of the material by weight has greater diameter particles and 10 per cent only has lesser diameter particles.

Final Set : Occurs when the concrete has definitely set but has not yet hardened sufficiently for the formwork to be removed. Final set occurs in about three to four hours with ordinary cement and should not take more than ten hours.

Fineness Modulus : A measure of the mean size of graded aggregate— term used in sieve test. It is a factor and by dividing the total of the percentages of materials retained on specified sieves by 100 when the whole sample of aggregare is tested on each sieve in turn. It gives an idea of the fineness or coarseness of an aggregate, the coarser the aggregate, higher the fineness modulus. The less the fineness modulus, the finer the material. Concrete mixes are sometimes designed with fineness modulus method. There is no fixed fineness modulus for each maximum aggregate but values within a suitable range are likely to give the best results.

Flash set : A setting of cement or concrete which occurs suddenly (while being mixed and placed) and prevents further working of the material.

Floating : Smoothing the surface of newly placed concrete or mortar with a trowel.

Grout : A mixture of cement (with or without sand) and water of a consistency about that of cream.

Hardening : Is the process that indicates the growth in strength of a mortar or concrete and commences at the end of the initial set.

Harsh or stiffmix : A concrete mix that causes difficulty in obtaining a smooth finish or good contact with forms, generally the result of an excess of middle sized particles or a deficiency of fine material to fill the voids in the coarse aggregate. An under-sanded mix, and with less water.

Initial setting time (of cement) : The period elapsing between the time when water is first added to neat cement to form a paste and the time when that paste ceases to be fluid and plastic to a specified degree under the specified conditions of test.

Knocking up : Breaking up and remixing concrete that has begun to set. This should not be allowed.

Laitance : A watery 'scum' which may form at the top of concrete in which too much water has been used or when too much floating or trowelling has been done. If laitance is not removed the upper portion of the concrete will be porous.

Lean mix : A concrete mix having a low cement content.

Mesh : An aperture in a sieve.

Pan mixer : A concrete mixer comprising a horizontal pan or drum in which mixing is carried out by eccentrically placed paddles.

Peripheral speed : The circumferential velocity of the mixing drum of a concrete mixer.

Permeability : Is the property of concrete which permits the percolation or passage of water through it.

Plums : Hard, clean natural stones used in mass concrete or foundations. The plum should not be larger than one-third of the cross-section of the concrete mass.

Pozzolanic materials such as, surkhi and fly-ash which increase the workability of concrete and also reduce the evolution of heat due to hydration of cement, and the hardened concrete is more resistant to chemical attack. This is used in mass concrete work. These materials have no cementing properties of their own. (see pages 12/7 and 8/157).

Punning : Same as ramming.

Quartering : The process of obtaining a reduced quantity as sample from a mass of material by dividing a heap into four roughly equal parts, removing opposite quarters and then repeating the same process with the remainder until the desired quantity is obtained.

Rendering : Adding a thin layer of cement mortar to the surface of concrete or brickwork, etc.

Retarder : An admixture which delays the setting and hardening time of the cement, thus increasing the time during which concrete may be worked. It is used where laying of concrete has to take longer time because of carrying to a distance. Retarders decrease the rate of development of strength and may reduce the ultimate strength by as much as 50 per cent. Gypsum is one of the commonly used retarders and used only in special circumstances. (See page 8/147).

Retempering : Remixing with water of concrete or mortar after it has partly set. Retempering should not be allowed except for patch repairs where retempered mortar or concrete is better than fresh material as it adheres better.

Rich mix : A concrete mix having a high cement content.

Rods : Term used for steel rounds, generally under 12 mm dia.

Rodding : Same as ramming, but done with a bar of iron which can pass between reinforcement.

Screeding : Obtaining a level surface as the correct height by means of a piece of wood or metal having a straight edge.

Segregation : The separating out of particles of different sizes in a concrete mix. The heavier aggregates settle at the bottom. The strength of the concrete is adversely affected.

Setting : Is the chemical action which begins to take place when water is added to cement and which causes the plastic nature of cement to disappear slowly. We have "initial set" and "final set" of cement.

Sieve analysis : The determination of the particle size of a granular material (aggregate) by means of a series of standard sieves.

Slicing : Same as spading.

Slump : The vertical depth through which wet cement concrete subsides from its standard moulded height when tested by the standard method.

Slump Test : The determination of the slump under specified conditions of test.

Slurry : A thin paste of cement and water.

Spading : Similar to rodding, but done with a narrow spade close to the formwork.

Specific Gravity : The ratio of the weight of any given volume of a substance to the weight of an equal volume of pure water.

Striking : Dismantling and removal of formwork or centering.

Sweating : Exudation of moisture on the under surface of a concrete roof slab due to the passage of minute quantities of water through the material.

Tamping : Same as ramming.

Trowelling : Smoothing over the surface of concrete or cement mortar rendering with a flat steel trowel.

Ultimate Load Theory : A method of design in which factor of safety is given to the load and not to the material stress value (and there is considerable saving in steel). Steel must be of standard quality which may not be always there for small jobs where tested steel is not used. (See page 8/5)

Uniformity co-efficient : Size of sieve that will pass 60% by weight/size of sieve that will pass 10% by weight.

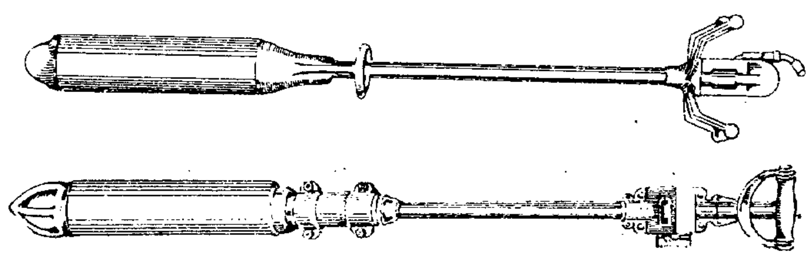
Water voids : Voids in concrete resulting from the excess of mixing water above that required for the hydration of cement. In hardened concrete they may contain air or water or both.

Weigh-batcher : A batching plant in which the quantities of the different materials are measured by weight.

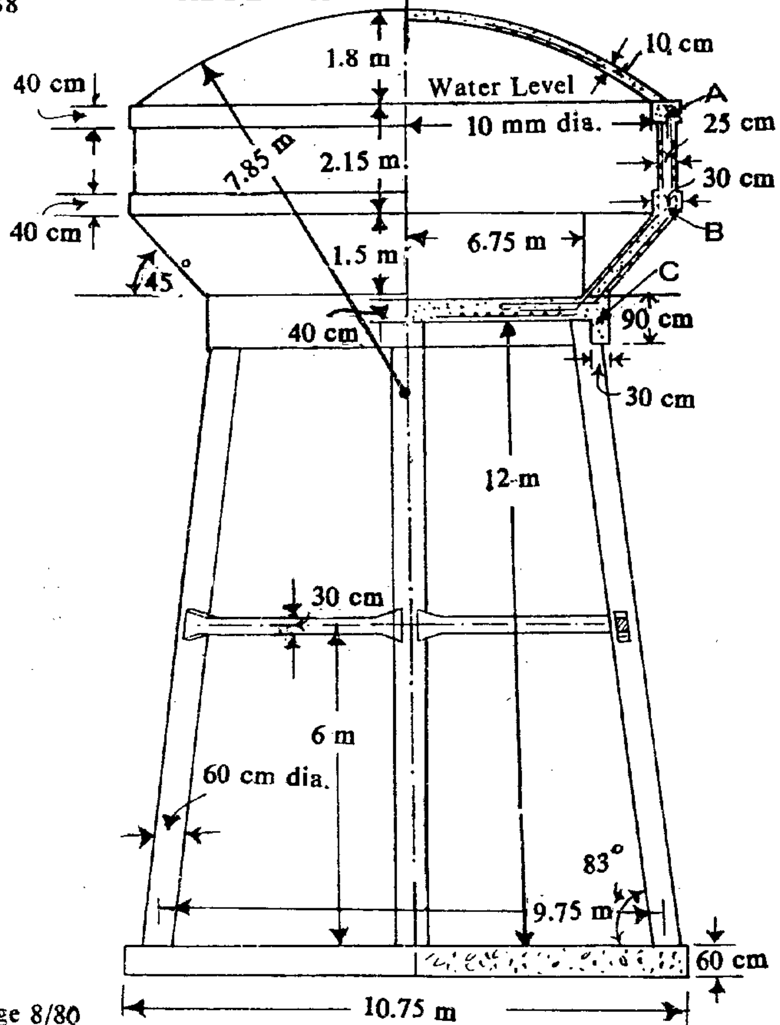
Wetting agent : Is a commercial admixture material the effect of which is to produce an increase in the workability of a given mix without an increase in the water content ; it enables the water to wet the solid particles more readily, and thus makes the flow of the concrete easier. But this is frequently accompanied by a loss in strength. (See "Dispersing agent" at page 8/166).

Winch : A small hoist.

Workability : That property of freshly mixed concrete (or mortar) which determines the ease or difficulty with which it can be manipulated or handled so as to produce full compaction. Is a relative term. A workable mix is one of such consistency and degree of wetness, neither too wet nor too dry, that it can be placed in the forms readily and that with spading or tamping will result in a dense concrete.



Rigid types of internal Vibrators see page 8/145



see page 8/80

Circular Water Tower to Hold 230,000 Litres of Water

Details of Reinforcements for the tower shown in the illustration:

Foundation Raft : 60 cm circular slab, 10.75 m dia. reinforced with 22 mm bars at 10 cm centres both ways, with a circular girder connecting the feet of the columns.

Columns : 6 in number, round, 60 cm in dia., reinforced with 8 bars of 22 mm dia. and 12 mm stirrups at 22 cm centres.

Braces : Provided midway up the column, 30 cm × 30 cm, reinforced with four bars of 22 mm at the top and bottom.

Floor Slab (Circular) : 6.75 m dia., 40 cm thick reinforced with 20 mm bars, 10 cm apart both ways at right angles to each other.

Vertical Portion of Side Wall : 25 cm thick reinforced with 12 mm rods provided vertical at 22 cm centres on both faces

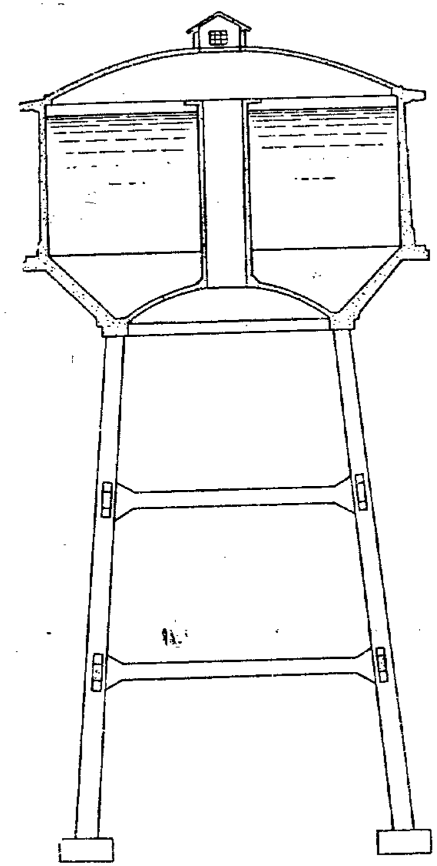
Slant Wall BC : Thickness 25 cm, reinforced with 12 mm bars at 25 cm centres.

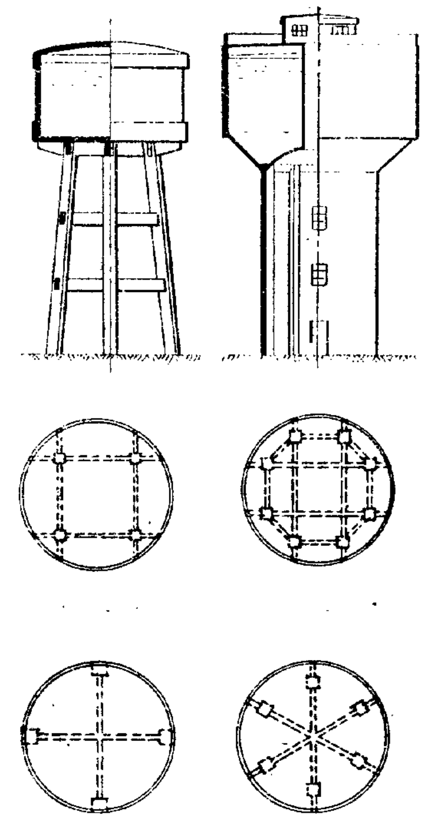
Ring Beam at A : 30 cm × 40 cm reinforced with ten bars of 20 mm dia.

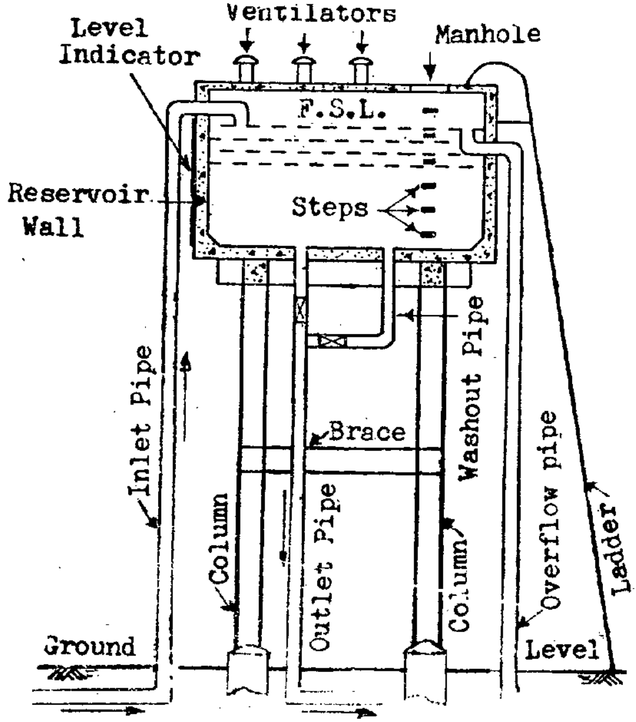
Ring Beam at B : 30 cm × 40 cm, reinforced with fifteen bars of 20 mm dia.

Roof Dome (Spherical) : Shell 10 cm thick, rise 1.8 m and span 10 m. Provide 10 mm bars 22 cm centres radially and circumferentially.

Ring Girder C. ; (Six columns are provided for the girder) 30 cm×90 cm. Provide twelve bars of 16 mm. Use 16 mm stirrups 2 legged 10 cm apart. Provide for each circle four bars of 18 mm and 10 mm spirals at 7.5 cm centres for about 1.5 m on either side of the column.







Elevated Reservoir

Flat Slab Floors

(There is no accurate or rational method of designing flat slabs and various methods are used involving laborious calculations. IS: 456 gives some elaborate methods. An easy to follow procedure is given here.)

This type of construction proves economical for medium span lengths, 5 m to 8 m where the floor loads are relatively large, such as in industrial buildings and warehouses, 700 kg/sq m to 1600 kg/sq. m. Slabs and columns are built monolithically and reinforced in two or more directions and no beams are provided to support the slabs. The columns are so arranged as to form square or nearly square panels. Ratio of the sides forming the panels must not be more than 1.33. The columns are generally provided with enlarged heads. Thickness of the slab over the column heads (or capitals) is increased by about 25 to 50 per cent. This thickened part of the slab is called "Drop" or "Drop Panel". The drops are rectangular in plan and have length in each direction not less than one-third and not more than one-half of the panel length in that direction. The columns are enlarged at the top so as to form a diameter equal to 0.20 to 0.25 of the span length at 4 cm below the drop. Interior columns are generally round and exterior columns generally made square or rectagular in shape.

Thickness of the Flat Slab is controlled by shear and bending moment and which should be the greatest of the following values :

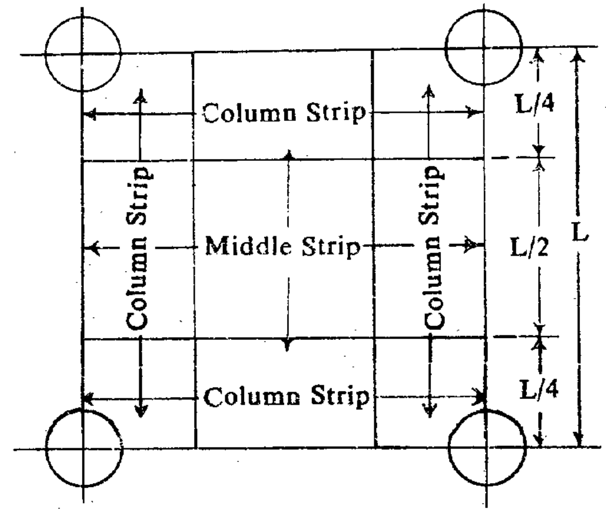
(i) L/40 when drop panels are used; (ii) L/36 when the drop panels are omitted and which may be L/32 for end panels-with a minimum of 12.5 cm. (L refers to the centre line spacing of the columns).

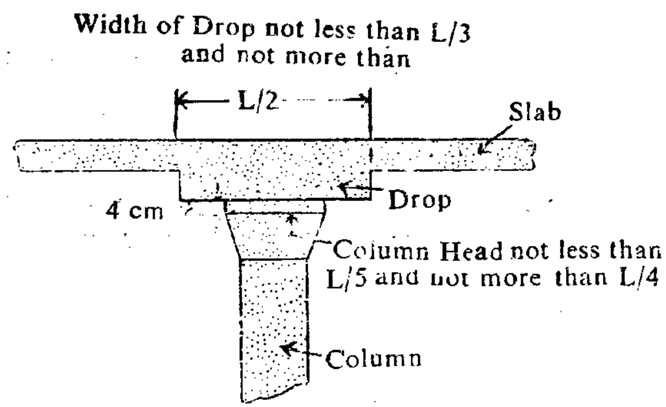
The maximum values of bending moments for a metre width of slab with drops are given below :

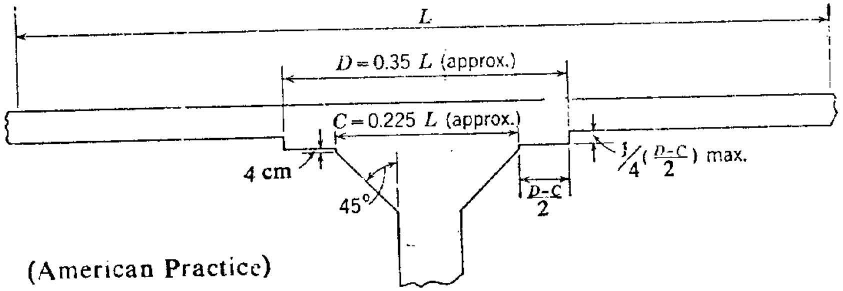
Max : value of Bending Moment per metre width of slab,

	Column strip	Middle strip
Positive B.M.	$WL^2/30$	$WL^2/45$
Negative B.M.	$WL^2/15$	$WL^2/45$

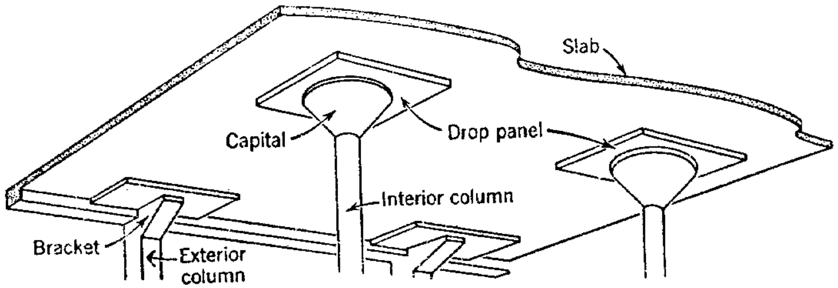
The critical section for shear to be examined is at a distance from column or column head or drop equal to half the overall depth of the slab or drop.







(American Practice)



Unsightly Rust Staining due to iron pyrites in sand

See page 8/119

SECTION 9

TIMBER STRUCTURES

For Timber Structures in Seismic Areas see under 'Miscellaneous Technical Information'—Earthquake Proof Buildings Design.

1. GLOSSARY OF TERMS

Batten : A piece of sawn timber, the dimension of which do not exceed 5 cm either in breadth or in thickness.

Baulk or Balk : A piece of timber whose cross-sectional dimensions exceed 5 cm in one direction and 20 cm in the other direction. (A baulk or balk is obtained by nearly squaring a log.)

Bearer : A beam supported at two or more points and provided for the purpose of carrying other members.

Common Rafter : A roof member which supports roof battens and roof covering, such as boarding, sheeting.

Dovetailing : In carpentry and joinery, the method of fastening boards or other timbers together by letting one piece into another in the expanded tail of a dove.

Fish-Joint : A splice where the pieces are jointed butt end to end and are connected by pieces of wood or iron placed on each side and firmly bolted to the pieces jointed.

Fox-tail wedging : Is a particular mode of morticing in which the end of the tenon is notched beyond the mortise.

Heartwood : The inner portion of wood in a tree or a log, which is usually of a darker colour.

Laminated wood : An assembled product made up of layers and adhesive in which the grains of the adjacent layers are parallel.

Log : The stem or trunk of a tree that is felled, branches lopped off and prepared for conversion.

Plank : A piece of sawn timber with thickness not exceeding 5 cm and the width exceeds twice the thickness.

Principal Rafter : A roof member which supports purlins.

Purlin : A roof member directly supporting roof covering or rafter and battens.

Rebate : A recess or groove along the edge of a board to receive another board or a door, sash or a frame.

Sash : The framework which hold the glass in a glazed window or door.

Scantling : A piece of timber whose cross-sectional dimensions exceed 5 cm in both the directions and do not exceed 20 cm.

Seasoning : A process involving the reduction of moisture content in timber under more or less controlled conditions.

Seasoned timber : Timber whose moisture content has been reduced to the specified minimum.

Stud or Studding : The small timbers used in partitions and outside wooden walls, to which the laths and boards are nailed.

Splayed edge : A chamfer extended to the full thickness of the timber board.

Timber terms such as Boards, Battens, Planks, Deals, Scantlings, Laths, Strips are varyingly defined as regards their sizes, therefore, while ordering timber scantlings their sizes should be quoted.

2. STRUCTURAL PROPERTIES OF TIMBERS

Design of timber structures is based more or less on the same principles as steel structures except that the strength of a particular timber is very unreliable as compared to steel. Data for the strength of timbers is given at page 9/28 for the timbers generally used in India for engineering works. Timber has minimum strength in green condition and it increases in strength as it gets seasoned. When the moisture content is reduced to 12 per cent of the weight of the wood substance, the timber is said to be seasoned and it increases in strength by 20 to 50 per cent ; working stresses given in the table at page 9/4 are based on this assumption.

From tests on timber it has been well established that its resistance to suddenly applied loads is much greater than its resistance to slowly applied or constant loading. The safe working stresses recommended in the table are for long continued or permanent loads. For suddenly applied loads producing impact, stresses up to 100 per cent of the force producing impact no adjustment of working stresses nor any assumption of an increased equivalent dead load are necessary.

The column for the weight of timbers gives average weight of a seasoned timber with 12 per cent moisture content. Weight is a good guide for the strength. A heavier scantling is generally stronger and a lighter one weaker than scantling of average weight of the same species. Very light scantlings showing a weight less than 75 per cent of the average weight should not be used as they may be unsound or very poor in strength.

Good Class Timber. In the same class of timber, the slower the growth or the narrower the annual rings, the better. The cellular tissue should be hard and compact. The fibrous tissues should adhere firmly together and should not clog the teeth of the saw or show woolliness at a freshly cut surface. Depth of colour indicates strength and durability. Freshly cut surface should be firm, shining and somewhat translucent. A dull, chalky appearance is a sign of bad timber. In resinous timbers, that with least resin in its pores is strongest and most durable. In non-resinous timbers, that with least sap is best.

A good class timber is uniform in colour, with straight grains, and is free from dead knots, cracks, shakes and lapwood. It has regular annual rings. Close grained timber is stronger than loose grained. Six rings per 2.5 cm should be considered as the minimum for the closeness of grain for a structural timber.

Softwoods and Hardwoods. Chir, deodar, kail, walnut, spruce are generally considered as softwoods and, teak, sal, shisham, babul as hardwoods. Softwoods are indented across the grain with a pressure of 70 kg/sq. cm to a depth of 1 mm or more, while hardwoods need a pressure of over 98 kg/sq. cm for the same indent or less.

Safe Working Stresses for Indian Timbers Commonly Used : Standard Grade (ordinary good class)

Trade Name	Co-efficient for Deflection L/360	Co-efficient for Strength WL/8	Average Weight in kg/cu. m	Modulus of Elasticity in tonnes/sq. cm. All grades	Bending and Tension along grain extreme fibre stress kg/sq. cm		
					Inside location	Outside location	Wet location
Babul or Kikar	1.27	1.61	835	108	182	154	124
Benteak	1.27	1.17	675	110	138	112	92
Blue Pine	0.80	0.58	515	68	66	56	50
Chir	1.13	0.72	575	98	84	70	60
Deodar	1.07	0.88	560	95	102	88	70
Fir, Partal	1.07	0.66	465	94	78	66	56
Haldu	1.07	1.17	675	91	138	112	92
Kail	0.80	0.58	515	68	66	56	50
Sal	1.47	1.44	800	127	168	140	112
Spruce	1.07	0.66	480	92	78	66	52
Teak	1.07	1.22	625	96	140	116	94
Walnut	1.07	1.00	575	91	116	94	78

Trade Name	Shear Stress all locations kg/sq. cm		Compressive Stress kg/sq. cm						Durability grade
			Parallel to grain			Perpendicular to grain			
	Horizontal	Along grain	Inside location	Outside location	Wet location	Inside location	Outside location	Wet location	
Babul or Kikar	15.4	22.2	112	102	80	65.5	50.5	41.5	III
Benteak	9.2	13.0	88	78	64	41.5	32.5	26.0	I
Blue Pine	5.6	8.0	52	46	38	17.0	13.5	10.5	III
Chir	6.4	9.2	64	56	46	22.5	17.5	14.0	III
Deodar	7.0	10.2	78	70	56	26.5	21.0	17.0	I
Fir, Partal	6.0	8.4	60	52	42	16.0	12.5	10.5	III
Haldu	9.4	13.4	84	74	64	36.5	28.0	23.0	III
Kail	5.6	8.0	52	46	38	17.0	13.5	10.5	III
Sal	9.4	13.4	106	94	78	45.5	35.0	29.0	I
Spruce	6.0	8.4	56	50	42	17.0	13.5	10.5	III
Teak	9.8	14.0	88	78	64	40.0	31.0	25.5	I
Walnut	8.4	12.0	66	60	50	23.0	18.5	15.0	III

(From IS : 883) working stresses are in kilograms per sq. cm.

The value of horizontal shears to be used only for beams. In all other cases shear along grain to be used.

The safe working stresses for timber of select grade (exceptionally good quality) and timber of common grade for bending, shear and compression shall be 7/6 and 5/6 times, respectively, of the safe working stresses for timber of standard grade (given in the table).

Deflection :—The deflection in the case of beams, joists, purlins, battens and other flexural members supporting brittle materials like gypsum ceilings, slates, tiles and asbestos sheets shall not exceed 1/360 of the span. The deflection in the case of other flexural members shall not exceed 1/240 of the span in the case of beams and joists, and 1/150 of the freely hanging length in the case of cantilevers.

The position of knots in a beam is of great importance as the weakening effect of a knot is greater in tension than in compression.

3. SEASONING OF TIMBER

The process of drying timber under more or less controlled conditions is called seasoning. Seasoning of timber is the first step in the efficient utilization of timber. Freshly felled timber contains a large quantity of moisture ; in many cases as much as 100 per cent, while a well seasoned timber may, on the other hand, contain only about 10 to 12 per cent. As such, all timbers need proper seasoning. Timber shrinks during drying ; the main object of seasoning is to eliminate this shrinkage before using the timber.

Moisture content of the timber bears a definite relation to the relative humidity of the atmosphere, and if a piece of timber with a low moisture content be placed in an atmosphere of high relative humidity, the wood will rapidly absorb moisture and expand. This is often noticed in the rainy seasons. Even seasoned timber has to be stored under cover protected from direct sun and rain. Denser varieties shrink more. For superior and indoor works, a moisture content of up to 5 per cent may be taken, but such highly dried wood is very hygroscopic and should be soaked in some preservative to prevent the absorption of moisture. Well seasoned wood lasts longer under all conditions.

A timber is considered fit for carpenter's work when it has lost 1/5th of its original weight and fit for joiner's work when 1/3rd of its weight has been lost.

After felling the tree the bark should be removed immediately. If cannot be sawn, the logs should be stored under water or out of contact with water. If sawn, the timber should be stacked under shelter for seasoning in a dry place about 30 cm above floor level with longitudinal and cross pieces arranged one upon another leaving a spuce of about 5 cm in between for free circulation of air. Moisture enters and leaves timbers more readily through the end grain and special care should be taken to ensure that where water tends to lie on the end fibres of wood they are adequately protected by a water proof membrane, as timbers often begin to deteriorate through an accumulation of moisture at these points.

4. GENERAL DESCRIPTION OF COMMON TIMBERS

Teak, Deodar, Sal, Benteak, Bijasal are considered as class I timbers while Khair, Babul, Haldu, Mungo, Chir, Hollock, Fir, Partal, Siris, Spruce, Walnut, Kail, as class II timbers.

Teak Wood is considered the best wood for retention of shape and in durability. Usually remains immune to white-ant attack and insect attack for very long periods, but is not always immune from fungus attack (rot). It is easy to saw and work to a fair surface and takes polish well. Used for important timber construction works and for making furniture.

Deodar Wood. Its weight and strength is 20 per cent less than teak. It is easy to saw and works to a smooth finish. It is not, however, a suitable wood for polish or paint work as the oil in the wood, aspecially near knots, always seeps through such finishes and discolours them. It is used for all types of engineering works and light furniture where finish is not very important.

Sal Wood is about 30 per cent heavier than teak, 50 per cent harder and about 20 to 30 per cent stronger. In shock resistance it is about 45 per cent above teak. Usually remains immune to attack by white ants and fungi for a long period. Well dried sal is not a really easy wood to saw and work. It is a rough constructional wood used for a variety of purposes such as beams, rafters flooring, piles, bridging.

Kail Wood is not a very durable wood but it is easy to saw and work. It can be brought to a fine smooth surface, is more suitable for paint and enamel finishes than for polish work.

Babul and Khair Woods. These are very strong, hard and rough timbers. Somewhat difficult to saw and machine but finish and polish well.

Haldu. Is a fairly hard and strong wood, about 10 per cent harder than teak. It is very easy wood to saw and machine and finishes very easily and takes strain and polish well. It cannot be dovetailed because it breaks away. Commonly used for door and window frames, floor boards, panelling, etc.

Mango. It is not a very durable wood in exposed positions. It is very liable to fungal staining and decay and is not immune to white ant attack. It can be easily finished to clean surface.

Chir is not a very durable wood unless treated with a preservative. It is easy to saw and can be brought to a fine smooth surface, but is more-suitable for paint and enamel finishes than for polish.

Benteak is very close to teak in strength. It is suitable for all constructional works and is not prone to fungus attack. It can be finished to a fine smooth surface and takes a good polish.

Plywood is made by cementing or glueing together three or more layers of thin sheets of wood (called veneers) into panels, usually with the grains of adjacent sheets running at right angles to each other.

Block-Board is a board having a core made of strips of wood, each not exceeding 25 mm in width, laid separately or glued or otherwise joined to form a slab which is glued between two or more outer veneers with the direction of the grain of the core blocks running at right angles to that of the adjacent veneers. Block-board are manufactured in various thicknesses from 12 to 50 mm.

Batten board is a board having a core made up of strips of wood usually 8 cm wide, each laid separately or glued or otherwise joined together to form a slab which is glued between two or more outer veneers with the direction of the grain of the core battens running at right angles to that of the adjacent outer veneers.

Batten Boards and Lamin Boards are also commercially made in somewhat the same manner.

5. TIMBER DECAY

Timber is liable to deterioration from a number of causes amongst which are fungi, insects, and marine borers. Fungi are low forms of plant life and are the most destructive ; damp is essential to their development and fungi will not attack dry wood with a moisture content of less than 20 per cent nor will they attack wood which is completely saturated. Fungi grow by means of hair-like threads which travel through the wood, ultimately reducing it to powder ; these are responsible for the major part of the destruction of timber. No timber is known to be absolutely immune from attack although a few (such as teak) are highly resistant. Timber in a dry and well ventilated place or continuously submerged under fresh water will last indefinitely. When subjected to alternate dry and wet conditions, or used in dark, damp and unventilated positions, it deteriorates very soon. Wood embedded in ground will decay unless treated with creosote, coal tar or some such material, or charred. Timber in salt or brackish water is particularly susceptible to attack by marine boring organisms ; the rapidity of attack depends on local conditions and the kinds of the organisms present. No species of timber in its natural condition is absolutely immune from the attack of marine borers.

There are two kinds of rot, *dry-rot* and *wet-rot*. Timber exposed to to confined air alone, without the presence of any considerable quantity of moisture, decays by dry-rot which converts the wood into fine powder.

Outbreaks of attack are nearly always due to excess moisture coming in contact with the wood. Wet-rot may occur while the tree is standing. White-ants or termites are also very injurious to timber. Teak, Sal, and Deodar withstand the attack of white ants. Remedial measures for white ants have been described in Section 7.

Timber should be kept dry during construction for example, the erection of wooden door frames and window frames concurrently with the building of the brick walls in wet weather is to negative all the benefits of seasoning. The construction should be such as to protect the timber from damp during life of the building. All timber should be protected from damp during life of the building by providing dampproof courses. Where floor boards from ground moisture by providing dampproof courses. Where floor boards on ground floors are fixed to sleepers laid on concrete or wood blocks direct to it, a layer of asphalt or bituminous mastic should be provided in between. Timber should be clear of the influence of damp earth or damp walls, and free from contact with mortar. All under-floor spaces should be well ventilated.

6. PRESERVATION OF TIMBER

Preservatives are used to protect timber aginst deterioration due to fungi and attacks of termites, borers, and marine organisms. Most efficient means of preserving timber are, good seasoning and free circulation of air. Protection against moisture is afforded by oil-paint provided that the timber is perfectly dry when first painted otherwise the filling up of the outer pores only confines the moisture and causes rot. A pre-requisite for satisfactory treatment is that the timber shall be seasoned so that the outer layers have a moisture content of less than 20 per cent. For exposed timber the only remedy at present available is impregnation by substances poisonous to fungi, these substances being either of the oil or chemical types.

Description of Preservatives

Oil Type Preservatives

Coal Tar Creosote is a fraction of coal tar distillate and is the most important preservatives of the oil type having been in use for a very long time and is specially suitable for the treatment of timber for exterior use, *e.g.*, railway sleepers, poles, piles, etc. Although creosote has been fairly satisfactory but after sometime in exposed situation, it tends to leach out, and has an unpleasant odour ; is not clean to handle and the timber treated with it cannot be painted or polished. Surface application of creosote has little, if any, value, and for satisfactory protection a deep impregnation of the preservative must be obtained.

Coal tar is a good preservative (but not so effective as the creosote derived from it) and is more suitable for surface applications. It is less toxic to wood destroying agencies and being very viscous does not penetrate the wood deeply. It should be applied hot. Coal tar is sometimes mixed with ordinary creosote. All timbers embedded in masonry or in contact with masonry should be well tarred before erection with three coats of hot coal tar into which quick lime powder has been throughly incorporated in the proportion of 1 kg of lime to 5 litres of tar. Framed joints must be coated with paint before frames are put together. When the end of a beam or any woodwork is buried in masonry or brickwork, an air space of 6 mm should be left at the ends, sides and top.

Chemical Type Organic Solvent Preservatives. These preservatives are used after dissolving them in suitable organic solvents such as, naptha, kerosene and white spirit. They are clean to handle and are more or less permanent but some of them are inflammable and care is necessary in handing the solution. These preservatives are applied cold and in most cases the timber treated with them can be painted or polished. DDT is an example.

Water Soluble Preservatives. They are comparatively cheaper and the timber treated with these preservatives can be painted or varnished when dry. The chemical solutions are, however, apt to leach out (and washed out, being water soluble) when the timber is exposed to wet conditions (*i.e.*, the preservative gets gradually depleted owing to the dissolving effect of water), though more recent preservatives employ acidified solutions which after impregnation deposit the chemical in the wood as an insoluble salt. This leaching of the preservative can be, to some extent, minimised if a water-proof paint coating is applied on the treated timber and is properly maintained. Most of these chemicals are suitable for inside locations only for protecting timber not in contact with ground, and are not suitable for works underground and severe conditions of exposure.

"Ascu" has been quite successful. It is in powder form and 1 part of the powder dissolved in 16 parts of water (by weight) gives a solution for ordinary use. The solution can be applied with a brush or the wood soaked into it. The treated wood can be painted or polished. *Zinc chloride* has some fire-retardant properties also.

The most common method of applying a preservative is by brush, but this gives only limited protection. Better results are obtained with a hot solution or by spraying, but dipping or steeping is much more effective. Still better results can be obtained by using what is known as the open hot tank and cold process. Impregnating timber by applying the preservative under pressure is the most effective method.

Timber to be treated should be dried to an appropriate moisture content and whenever possible all work on the timber should be completed and it should be fully fabricated and all cuttings and drillings done before preservative treatment is applied. Where subsequent cutting or working is unavoidable, preservative should be liberally applied to the freshly worked surfaces.

Fire-Proofing Timber. For fire-proofing timber, the method recommended is the pressure impregnation of the timber with large quantities of chemicals, the most common of which are ammonium diphosphate, sodium arsenate, sodium tetraborate. Fire-resistant paints are also available. White-washing is effective to some extent in retarding the action of fire. It is not possible to make timber fire-proof, chemicals and paints only retard the action of fire. Timber can be rendered non-inflammable in that it will not flame or glow but merely char, and will not, therefore, assist in the propagation of fire. A dense wood offers greater resistance to fire than a lighter one. Presence of resins and oils in wood increase combustibility. No woodwork of any kind should be laid within 60 cm of a fire-place or a flue.

All portions of timber abutting against masonry or concrete or embedded in ground shall be painted with a wood primer or two coats of boiling tar.

7. DESIGN OF BEAMS AND BATTENS

All timber beams are generally designed as simply supported considering both strength and deflection.

Timber never fails by shear either along or across the grain. Design formula has been given at the end for Indian timbers used for engineering works. Deep beams are more economical than shallow ones but this proportion can be used only up to a ratio of about d/b of 3 beyond which beams will have to be secured laterally so that they have no tendency to turn over or fail by buckling. Beams are supported laterally by fastening boarding on top, or by using wooden packing blocks, or by cross braces (called "bridging") at intervals. All beams and battens having a depth exceeding three times their breadth and/or a span exceeding twenty five times their breadth should be laterally restrained from twisting or buckling and the distance between such restraints shall not exceed twenty five times its breadth. (ISI)

The strength of a circular beam is only 3/5th that of a square beam, the side of the square being equal to the diameter of the circular beam. The strength of a square beam on edge (when one of its diagonals is vertical) is 7/10th of the strength when it is resting on either side. Strongest beam cut from a cylindrical log is one when breadth to depth is 5 to 7. Draw any diagonal and divide it into three parts as shown in the illustration.

Roughly, the depth of a beam is 1/24 of the span. For a well proportioned beam, breadth is 0.6 to 0.75 depth. For deep beams in floors and flat roofs the width need not be more than 1/40 of the span, especially when boarding is nailed to it. The width of a beam should not be less than 50 mm (40 mm min :) and spacing 30 cm.

Timber Beams

Distributed Safe Loads in kg on Beams 25 mm wide

Table-I. Simply Supported—Designed for Deflection

d	Clear span in metres										
mm	1.2	1.5	1.8	2.1	2.4	2.7	3.0	3.4	3.7	4.0	4.3
75	124	80	56	41	31	23	20	16	14	11	10
100	300	192	133	97	75	59	48	39	33	28	24
125	584	372	260	191	146	115	93	77	65	55	47
150	1012	648	450	330	253	200	162	133	112	95	82
175	1604	1029	714	525	401	317	257	213	178	158	131
200	2400	1536	1066	783	600	474	384	317	266	227	196
230	3416	2184	1518	1130	854	675	546	451	379	323	279
250	4684	3000	2083	1530	1171	926	750	619	520	443	382

Table-II. Simply Supported—Designed for Strength

d	Clear span in metres										
mm	1.2	1.5	1.8	2.1	2.4	2.7	3.0	3.4	3.7	4.0	4.3
75	202	162	135	115	101	90	81	73	67	62	57
100	360	288	240	205	180	160	144	130	120	110	102
125	562	450	375	321	281	250	225	204	187	173	160
150	810	648	540	462	405	360	324	294	270	249	231
175	1102	715	685	630	551	490	441	401	367	340	315
200	1440	1152	960	823	720	640	576	523	480	443	375
230	1822	1458	1215	1041	911	810	729	662	607	561	520
250	2250	1800	1500	1285	1125	1000	900	818	750	692	642

(*a*) The above tables are worked out for 25 mm width of beam. Actual width of the beam in multiples of 25 mm should be multiplied by these figures for full load.

(*b*) Where the ends of beams are partially fixed with bending moment of WL/10, Table I loads should be multiplied by 2.5 and Table II loads by 5/4.

(*c*) Tables have been worked out with co-efficient of 1 (which will work for most of the common good class timbers). For different class of timbers co-efficients are given in the tables at page 9/4, which may be multiplied with the loads given.

Deflection in the case of beams, joists, purlins, battens and other flexural members supporting brittle materials like gypsum ceilings, slates, tiles and asbestos sheets shall not exceed 1/360 of the span. Deflection in the case of other flexural members shall not exceed 1/240 of the span in the case of beams and joists and 1/150 of the freely hanging length in the case of cantilevers.

Flexural members shall not be cut, notched or bored except as follows :

(*a*) Notches may be cut in the top or bottom neither deeper than 1/5th of the depth of the beam nor further from the edge of the support than 1/6th of the span.

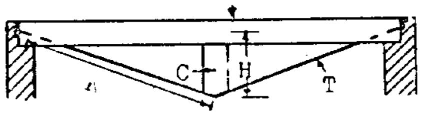
(*b*) Holes not larger in diameter than 1/4th of the depth may be bored in the middle-third of the depth and length.

Percentage Reduction for Concentrated Loads in the Vicinity of Supports

Distance of load from the nearest support	1.5 D or less	2 D	2.5 D	3 D or more
Percentage reduction	60	40	20	nil

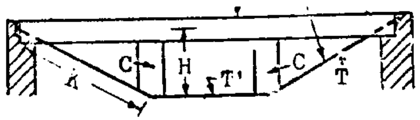
DESIGN OF TRUSSED (WOODEN) BEAMS

Wooden Beam



Up to 9 metres Span

Wooden Beam



Up to 12 metres Span

(*a*) Uniformly Distributed Load W on Full Span L

	Single Strut	Double Strut
Tension in rod (T)	5 WK/16 H	WK/3 H
Compression in strut (C)	5 W/8	W/3
Compression in top beam	5 WL/32 H	WL/9 H
Bending in top beam	WL/32	WL/72

(*b*) Concentrated Load P over Strut

Tension in rod (T)	PK/2 H	PK/H
Tension in rod (T')	—	WL/8 H
Compression in strut (C)	P	P
Compression in top beam	PL/4 H	PL/3 H

The beam may be composed of a single timber or of two or more timbers placed side by side.

8. DESIGNS OF COLUMNS

The safe working stresses are given at the end for compression (parallel to grain) which have to be taken for the design of columns. The main design factor for columns is the slenderness ratio, *i.e.*, the ratio of the unsupported length to the least dimension of cross section (L/d).

Short columns, where L/d is up to 10, may be designed for the full value of the compressive strength given in the table. For L/d between 10 and 15, 3/4th of the compressive strength should be taken and beyond 15 values may be taken from the following table :

L/d	A	L/d	A	L/d	A	L/d	A
1 to 10	1.00	10	.683	28	.533	38	.366
10 to 15	.750	20	.656	30	.500	40	.333
16	.733	22	.633	32	.466	42	.300
17	.716	24	.600	34	.450	44	.262
18	.700	26	.566	36	.400	45	.250

Multiply the allowable compressive strength by "A".

The corner posts will usually need a bigger size. In timber-framed constructions, there should be provided an additional post of the same size as the corner post at a distance of 0.91 m from the corner post in each direction, or alternatively, the corner post should be strutted at this distance into the top beam.

For determinining L/d ratio of a tapered column its least dimension shall be measured at a point one-third of the length from the top but shall not be taken as more than one and a half times the least dimension at the top.

Take only 7/8th load where corners are chamferred. Round columns take about 0.77 load of a square column of the same side as diameter. Compression members should not be notched.

The following safe loads in kg may be allowed on Verandah posts, for common timbers :—

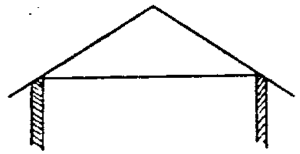
Section of Post	Height of Post in metres				
mm	1.8	2.1	2.4	3	3.7
100×100	3500	3200	3000	2400	1900
125×125	6400	6000	5600	4900	4200
150×150	10000	9600	9100	8200	7300

The following sizes of posts are recommended by the Bombay Municipality for timber-framed buildings :—

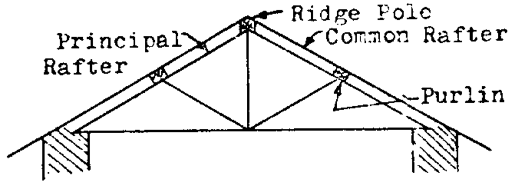
	Height of Post	Size mm
(*i*) Where the building consists of not more than one storey :—		
	Not exceeding 3 m	125×125 or 150 dia.
(*ii*) Where the building consists of not more than two storeys :—		
Lowest or 1st storey	Not exceeding 3 m	150×150 or 175 dia.
2nd storey	,,	125×125 or 150 dia.
(*iii*) Where the building consists of not more than three storeys :—		
Lowest or 1st storey	Not exceeding 3 m	175×175 or 210 dia.
2nd storey	,,	150×150 or 175 dia.
3rd storey	,,	125×125 or 150 dia.
(*iv*) Where the building consists of not more than four storeys :—		
Lowest or 1st storey	Not exceeding 3 m	200×200 or 240 dia.
2nd storey	,,	175×175 or 210 dia.
3rd storey	,,	150×150 or 175 dia.
4th storey	,,	125×125 or 150 dia.

The above sizes are for hardwood, if softwood is used, increase sectional area by 25 per cent. Where the height of posts exceeds 3 m, the sizes shall be increased by 5 mm for each additional 30 cm of height. The posts should not be more than 3 m apart. Where the distance apart of posts is less or more than 3 m, the dimensions should be proportionately decreased or increased but in no case shall the smallest dimension be less than 1/30th of the height of the post or less than 125 mm.

9. ROOF TRUSSES



Couple Close Truss
Up to 5.5 metres Span



King-Post Truss
6 to 9 metres Span

(For Design of Roofs in general, see Section on "Roofs")

The following types of timber roofs are generally built :

A *couple-roof* has only two rafters halved and nailed at the top without any tie. This type of roof is suitable up to 3 metres spans and on walls which can take thrust without spreading.

A *couple-close* roof has a tie fixed at the feet of the rafters over the walls and is suitable for spans up to 5 metres with 5.5 metres maximum. Where headroom is required, a *collar beam* is fixed half-way up the rafters instead of the tie beam. This roof is not so strong as the couple-close roof and is not used for more than 4.5 metres span.

Collar beams should not be very high as that weakens the truss. For high walls and high collars, *knee braces* should be provided. A collar or tie may be fixed to every third or fourth coupled rafters instead of to all. Where ties or collars are not provided, there will be "thrust" on the walls due to truss loads for which the following thickness of walls is recommended :

Span	Brickwork in lime: Thickness	Brickwork in lime: Height of wall not to exceed	Stone walling
3 m	9" (1 brick)	2.1 m	16" (40 cm)
	13½" (1½ bricks)	4.2 m	Height must not
5.5 m	13½" (1½ bricks)	3 m	exceed 4.5 m

Rafters are bird's mouthed over the wall plates to prevent their slipping. If a light king-rod is fixed passing through the centre of the rafters at top and through the tie or collar at the bottom, it will make a couple-close roof much stronger,

Coupled rafters should be spaced the maximum distance apart over which battens or boardings will carry the load (generally 65 to 100 cm.). These types of roofs should not be spaced more than 90 cm apart for 150 kg/sq. m and 120 cm apart for 100 kg/sq.m roof load.

King-Post Truss

King-post trusses are suitable for spans from 6 metres to 9 metres and Queen-post trusses 9 metres to 13 metres. King-rod of mild steel or wrought iron is preferable to a wooden king-post, diameter is about 3 mm per metre of span. The king-rod has a long thread and can be tightened up, thus bracing the whole truss. Where an iron rod is not to be used it

is preferable to provide two scantlings of timber, about 40 mm thick and of width according to the design, one on each side of the rafters and the tie ; nailed with the rafters and the tie, without making any tenon joints. The struts can also be jointed directly on to the tie and the rafters (without tenon joints) with iron straps fixed on to them. Where iron straps are not to be used for joints, wooden pieces for lap joints can be used, and nailed.

Ridge Pole is provided over the king-post truss to support the common rafters. Usual size is 75 × 150 mm, which should be fixed according to the roof load as a simple beam.

Tables giving sizes of scantlings for king-post trusses for hardwoods (teak, sal, etc.), spacing 3 metres, slope of roof 1 in 2 :

Span metres	Principal Rafter mm	Struts mm	Tie Beam: With Ceiling mm	Tie Beam: Without Ceiling mm	King Rod Dia. mm
Roof Load 220 kg/sq. metre					
4.25	100×125	100×75	100×125	100×100	20
5.00	100×125	100×75	100×125	100×100	20
5.50	100×150	100×75	100×125	100×100	20
6.00	100×150	100×75	100×125	100×100	22
6.50	100×175	100×100	100×150	100×125	25
7.50	100×200	100×100	100×150	100×125	25
8.00	100×200	100×100	100×175	100×150	32
8.50	100×200	100×125	100×200	100×175	40
Roof Load 170 kg/sq. metre					
4.25	100×100	75×75	100×125	75×100	16
5.00	100×125	100×75	100×125	100×100	16
5.50	100×125	100×75	100×125	100×100	20
6.00	100×150	100×75	100×125	100×100	20
6.50	100×150	100×75	100×125	100×100	20
7.50	100×175	100×75	100×125	100×100	25
8.00	100×175	100×100	100×150	100×125	25
8.50	100×175	100×100	100×175	100×150	32
Roof Load 120 kg/sq. metre					
4.25	100×75	75×75	100×100	75×100	12
5.00	100×100	75×75	100×100	75×100	12
5.50	100×100	100×75	100×100	75×100	16
6.00	100×100	100×75	100×100	75×100	16
6.50	100×125	100×75	100×125	75×100	20
7.50	100×125	100×75	100×125	100×100	20
8.00	100×125	100×75	100×150	100×100	22
8.50	100×150	100×100	100×150	100×125	25

Principal rafters are usually 2.5 m centres and common rafters 0.7 m centres.

Variations in the span, spacing or rise of trusses ± 25 per cent will make no appreciable difference in the sections of the members.

For roofs with heavy type of ceiling, about 3/4th of the above spacing may be adopted.

For making trusses, a full size truss diagram should first be made on a levelled platform from which templates of all joints as for tenons, mortices, scarves etc. should be made as a guide to ensure all the trusses being of the same size. The templates should be made to correspond to each member, and plate holes for screws and bolts marked accurately on them and drilled. Before fabrication of the truss, individual members should be assembled together to ensure close abutting or lapping of the surfaces of the different members and fitted close together as per drawing.

The trusses should be stayed temporarily till they are permanently secured in position and connected with each other by means of purlins. The trusses should be screwed to walls by means of holding down bolts.

The spacing between timber trusses is usually kept between 2.5 m and 3.0 m. It is economical to arrange the panels of principal rafters of length between 2 m and 2.5 m.

Couple-Close, King-post and Queen-post are the most common types of timber roof trusses used.

Steel trusses are preferred for spans beyond 7.6 metres. It is difficult to get suitable timber for trusses beyond 8.5 metres span.

Table on the last page gives sizes of scantlings for different king-post trusses. Cross-sectional areas of the scantlings are nearly proportional to the spacing of trusses. If soft-wood is used instead of hard-wood, the size of the scantlings (cross-sectional area) should be multiplied by 1.7.

The size of purlins and common rafters should be calculated as beams under the roof load.

The king-rod should be screwed up at the bottom with the tie with a washer and nut before the bolts in the straps are tightened.

Size of Roof Purlins in millimetres

Span	Max: inclined distance apart in metres		
metres	1.8	2.3	2.75
1.8	115×75	125×75	150×75
2.4	150×75	175×75	175×75
3	175×75	175×100	200×100
3.65	225×100	223×115	225×125
4.25	225×125	250×115	250×100
5	250×100	250×125	250×150

For design load on roofs, see Section on "Roofs".

Procedure for Design of Roof Trusses :

Fix size of battens according to the weight of the roof coverings.

Minimum size of battens is 25 mm × 32 mm and spacing 30 cm centres. If the battens are of softwood, take minimum size 32 mm × 38 mm. Fix maximum span for the battens (or boardings) according to the roof load (as simply supported beams).

Spacing of rafters = Span/4 + 14 cm. Check with span for battens.

Design rafter for deflection as a simple beam with load normal to roof. Span of rafter is $= L/2 \sec. \phi$. ϕ is roof angle. For rafters a span of 2.5 m should rarely be exceeded. Common rafters are considered partly fixed if continuous and spiked to ridge pieces, purlin and wall plate.

Battens and Purlins in Sloping Roofs

Purlins and battens with half lap joints where semi-continuity exists over the supports need be designed only for strength as simply supported beams and not for deflection.

In sloping roofs with no rigid covering such as boarding or sheeting, the battens, purlins, and bressumers are subject to tangential force down the roof slope as well as to the normal force, but if they are designed as simply supported beams for the normal load and are nailed at each rafter intersection they will have sufficient strength in both directions.

Ties : Ties may be of wood or iron.

Design tie for tension $= \frac{WL}{4h}$ Min: size of a wooden tie is 65×25 mm

$W = w\,LD \sec. \phi$,

w = weight per sq. m of roof load (normal to roof),

L = span in m,
D = spacing of rafters,
h = height of truss.

Section available foe tension is the net area at the connection to the rafter, or at the joint if the tie beam is in two pieces.

A tie should have a min. bearing of 20 cm on walls or piers.

Ties should not be used to support a ceiling for spans of more than 3.6 m without supporting the ties. Where a ceiling is supported, ties should be calculated for a deflection of L/480.

The rods may be of steel or wrought iron and are preferable to wooden ties. Diameter is 5 mm per metre of span. Iron ties are fixed in the centres of the rafters by drilling holes in them and are tightened up by nuts and washers. These ties should not be fixed on the sides of the rafters.

Where there is an additional load on the tie (concentrated or distributed) as a beam, find out the section required, keeping a constant width, due to this bean load and add this width to the section of the tie found from the truss load. Deflection will seldom govern.

Tie beams may be given a *camber* of 1/240

Tie beams should be checked for stiffness deflection where ceiling is provided. A suspender from the ridge may be required with a ceiling.

The following sizes may be taken for the design of minor trusses (couple close or collar beam), with roof slopes of 1 in 2 :—

Load – 100 kg/sq. m of roof surface Spacing—1.22 m Timber—Sal or Teak	Load—150 kg/sq. m Spacing—0.85 m Timber—Sal or Teak
Load—100 kg/sq. m Spacing—0.85 m Timber—Deodar or Kail	Load—150 kg/sq. m Spacing—0.61 m Timber—Deodar or Kail.

Effective span in metres	Rafter	Tie or Collar	Ridge* Pole	Remarks
2.4	50×75	25×75	75×175	*Rafters are nailed to ridge pole.
3.0	50×90	40×75 or 25×100	,,	Size of ridge pole is suggested only for
3.7	50×100	,,	,,	common cases. This
4.3	50×115	40×90	75×190	should be calculated
4.9	50×125	40×100	90×200	for the load on the
5.5	50×140	50×115	,,	roof span as a simple beam.

Iron Stirrups and Straps may be made out of 50 mm×10 mm bars for trusses of spans 6 metres and over, and of 50 mm×6 mm bars for trusses below 6 metres. Three-ways (T) straps may either be cut from solid plate or welded ; they are made of wrought iron and fixed on both sides of the truss members. Both are generally of 16 mm diameter and fixed about 75 to 100 mm centres. Stirrup ends are forged out of the heel strap which will give a size of 16 mm diameter and 22 mm diameter from 50×6 mm and 50×10 mm bars respectively. A long washer is provided at the top and the strap tightened with nuts.

For estimating purposes, the weights of stirrups, straps, bolts, nuts and washers, etc., may be taken about 41 kg per truss for spans of 6 metres and above, and 32 kg for spans under 6 metres. Weight of the king-rod is extra.

Wall Plates should be securely fastened to the wall by 16 mm diameter anchor bolts with 45 cm lengths embedded in the wall with plate washers at the ends, at intervals of 1.5 m. Size of the wall plates may be 100 mm×75 mm for spans up to 3.7 m and 150 mm×100 mm above 3.7 m span. Bed plates should be provided under heavy trusses.

Heavy trusses must be anchored to the walls at each end with 20 mm dia. bolts 0.9 m long, with 100 mm×100 mm×6 mm washer plates at the ends.

All screws, bolts and nails shall be dipped in oil before use.

Verandah Roofs

Principal rafters are either prolonged by addition of another scantling bolted to their sides, or independent rafters are fixed on the main wall on one side and resting on the top of the verandah pillar on the other side. Principal rafters carrying a purlin are placed at 2.5 to 3 m apart when the width of the verandah exceeds 2.5 m. For verandah widths above 3 m, it is preferable to strut the principal and hip rafters to the extent of 1/3rd of their length from the wall. The struts to spring from the wall at 2 m from the floor level. In verandahs of widths less than 2.5 m common rafters are used 38 to 46 cm apart, (with max : up to 90 cm), according to the roof design.

To design scantlings, treat as half trusses, or calculate sizes independently as beams.

Weights of Roofing Materials

The following weights acting vertical (on horizontal projection) may may be taken for the roof coverings. These should be multiplied by the cos. of the roof angle to obtain weights normal to the roof surface.

	kg/sq. m
Asbestos sheets 6 mm thick (corrugated)	16
—do (flat)	11
C.G.I. sheets 24 B.G. gauge	7.3
C.G.I. sheets 22 B.G. gauge	8.5
C.G.I. sheets 18 B.G. gauge	13
Bituminous roofing felt	7.3
Glazed roofing (with 6 mm glass with lead covered steel bars) ...	29
Boarding 25 mm thick	15
Boarding 20 mm thick	10
Eternit sheets or tiles	12
Slates on batten	34
Slates on 25 mm boarding	59
Thatch with frame 23 mm	49
Thatch with frame 15 cm	32
Timber trusses light roofs	10
Timber trusses heavy roofs	15
Rafter	5
Battens	2.5
Single Allahabad tiles including battens	83
Double Allahabad tiles including battens	166
Mangalore tiles	54
Mangalore tiles with flat tiles	78
Mangalore tiles bedded in mortar over flat tiles ...	107
Mangalore tiles with battens	68
Flat and pan tiles	146
Plain pan tiles	107
Country tiles with battens, single	107
Country tiles with battens, double	117

Allowance for weight of truss, purlins, rafters, etc., may be taken at 25 kg/sq. m for common roofs and 30 kg/sq. m for heavy roofs, of covered area.

Load due to wind pressure has also to be added (see Section on "Roof").

Roof Slopes are generally kept as follows :—

			Min :
Mangalore Tiles screwed down	...	1 in 1 or	steeper
Mangalore Tiles not screwed down	...	1 in 1 or 2	1 in 3
Pan Tiles	...	1 in 2	1 in $1\frac{1}{2}$
Allahabad Tiles, Country Tiles	...	1 in 2	
Nainital or Dalhousie pattern etc.	...	1 in 2	
Corrugated iron—common	...	1 in 2	1 in 5
Thatch roof	...	1 in 1	1 in $1\frac{1}{2}$
States	...	1 in 2	1 in $2\frac{1}{2}$
Mud roof on tiles or C.G.I. sheets	...	1 in 8	

Allahabad and country tiles will slip if laid steeper than 1 in 2 ; tiled roofs leak if laid flatter than 1 in 2.

Truss Joints

Scarf Joints are made to resist compression.
The length of a scarf should bear the following proportions to the depth :—

	Without bolts	With bolts	With bolts & fish plates
Hardwoods	6 times	3 times	2 times
Softwoods	12 times	6 times	4 times

Keys should be 1/3rd the depth of the member.

Fish Joints are made to resist tension. For lengthening ties a plain fixed joint is most economical. The two ends of the beam are butter together and secured on each side by means of bolts.

Rafters should be half lapped at the ridge joint.

Collar beam may be halved on to the rafter but the rafter must not be cut in the centre.

Tie of a couple close roof may be halved on to the rafter (both cut).

In compression joints, members are generally notched to bear on each other.

As far as possible members should not be cut but lap joints should be made and fixed with iron or wooden straps and bolts.

Glue should not be used in joints which are exposed to the weather, any hard stopping should be done with tight driven plugs.

Tests for Soundness of Old Timber Trusses

Sag in a truss may be due to failure of splices, improper adjustment of vertical rods or crushing of struts at their ends. Boring the members with a 10 mm wood auger will show whether there is any decay inside ; where iron rods pass through timber, decay occurs at contact points. Sounding the iron rods with a small hammer will show whether each rod is carrying the same amount of tension.

10. PARTITIONS

Sizes of the principal members of the frames :

For spans not exceeding 6 m—100 mm × 75 mm
" " 9 m—125 mm × 90 mm
" " 12 m—150 mm × 100 mm

The filling in pieces (called "studs" or "quarterings") need not be thicker than 50 mm, or of just sufficient thickness to nail the laths on to them. They are tenoned to the top and bottom plates but butted and nailed on to the braces. If they exceed about one metre in length they should be strengthened by short struts or horizontal pieces, called "nogging pieces". The studs or quarterings for a lath-and-plaster partition should be spaced at from 30 cm to 45 cm centres ; in a brick-nogged partition they should be 45 cm × 65 cm to 90 cm apart.

Trellis work or Jaffri. Wooden battens (strips or laths) 40 to 60 mm wide and 10 to 12 mm thick are fixed crossing diagonally in opposite directions or at right angles (nailed together at every alternate crossing), leaving 40 to 50 mm openings in between them. These are fixed to frame or beading 50 x 12 mm with screws.

11. FLOORS (Hardwood)

Floor boards should be 30 mm to 40 mm thick, 100 mm to 150 mm wide and 1.8 m to 3.6 m long ; resting on joists, usually 38 cm apart. For Deodar, Kail or Chir wood, the boards should not be less than 40 mm thick, and not more than 150 mm wide, and not more than 3 m long. For teak wood the size may be 25 mm thick and 100 mm wide.

Joints : Sides to be tongued and grooved or tongued and ploughed. Ends rest on the joist below and break joint with one another. Heads of all screws are counter-sunk ; two 75 mm screws should be used for each 150 mm plank wherever it crosses or ends on joist.

Single Floors :

Span for bridging joists ... 3.6 m to 4.6 m
Spacing centre to centre ... 30 cm to 40 cm
Width 50 mm to 65 mm
Depth in mm $= \frac{\text{Span in mm}}{24} + 50$ mm
Bearing on wall plates 115 mm

} Breadth is usually 1/4th to 1/3rd of the depth.

When bridging joists exceed 2.4 m in length, they should be strutted apart at intervals of 1.2 to 1.8 m by struts 25 mm wide and of full depth of the joist, fixed at right angles to the joists, or of 40 mm × 75 mm fixed by herring-boning. (The struts are fixed between the joists).

Double Floors :

Binders ... 1.8 to 2.4 m centre to centre
Bearings on templates ... 150 mm
Bridging joists rest on binders to which they are notched.

Framed Floors. Girders are placed 1.8 to 3 m centre to centre, binders are tusk tenoned into them – as this method weakens the girders it is preferable to use iron stirrups. To obtain maximum stiffness the breadth should be 5/7th of the depth. Sizes should be calculated as simple beams. Bearings on wall plates to be 23 to 30 cm.

Floors may be made high up in the centre by about 20 mm per 6 m to allow for the subsequent settlement which is likely to take place.

Wooden girders should be braced laterally to prevent buckling when the ratio of length to breadth exceeds 20 ; or designed with reduced fibre stress as follows :

Percentage of reduction :—

Ratio of length to width	20 to 30	30 to 40	40 to 50	50 to 90
Percentage reduction	25	35	40	50

The ends of all timbers set in masonry should have a space of 6 mm left on both sides to permit free circulation of air.

In damp locations provide metal caps on masonry foundation walls under wooden beams ; the metal sheets to project slightly on both sides of the walls. Damp-proof course or a damp-proof layer should invariably be provided on all walls. An air-space of about 45 to 60 mm should be left under the floors and ventilation provided. This will also stop termites.

Where ceiling joists are fixed, they are notched on to the bridging joists or binders.

Size of Wooden Floor Joists for Domestic Floors
Spaced at 40 cm centres (Hardwood)

Max: clear span (m)	Size of joist (mm)	Max: clear span (m)	Size of joist (mm)	Max: clear span (m)	Size of joist (mm)
1.0	40×75	3.0	40×150	4.33	50×200
1.4	50×75	3.2	50×150	4.44	40×225
1.7	40×100	3.5	40×175	4.65	65×200
2.1	50×100	3.8	50×175	4.85	50×225
2.4	40×125	4.0	40×200	5.21	65×225

12. STAIRCASES

Hardwood should be used for staircases, especially for the treads.

Wooden steps are generally supported on strings which are sloping members, one on the outside, one adjacent to the wall and one intermediate between the two where the steps are more than 122 cm wide. Treads and risers are either housed and wedged into the strings or the strings have their upper surface cut or notched to conform to the tread and riser of each step, the lower edge of the string remains parallel to the slope of the stairs. Housed or closed strings have grooves or housings cut in their inner sides to receive the ends of the treads and risers. The intermediate string which is

also called a *carriage piece* or a *bearer* is not cut but the treads and risers are fixed on to it by small angle blocks or brackets.

The following sizes may be used for hardwoods :—

Carriage pieces with brackets fixed thereto :

110 cm width of stairs 10 or 11 step flight, one carriage piece 50 x 100 mm, increasing for 140 cm wide stairs, 14 or 15 step flight to 50 x 175 mm. When the width of staircase exceeds 245 cm two carriage pieces should be provided.

Strings : 50 x 330 mm for close strings 110 cm wide stairs 10 or 11 step flight, increasing to 50 x 380 mm for close strings (wall strings) and 75 x 380 mm for cut strings 140 cm wide stairs 14 or 15 step flight.

The strings should be not less than 40 mm in thickness and of such breadth as will permit 25 mm above the front edge of the tread and 25 mm below the bottom edge of the riser.

Treads and Risers : Treads 32 mm thick, with risers 25 mm thick for 110 cm and 125 cm wide stairs each increasing by 3 mm up to 45 mm and 40 mm respectively maximum for every 15 cm extra width of stairs over 120 cm. The tread is generally made to project about 20 to 25 mm beyond the face of the riser and rounded off to form a "nosing" so as to increase the available width of the tread.

For general design principles of staircases and definitions of terms see Section 7 on "Masonry Structures".

For staircases in seismic areas see under "Miscellaneous Technical Information"—Earthquake Proof Buildings Design.

13. DOORS AND WINDOWS

Size of Frames (Chowkats) for Doors : Where the frames have to carry two sets of leaves (wire-gauzed leaves in addition to the panelled) increase breadth of the frames by 12 mm, i.e., 127 mm and 137 mm instead of 115 and 125 respectively.

For the size of the frames given the first dimension is the 'depth'—the dimension at right angles to the wall plane, and the second dimension is the 'breadth' – the dimension in the plane of the wall.

Size for iron-barred doors— 150 mm × 115 mm.

The door and window frames have rebates to house the shutters, and the depth of such rebate is 1.25 cm.

The minimum thickness of *panels* shall normally be 15 mm where the clear width of panel is not more than 300 mm, and 20 mm where the clear width of panel is more than 300 mm. It is preferable to use strips (of solid wood) of not more than 200 mm width to reduce chances of warping, splitting or other defects. The timber strips shall be joined together with continuous tongued and grooved joints glued together and reinforced with metal dowels. The grains of timber panels should run along the longer dimensions of the panels.

Where plywood or hard board is used for panelling, it should be of 10 mm thickness for two or more panels shutters and of 12 mm thickness for single pannel shutter.

Table showing Size of Frames and Parts for Doors and Windows

Particulars	Frames (Chowkats)	Shutters or Leaves Thick-ness	Width Stiles	Width Rails
	mm	mm	mm	mm
Doors :				
Glazed panelled				Not
Double, up to 1.22 m × 2.13 m	75 × 115	45	90	less
„ exceeding 1.22 m × 2.13 m				than
up to 1.52 m × 2.45 m	75 × 115	50	100	115
exceeding 1.52 m × 2.45 m	90 × 125	50	115	200
Single, up to 0.91 m × 1.98 m	75 × 115	45	100	for
„ exceeding 0.91 m × 1.98 m	75 × 115	50	115	lock
				rail
Windows :				
Glazed, double up to 0.91 m × 1.52 m	75 × 90	40	75	Not
„ „ up to 1.22 m × 1.52 m	75 × 100	45	75	less
„ „ exceeding 1.22 m × 1.52 m	75 × 115	50	80	than
Glazed, single, up to 0.61 m × 1.52 m	75 × 90	40	80	90
„ „ exceeding 0.61 m × 1.52 m	75 × 100	45	80	mm
Iron-barred up to 0.76 m × 1.07 m	100 × 75	—		—
„ up to 1.22 m × 1.52 m	125 × 75	—		—
„ with plank shutters	125 × 125	—		—
Clerestory windows	75 × 75	40	75	75

The middle of lock-rail is 75 to 80 cm above floor level. Rails which are more than 180 mm in width have two tenons. The thickness of each tenon is one-third the finished thickness of the member and the width not to exceed five times its thickness.

Rails and Stiles for Glazed and Panelled Doors and Windows : Middle of lock-rail is 0.76 m above floor level. Rails which are more than 180 mm in width have two tenons. The thickness of the tenon should not exceed one-fourth the thickness of the plank and the width should not exceed five times the thickness.

For *Wire-gauzed Shutters*, thickness of stiles and rails is the same as for the glazed and panelled shutters. Wire to be of galvanised MS wire-gauze designated 85 G or 140 G with wire of diameter 0.560 or 0.710 mm respectively.

In the case of double leaved shutters, the meeting of the stiles (closing junction of the two shutters) should be rebated by one-third of the thickness (not less than 2 cm) of the shutter.

Sash bars will usually not be less than 32 mm wide, the rebates therein to be 10 mm wide and 20 mm deep. Sash bars will have mitred joints with the stiles.

Ledged, Braced and Battened Door and Window Shutters : The ledge (horizontal) is 175 mm wide and 25 to 32 mm thick, and brace (inclined) is 125 mm wide. The ledges and braces are fixed to the inside face of door shutters with screws. The braces are inclined down wards towards the side on which the door is hung. Battens are 20 to 25 mm thick by 75 to 100 mm wide.

Iron-barred Shutters : Stiles, top, bottom and lock-rails to be 65 mm thick. Round iron bars 20 mm to 25 mm diameter or 20 mm square bars are fixed at 100 to 150 mm centres. The bars are housed into the top and bottom rails to a depth of 25 mm and passed through the lock-rail.

For fixing iron bars in wooden frames of windows, through holes are drilled in one frame and 5 cm deep in the other frame. The bars are passed into the frame from one side and fit correctly with the other frame and remain flush.

Internal doors should not, as a rule, be less than 0.85 m (minimum 0.70 m) wide by 1.98 m high. Doors of greater width than 0.92 m are generally made in two leaves. (The size given is between the door frames, the size of the opening through the wall being greater.)

Normal height of windows above floor is 0.76 m.

In clerestory windows the shutters are hung 75 mm off-centre to make them self-closing.

Louvered (Venetianed) Shutters : The louvers may be of wood, glass or A.C. Sheets. Blades are generally 90 mm wide and 10 or 12 mm thick and overlap about half their width ; they are secured to a moulded stanchion by 25 mm hinges or by wire clips and have rounded edges. The frame of each shutter is rebated outside all round in the sides and bottom rail and inside on the top rail. The ends of the blades are rounded off in the centre to 10 mm diameter by 20 mm long, and fit into holes in the rebated portion of the frame ; or they are fixed to the stiles by making grooves in the stile minimum 125 mm depth to receive the ends of the venetians. The venetian blades slope down towards the outside at an angle of 45°. The venetians may be fixed or movable.

Ledged, braced and battened door shutters fixed directly to the wall: Each door shutter is hung by means of two sets of iron pintals for shutters up to a size of 3.0 sq. m and three sets of pintals for shutters of size exceeding 3.0 sq. m. Each set of pintals consists of pin clamps of 50 x 6 mm flat iron 45 cm with 20 mm diameter pin at one end forked at the other end and 50 x 6 mm double strap 60 cm long. The pin is firmly riveted or welded, to the pin clamp. The pin is fixed into the masonry by means of cement concrete block 40 x 20 x 20 cm size. The double strap is so bent as to fit into the pin and swivel freely. The strap is fixed to the shutter by means of 12 mm dia. bolts and nuts, to the shutter at the ledges.

Ledged, braced and battened Garrage door shutters : 20 mm thick battens are fixed together by 30 mm thick ledges and braces on the inside face of door shutter. The ledges are 225 mm wide and braces 175 mm wide. The braces incline downwards the side on which the door is hung. Door shutters are fixed to the wall masonry with six pintle sets. Each set consisting of a clamp of 50 x 6 mm flat iron 45 cm long, bent and forked at one end and provided with 20 mm dia. M.S. pin on the other end and 50 x 6 mm double strap 60 cm long. The pin is firmly riveted or welded to the pin clamp, the other end of which is embedded in masonry by means of cement concrete block 40 x 20 x 20 cm. These are so placed that the bottom pin faces upwards and the top pin downwards, in order that the gate my not be removed by lifting over pins. The fixing is so done that the door shutter can open on the outside by 180 deg.

The double strap fits in the pin on one side and is fixed to the shutter at ledge on the other side with four bolts and nuts of 12 mm dia. One hook and eye, 450 mm size of mild steel is provided for each shutter to keep it fixed in open position. A cement concrete block of size 15 x 10 x 20 is embedded in the floor at junction of two shutters so that the door shutter open only on the outside and not on the inside. An iron sliding bolt (aldrop) 450 x 16 mm dia. is provided and fixed.

Trellis doors and windows shutters. Shutter frame consists of two stiles, top rail, lock rail and bottom rail, each of section 75 x 40 mm. The tenons pass through the stiles for at least 3/4th of width of the stile. The joints should be pressed and secured by bamboo pins of about 6 mm dia.

Hinges. 50 mm screws are used with 125 mm and 159 mm hinges and 25 mm screws with smaller sizes.

Rendering Panes of Glass Opaque : Frosting to make it ground glass :

White lead	1 kg	Mix the whole till becomes plastic.
Linseed oil	260 cu. cm	
Varnish	65 cu. cm	

The mixture should be applied by tying it up in a piece of linen into small balls and tapping these balls against the glass. They should not be rubbed over the glass as, if this is done, streaks make their appearance.

(For Glass and Glazing see Section 12.)

Dimensions of Wire Gauze for General Purposes (IS : 1568)

Gauge Designation	Av. width of aperture in mm	Nominal dia. of wire in mm	Remarks
160 G	1.60	0.950	Suitable for fly-proof screens
140 G	1.40	0.710	
120 G	1.20	0.600	Suitable for mosquito-proof screens
100 G	1.00	0.600	
85 G	0.84	0.560	Commonly used
80 G	0.79	0.530	
70 G	0.71	0.450	
60 G	0.59	0.425	
50 G	0.50	0.355	
40 G	0.42	0.280	

Expanded Metal (XPM) generally used for windows :

20 x 60 mm strands 3.25 mm wide and 1.6 mm thick (see page 4M/12) ; weight 4.078 kg/sq. m with wooden beading of 62 x 19 mm.

Steel wire fabric may be of mesh 75 x 25 of weight not less than 7.75 kg/sq. m.

Wooden Beading. The size of wood beads for glass panes not exceeding 0.5 sq. m in area and having thickness up to 4 mm is 1 cm x 2 cm, while for glass panes having thickness more than 4 mm, its size is 1.5 cm x 3 cm.

Hold-fasts are made of 40 x 3 mm flat iron 40 cm long. It has a hole at one end for fixing to frame with 10 mm dia. bolt. A 5 cm length at one end of the flat is bent at right angles and a hole made in it for fixing to the frame with 10 mm bolt. At the other end the flat iron is split and bent at right angles in the opposite direction. Three hold-fasts are fixed on each side of a door frame and two on each side of a window frame. Give an additional hold-fast where no sill has been provided.

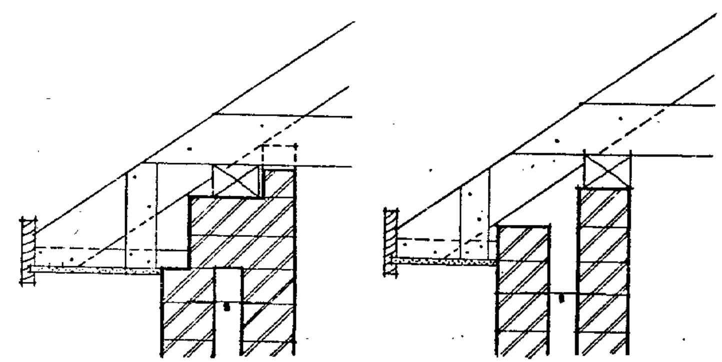
Fittings for Doors and Windows (Ironmongery)

Butt hinges, Parliamentary hinges, Single acting Spring hinges, Double-acting Spring hinges, Rising hinges, Sliding door bolts, Tower bolts, Flush bolts, Handles, Door latches, Mortice latch and lock, Mortice latch, Rim latch, Floor door stopper, Hooks and eyes, Casement window fastner, Quadrant stays, Hasps and staple, Helical spring, etc.

These are made of brass, copper oxidised (brass), chromium plated (brass), Iron, copper oxidised (iron), anodised aluminium.

Timber shall be sawn in the direction of the grains.

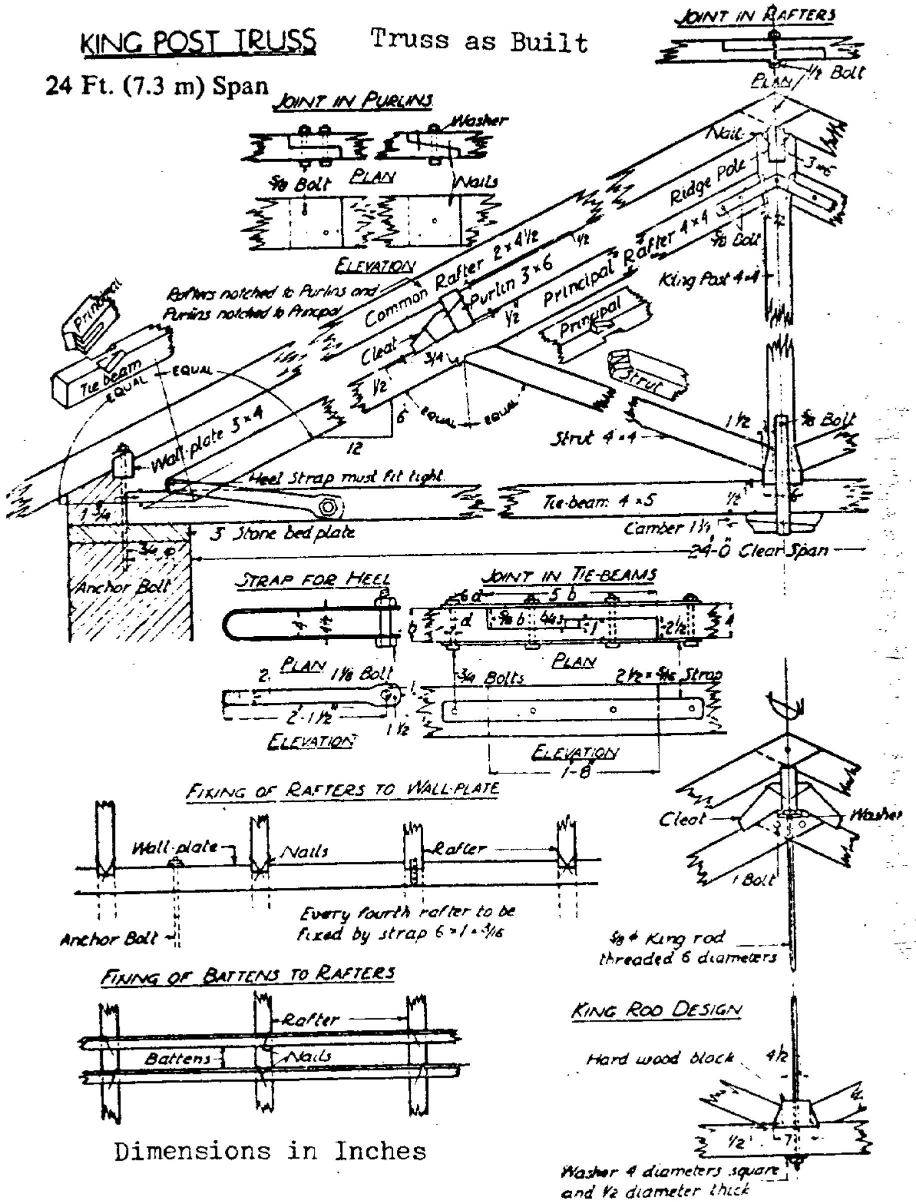
The joints shall be glued, framed, put together and pinned with hard-wood or bamboo pins 10 mm to 15 mm dia. after the members of the frame have been pressed together in a vice mechanism.



Methods of carrying roof load on cavity wall

KING POST TRUSS Truss as Built

24 Ft. (7.3 m) Span



Dimensions in Inches

SECTION 10

DESIGN OF SIMPLE STEEL STRUCTURES

(For Masonry, RCC and Timber Columns see under respective Sections)

Permissible Working Stresses or Safe Working Loads for Common Steel Structures

1. Direct stress on axial tension on the net area of the section— P_T

		kg/sq. cm
(*i*) Rolled I-beams and channels	...	1500
(*ii*) Plates, flats, bars, angles, tees, rounds, squares, etc.		
Up to and including 20 mm	...	1500
Over 20 mm	...	1420

2. Tensile and compressive stresses in bending— P_{BT} or P_{BC}

(*i*) Rolled I-beams and channels	...	1650
(*ii*) Plates, flats, bars, angles, tees, rounds, square, etc.		
Up to and inclnding 20 mm	...	1650
Over 20 mm	...	1575

3. Bending stress for slab bases, for all steels - P_B	...	1890
4. Bearing stresses on the net area of contact - P_B	...	1890
5. Max: shear stresses on any part of a member P_Q (For shear stress in beam webs, see under "Lateral Stability of Beams)	...	1100
6. Max: equivalent stress due to combined (co-existant) bending, bearing and shear stresses — P_E		
Up to and including 20 mm	...	2285
Over 20 mm	...	2205

7. Direct stress in axial compression on the gross sectional area of axially loaded compression members (columns and struts)—P_C See page 10/16

L_E/R	*kg/sq. cm*	L_E/R	*kg/sq. cm*	L_E/R	*kg/sq. cm*	L_E/R	*kg/sq. cm*
0	1250	70	1075	140	531	210	243
10	1246	80	1007	150	474	220	219
20	1239	90	928	160	423	230	199
30	1224	100	840	170	377	240	181
40	1203	110	753	180	336	250	166
50	1172	120	671	190	300	300	109
60	1130	130	597	200	270	350	76

L_E is the effective length of the member in mm and R the appropriate (mostly least) radius of gyration (in mm).

Notes :

(*i*) Net sectional area is the gross sectional area less deduction for holes.

(*ii*) An appreciable proportion of the present structural steel production in India is available on the market in the catagory of "untested steel". Working stresses for untested steel may be taken 1260 kg/sq. cm for tensile and compressive stresses in bending instead of 1650 kg/sq. cm.

and 800 kg/sq. cm in shear instead of 1100 kg/sq. cm. Old British manufactured steel obtained from dismantling old structures, should also be treated as untested steel. Higher stresses are allowed for 'high tensile structural steel' by about 30 per cent.

(*iii*) Permissible stress for rivets and bolts are given under "Riveting".

(*iv*) For properties of other steels and working stresses for wrought iron and cast iron see Section—"Properties and Uses of Metals"

(*vi*) When the effect of wind or sesmic load is taken into account, the permissible working stresses specified above may be exceeded by 33 per cent.

(*vii*) For stresses in plate giders see "Plate Girders" under Girder Bridges in Section 19.

DESIGN OF SIMPLE BEAMS

(See also Section 3)

The capacity of a beam to take loads or stresses is mainly dependent on : —

(*i*) Section Modulus for bending ; (*ii*) Moment of Inertia (I) for resistance to deflection : (*iii*) Cross-sectional area for tension and compression; (*iv*) Radii of Gyration for compression.

For a bending member, deeper and thinner sections are advantageous. Design of compression of flexural members depends on radii of gyration as well as cross-sectional area. The ratio of the effective length to the appropriate width of compression flange shall not exceed 50 except that for cantilever beams this ratio shall not exceed 25.

Channel sections may be used as single members or as parts of components of built-up members. Single channel sections may be used as beams or as framing members under conditions where channels have special advantage over the other sections, *viz.*, as purlins and runners, lintels, framing around large openings, lacings of heavy built-up coloumns.

Variable Loads or Reversal of Stresses—Varying from tension to compression :—

The maximum stresses due to tension and compression shall be determined and each increased by 50 per cent of the smaller of these two. The section should be capable of resisting either stress so increased. Any tension member, in which a reversal of direct stress due to loads other than wind or seismic forces would occur, should meet the stiffness requirements (slenderness ratio) for a compression member, as specified in the Table or simply : Equivalent dead load may be taken = max: load + variation.

Launhardt — Weyraunch formula :

$$\text{Working stress} = \frac{f}{1.5}\left(1 + \frac{\text{min: stress}}{2 \times \text{max: stress}}\right)$$

The figure obtained is multiplied by the stress usually allowed. f is the stress usually allowed.

If a structure member is repeatedly stressed above its elastic limit by alternate loading and unloading, the member can be brought to rupture at a stress much below the ultimate stress as determined by the ordinary strength test or even below the yield point. Rupture that occurs in this way is called *fatigue rupture*. The magnitude of the stress which can be borne is dependent upon the number of load fluctuations and on the mean static stress about which the load fluctuates.

For the Design of Tension Members only the net sectional area, after deducting the area of holes for rivets or bolts from the gross sectional area shall be counted. In making deductions for rivets and bolts less than 25 mm in diameter, the diameter of the hole shall be assumed to be 1.5 mm in excess of the nominal diameter of the rivet or bolt. If the diameter of the rivet or bolt is greater than 25 mm, the diameter of the hole shall be assumed to be 2 mm in excess of the nominal diameter of the rivet. (See Table at page 4M/13)

Angles are more suitable than flat bars as angles give more rigidity to the structure than flat bars. Angles can take reversal of stresses from tension to compression whereas flat bars are unfitted to carry a compression load.

The effective net area of a tension angle is taken as the area of the riveted leg minus its rivet hole (as expiained in the preceding para) and half the area of the outstanding leg (*i.e.*, non-riveted leg.)

Steel Beams Encased in Concrete. An I-beam may be solidly encased in 1:2:4 cement concrete when the depth does not exceed 75 cm. The minimum width of solid casing is equal to the width of the beam flange plus 100 mm and there is a concrete cover of not less than 50 mm over the surface of the flange. The casing is reinforced with stirrups of steel wire of at least 5 mm at not more than 150 mm pitch and so arranged as to pass through the centre of the covering of the edges and soffits of the lower flange.

The steel section shall be considered as carrying the entire load but allowance may be made for the effect of the concrete on the lateral stability of the compression flange. This allowance should be made for assuming for the purpose of determing the permissible stress in compression that the equivalent width of flange of the cased beam is, 0.8 (width of the steel flange + 100) mm. The permissible bending stress so determined shall not exceed 1.5 times that permitted for the uncased section.

COLUMNS AND STRUTS

General Definition of Terms

Column is a general term applied to vertical members supporting a load, the effective length of which exceeds 12 times its least radius of gyration. In the case of a rectangular section, the effective length of which exceeds three times its least lateral dimension.

Long Column. A column having a ratio of effective length to its least radius of gyration greater than 50. In the case of rectangular sections, it is a column having a ratio of effective length to least lateral dimension greater than 15.

Short Column. A column having a ration of effective length to its least radius of gyration not more than 50. In the case of rectangular sections, it is a column having a ratio of effective length to least lateral dimension not more than 15.

The term *strut* is also applied to any structural member under compression whose slenderness ratio is larger than 20. It is the smaller vertical or inclined compression member.

Effective Sectional Area. The gross sectional area shall be taken for all compression members connected by rivets, welds, tight fitted bolts and pins. The holes which are not filled with rivets, welds or tight fitting bolts shall be deducted. (Gross sectional area is the area of the cross-section as calculated from the specified size. Net sectional area is the gross sectional area less deduction for rivet and bolt holes.) The diameter of rivet and bolt holes for deduction is generally assumed to be 1.5 mm in excess of the normal diameter of the rivet or bolt.

Stanchions are columns generally carrying heavy loads.

Pillar is used for round columns.

Pier is used for bridges.

Post is usually of timber.

Pedestal is an upright compression member whose height does not exceed three times its least lateral dimension.

Effective lateral restraint. Restraint which will produce sufficient resistance in a plane perpendicular to the plane of bending to restrain a load, beam, or column from buckling to either side at its point of application. The term "restrained" refers to restraint against buckling due to an axial load.

Slenderness Ratio :

The strength of a column or strut depends on its slenderness ratio and the method by which the ends are fixed. Columns generally fail by buckling unless the length is very small.

$$\text{Slenderness ratio} = \frac{\text{effective length or unsupported length}}{\text{least radius of gyration}} = \frac{L_E}{R}$$

$$\text{Radius of gyration} = \sqrt{\frac{I}{A}} \quad \text{or} \quad I = AR^2$$

The measure of *stiffness* of a member is the ratio of its Moment of Inertia to its length. The most economical section is that which for the same area of cross section has the greatest radius of gyration.

Long columns tend to fail by bending in the direction of their least dimension (or the least radius of gyration), therefore, for economy of the material variation between the maximum and minimum radii of gyration about the two principal axis of the section and the value of A/R should be as small as possible. The best forms are then circular and square tubes and the least satisfactory form is a rectangular section. For this reason single sections of rolled-steel joists, angles or channels etc. are used for light columns and for heavy columns two or more sections are joined or extra plates are fixed to the ends of rolled joists. Methods for calculating moments of inertia and radius of gyration are given in Section 3.

Where singles angle are used as columns or struts either (*i*) only the gross area of the connected leg should be considered and strut designed as an axially-loaded member with permissible working stresses given in the tables following to compensate for the eccentricity of the end connections or, (*ii*) full cross-sectional area of the angle be considered and only 2/3rd of the permissible working stresses be taken. Effective length should be taken 0.8 of the length of the strut, centre to centre of the fastenings at each end. No deduction is made for rivet holes for calculations of radius of gyration and modulus of section.

Max: Slenderness Ratio of Compression Members

The ratio of effective length 'L_E' to the appropriate (generally the least) radius of gyration 'R' of any compression member shall not exceed the following values ;—

(*a*) For any member carrying loads resulting from dead weights and superimposed loads. 180

(*b*) For any member carrying loads resulting from wind forces only, provided the deformation of such member does not adversely affect the stress in any part of the structure. 250

(*c*) For ties in roof trusses or any other member subject to reversal of stresses from the action of wind suction. 350

Some engineers recommend that no steel column should have a value L_E/R more than 150 for main members and 200 for secondary members in compression.

Slenderness Ratio for Bridge Structures :

L_E/R should not exceed 100 for main members and 125 for bracings.

Euler's and Rankine's formulae are not now much in use, therefore, have been omitted.

Cast Iron Columns. No cast iron column should have an unsupported length greater than 80 times its least radius of gyration.

I-Section Steel Columns Encased in Concrete

If a steel column of I-section (or channels back-to-back) is solidly encased in cement concrete of 1:2:4 mix with a cover of at least 50 mm all round its surfaces and edges (so as to increase the width of the flange by 100 mm) and the casing is reinforced horizontally with binding wires of at least 5 mm diameter at 15 cm pitch in the form of stirrups so arranged as to pass through the centre of the covering of the edges and outer faces of the flanges, and supported by longitudinal spacing bars not less than four in number, the axial load can be increased to 2 times that which would be permitted on the uncased section. For calculating the permitted load, assume the radius of gyration 'R' of the column section about the axis of the plane of its web to be 0.2 (width of the steel flange + 100 mm). The radius of gyration about its other axis shall be taken as that of the uncased section. The slenderness ratio of the uncased section for its full length centre-to-centre of connections shall not, however, exceed 250. (The steel section is considered as carrying the entire load, but allowance is made for stiffening effect of the concrete).

If the concrete cover is more than 50 mm, it may be preferable to design the column as a composite column in which the concrete is permitted

to assist in carrying the load, as explained in Section 8—"Reinforced Concrete".

The minimum thickness of flange or web plates of a built-up column in a multi-storey building shall be not less than 10 mm. In all other cases, steel should be not less than 6 mm thick except in the case of standard rolled steel sctions.

In computing the allowable axial load on the encased strut the concrete shall be taken as assisting in carrying the load over its rectangular cross-section, any cover in excess of 75 mm from the over-all dimensions of the steel section of the encased strut being ignored. This cross-section of concrete shall be taken assisting in carrying the load on the basis of a stress equal to the allowable direct stress in axial compression Pc in the steel divided by 30 for ordinary steel (compressive stress ratio between steel and concrete.)

Effective Length of a compression member for the purpose of determining allowable axial stresses shall be assumed in accordance with the table following.

The actual length L of the strut is the distance measured between the centres of lateral supports. In the case of a column provided with a cap or base the point of lateral support is assumed to be in the plane of the top of the cap or the bottom of the base. In practice no strut is completely fixed or hinged at the ends and the half-fixed end conditions can be generally taken in design ; full theoretical fixidity is not attained in a stanchion as ordinarily used in a steel frame building. Stanchions fixed by anchor bolts at the bottom are considered partially fixed. The end of a stanchion can only be taken as fixed if it is fixed in two planes. Where it is fixed in one plane only as in the case of the stanchion supporting a gantry girder, it should be taken as fixed in direction but not in position.

Effective Length of Compression Members

Type of End Fixing	Effective length L_E of member
Both ends held in position and restrained in direction (fixed ends).	0.67 L
Both ends held in position and one end restrained in direction.	0.85 L
Both ends held in position but unrestrained in direction.	L
One end held in position and restrained in direction and the other end partially restrained in direction but not held in position.	1.0 L to 1.5 L according to the degree of restraint
One end held in position and restrained in direction but not restrained at the other end in position and direction.	2.0 L

When better end fixing can be obtained in one plane than in the other, arrange the column so that the lesser radius of gyration goes with the better end fixing.

End fixing affects the "effective length".

Stanchion Bases

The foot of every stanchion after riveting up complete with all gussets, cleats, etc., should be machined over the whole area so that the base plate is in effective contact with the whole area of the stanchion foot, and all joints should be close butted. When it can be assumed that the base slab (or plate) distributes the loading uniformly, the min: thickness of a steel rectangular slab can be found from the formula :—

$$T=\sqrt{\frac{3w}{P_B}\left(A^2-\frac{B^2}{4}\right)}$$

where

T = the slab thickness; A = the greater projection of the plate beyond the column; B = the lesser projection of the plate beyond the column; w = the pressure or loading on the underside of the base, and P_B = the permissible bending stress in slab bases—1890 kg/sq. cm.

In general practice the width of the base will vary from 2 to 3 times the width of the stanchion and the height of the gusset plates from 1.5 to 3 times the width of the stanchion. The base plate should not project more than 8 times its thickness to avoid shear and bending momement in the plate itself and for the overhanging portion acting as a cantilever. The number of rivets connecting the base plate to the stanchion should be sufficient to carry 2/3rd of the total load provided the remainder is transmitted by direct bearing. The size of the cap should also be as small as possible to prevent eccentricity of loading.

Stanchions supporting upper floor loads and roofs frequently require to be spliced. The joint should be made just above the girder connections (say 45 cm) at any storey level. The ends of the sections to be joined should be planed to obtain good contact, and sufficient rivets must be provided on each side of the joint which can always carry the bending moments and the axial loads when the two ends are not machined. Loads on the lower storey stanchions will be reduced as explained in the Section on "Foundations."

Masonry Footings under Stanchions

The thickness of the concrete should not be less than twice its projection (to avoid cantilever action) ; give 30 cm min: The thickness of the stone cap or template should not be less than 1/4th of the length of its side, or less than 1.5 times the projection from the base plate. Holes should be provided where necessary in stanchion bases for the escape of air when grouting is done.

Bedding of stanchion bases should not be carried out until the steelwork has been finally levelled, plumbed and connected together, the stanchions being supported meanwhile by steel wedges. In multi-storey buildings beddings must be postponed until a sufficient number of bottom lengths of pillars have been properly lined, levelled and plumbed and at least two floors of beams fixed in position.

Eccentric Loads

Columns with eccentric loadings have to be designed for combined bending and direct stresses. The max. compressive stress at the extreme fibre due to the bending actions (about each principal axis) shall be added to the axial loading.

Combined Stresses

Combined Bending and Axial Compression or

Combined Bending and Axial Tension

Members subjected to both bending and axial compression/or axial tension shall be so proportioned that the quantity.

$$\frac{f_C/\text{or } f_T}{P_C/\text{or } P_T}+\frac{f_{BC}/\text{or } f_{BT}}{P_{BC}/\text{or } P_{BT}} \text{ does not exceed unity}$$

where :

f_C/or f_T = the calculated average axial compressive stress/or axial tensile stress = load divided by area of member ;

P_C/or P_T = the permissible compressive stress/or axial tensile stress ;

f_{BC}/or f_{BT} = the calculated bending compressive stress/or tensile stress = the bending moment divided by the appropriate section modulus of the member.

P_{BC}/or P_{BT} = the permissible bending compressive stress/or tensile stress in bending.

When bending occurs about both axis of the member, f_{BC} and f_{BT} shall be taken as the sum of both the stresses.

Where an increase in allowable working stresses is specified, both P_C and P_{BC}, and both P_T and P_{BT} shall be the increased working stresses.

Gusseted Base (Based on IS: 800)

Thickness of gusset plate is assumed 16 mm. Size of gusset angle is assumed such that its vertical leg can accommodate two rivets in one vertical line and corresponding to that, leg length of the other leg is assumed in which one rivet can be provided. Thickness of cleat angle is kept approximately equal to the thickness of gusset plate. 150 × 115 mm angle is assumed to be used as cleat angles.

Fastenings having flush ends may be designed for 50 per cent load. Where ends of the column section and gusset plates are faced for complete bearing, the fastenings connecting them to the base plate should be sufficient to transmit all the forces to which the base is subjected.

A 22 mm nominal diameter is assumed for rivets to be used and rivet value and number of rivets required are computed. Number of rivets connecting the gusset plate and gusset angle is adopted equal to the number of rivets connecting the gusset plate and the column. The horizontal shear is assumed equal to the vertical shear.

Thickness of base plate is computed by equating the moment of resistant with the moment due to intensity of pressure at the underside of base plate at two sections. Out of these two points one is the vertical plane touching outer vertical plane passing through the axis of the column and second point being bisecting the width of the baseplate. Out of these, the section which required greater thickness is adopted. Thickness thus computed is redued by the thickness of the cleat angle. It is usual to make the base plates at least as thick as the gussets or angles.

In the above computations, base-plate is assumed to have supports at gusset plates and bending of plate takes place in one plane only. But actually bending of central portion occurs in two planes mutually at right angles. It is also having support from the web of the column. Computation of thickness with the said assumption is on safer side.

Bed plate should rest on a carefully levelled bed block of stone or concrete. To take up any inequalities a piece of 40 kg/sq m lead or felt should be interposed between the plate and the stone or alternatively, it may be grouted with neat cement.

The diameters of the holding down (or anchor) bolts are governed by the shear and tension at the base of the column, from the breaking and tractive forces it has to withstand. Washers are always used with foundation bolts, as the holes have a clearance of about 3 to 6 mm over the bolt diameter. The washer helps in spinning the hole and in distributing the load on to the nut.

A hollow shaft is stronger as a column than a solid shaft for the same cross-sectional area.

Lacing or Latticing

Lattice bars are provided to reduce the slenderness ratio of a long strut, and for connecting together two or more sections. Lacing may be single or double.

The thickness of flat lacing bars shall be not less than 1/40 of their length for single lacing, and 1/60 of their length for double lacing.

Width to be not less than three times the diameter of the connecting rivet. The angle of inclination of single or double lacing bars shall not be less than 40 deg. nor more than 70 deg. to the axis of the member.

Maximum spacing of lacing bars shall be such that the slenderness ratio L_E/R (min:) of the components of the strut between consecutive connections is not greater than 50 or 0.7 of the min: slenderness ratio of the strut as a whole, whichever is less. L_E is the distance between the centres of connection of the lattice bars to each component.

The ratio L_E/R of the lacing bars shall not exceed 145. The effective length of lacing bars for the determination of permissible stress shall be taken as the length between the inner end rivets of the bar for single lacing, and 0.7 of this length for double lacing.

As far as practicable, the lacing system should not be varied throughout the length of the strut.

Where welded lacing bars overlap the main members, the amount of lap shall be not less than four times the thickness of the bar or the member, whichever is the lesser.

Ties Plates or Battens are provided in laced compression members at the ends of lacing systems and at points where the systems are interrupted. The thickness of battens or tie plates shall be not less than 1/50 of their length. (This does not apply when angles, channels or I-secttions are used.) The battens should be placed opposite each other at each end of the member.

For battened struts, the effective length L_E should be inceased by 10 per cent.

Columns continued through several floors. Roughly there are two methods of calculations employed. By far the most common one is to assume that for vertical loading the floor beams are simply supported by the columns; in other words, they have simple pin connections at their ends. Since the beam reactions are usually assumed to be situated about 5 cm away from the column face there is a bending moment, at floor level, on the stanchion equal to the beam reaction multiplied by the total lever arm to the gravity axis of the column. When dealing with wind stresses, however, the horizontal members which may now form part of the wind bracing system, are assumed to have fixed ends and the restraint values of the beam to the column cleats and their fastenings are fully allowed for in the calculations; although the same cleats and fastenings have been considered as simple pins when carrying their vertical loads.

The other method of calculating the stresses caused by vertical loads in multistoreyed building frames is to make due allowance for the restrains or fixities provided by the rigid or semi-rigid end connections between beams and columns. This method, although admittedly the correct one, has not found many adherents in practice because of the very heavy and laborious work involved in the calculations.

4. RIVETING

Tables for Rivets are given in Section 4M.

Rivets are superior to bolts for being hammered up hot they contract on cooling and cause a frictional resistance between the plates, thus adding considerably to the rigidity of the work.

Riveted joints are classified as: (*i*) Lap joints; (*ii*) Single cover; (*iii*) Double cover. Lap and single cover joints are not to be recommended for connecting tension plates, here the rivets are in single shear and in the double cover joints, the rivets are in double shear.

The rivets in a lap joint are in single and in a butt joint in double shear. Lap and single cover joints are not recommended for connecting tension members.

Permissible Working Stresses in kg/sq. cm for Rivets, Bolts & Tension Rods

	In Tension	*In Shear*	*In Bearing*
Power driven shop rivets	785	1025	2360
Power driven field rivets	630	945	2125
Hand driven rivets	630	785	1575
Bolts over 38 mm dia.	1260		
Bolts 20 mm up to 38 mm dia.	945		
Bolts less than 20 mm dia.	785		
Tight fitting turned bolts		1025	2360
Black bolts		865	2045
Tension rods	1260		

It will be seen that permissible working stress for hand driven rivets is only 4/5th of the machine driven rivets.

Strength of riveted connections. The strength of a riveted joint or splice is governed by the smallest of the following three items:

(*a*) The shearing strength of the rivets,

(*b*) The bearing strength of the rivets on the riveted material, and

(*c*) The tensile strength of the riveted parts at the weakest section.

For tension, take gross area of rivets, net area of bolts and tension rods. For shear, take gross area of rivets and bolts. For rivets and bolts in double shear, the area to be assumed shall be twice the area defined. For bearing, take gross diameter of rivets and bolts.

Riveted joints may fail in four ways :

(*i*) Shearing—single or double shear :

$Fs = fs\,\pi\,r^2$ ... single shear
$= fs\,2\pi\,r^2$ ... double shear | r is radius of rivet

(*ii*) Bearing or crushing : This is usually due to the plates being too thin or the dia. of the rivets being too small :

$F_B = f_B\,D\,T$

(*iii*) Tearing of the plates : Plates connected by riveting are weakened by the holes drilled for the rivets.

$F_T = f_T\,(B \times T) - (N \times D)$

(*iv*) Bursting of the plates : This failure is likely to occur when the rivet hole is drilled or punched too near to the edge of the plate.

Strength of joints and weakest section (in tension) :

$= T\,(B - D)\,f_T$

$$\text{Efficiency of joints} = \frac{\text{least strength of joint}}{\text{strength of solid plate}}$$

The loss of strength is ordinarily—for chain riveted joints, 15%; for double riveted, 30% ;for single riveted joints, 44%, as compared with the strength of the plate unpunched.

Cover plates for joints : Thickness :

In butt joints—cover plate on both sides, the thickness being each 5/8th that of the main plate to be jointed. In single cover, the cover plate is 1.25T or 1.12T.

Where several thickness of plates are to be jointed, they may with advantage be all jointed between one pair of cover plates, arranging the several layers to break joint one with another, and leaving between every two joints a space containing a sufficient number of rivets to take up the pull of one layer of plates.

Diameter of Rivets :

For equal strength in bearing and shear, diameter of rivets may be as follows :

In a single shear joint $D = 2.55\,T$
In a double shear joint $D = 1.27\,T$

The nominal diameter of a rivet is the diameter of the cold material.

The gross diameter of a rivet is the diameter of the rivet hole.

The gross area of a rivet is the cross-sectional area of the rivet hole. (See Table at page 4M/13.)

The gross diameter of a bolt is the nominal diameter of the bolt.

The gross area of a bolt is the nominal area of the bolt.

The net area of a bolt is the area at the root of the thread.

Pitch of Rivets

Minimum pitch. The distance between centres of rivet holes should be not less then 2.5 times the diameter of the hole.

Maximum pitch. The distance between centres of any two adjacent rivets connecting together elements of compression or tension member shall not exceed 32T or 300 mm, where T is the thickness of thinner outside plate.

The distance between centres of two adjacent rivets, in a line lying in the direction of stress, shall not exceed 16T or 200 mm, whichever is less in tension members; and 12T or 200 mm, whichever is less in compression members. In the case of compression members in which forces are transferred through butting faces this distance shall not exceed 4.5 times the diameter of the rivets for a distance from the butting faces equal to 1.5 times the width of the member.

The distance between centres of any two consecutive rivets in line adacent and parallel to an edge of an outside plate shall not exceed 100 mm+4T, or 200 mm, whichever is less in compressin or tension members.

When rivets are staggered at equal intervals and the gauge does not exceed 75 mm, the distances specified above, between centres of rivets, may be increased by 50 per cent.

The lengths of rivet shanks are taken as follow :—

For snap heads $L = T + T/8 + 1.5D$
For countersunk heads $L = T + T/8 + D$

Deduction for rivet holes :

In tension members an area of T(D+3 mm) for hand driven rivets and T(D—1.5 mm) for machine driven rivets should be deducted for each hole. No deductions are made for members in compression or shear.

F_S = strength of one rivet in shear,
F_B = strength of plate in bearing,
F_T = strength of plate in tearing,
f_S = allowable shear stress in rivets,
f_B = allowable bearing stress in plate,
f_T = allowable tensible stress in plate,
D = dia. of rivet,
T = thickness of thinnest plate transmitting the full load,
P = pitch of rivets,
N = no. of rivets per side of joint,
B = breadth of the plates.

When two or more parts are connected together, a line of rivets or bolts shall be provided at a distance of not more than 37 mm+4T from the nearest edge, where "T" is the thickness in mm of the thinner outside plate. In the case of work not exposed to weather this may be increased to 12T.

For *countersunk heads*, one-half of the depth of the countersinking shall be neglected in calculating the length of the rivet in bearing. For rivets in tension with countersunk heads the tensile value shall be reduced by 33.3 per cent. No deduction need be made in shear.

Where the grip of rivets carrying calculated loads exceeds 6 times the diameter of the holes, the number of rivets required by normal calculation shall be increased by not less than 1 per cent for each additional 1.6 mm of grip; but grip shall not exceed 8 times the diameter of the holes.

Where bolts have to be used instead of rivets, these shall be provided with washers not less than 6 mm thick. Where there is risk of nuts becoming loose due to vibrations or reversal of stresses, these shall be secured from slackening by the use of lock-nuts, spring washers, cross-cutting or hammering down of threads into deformation (blurred).

Edge Distance of Holes

Dia. of Hole	Distance to Sheared or Hand Flame Cut Edge	Distance to Rolled, Machine Flame Cut, Sawn or Planed Edge
mm	mm	mm
13.5 and below	19	17
15.5	25	22
17.5	29	25
19.5-21.5	32	29
23.5	38	32
25.0	44	38
29.5	51	44
32.0-35.0	57	51

Making Holes for Rivets

Holes for rivets or bolts are made in three ways—Drilling; Punching; Punching and Reaming. As far as possible all holes should be drilled and to the full size of the hole. Punching slightly injures the plates immediately about the hole and there is also possibility of inaccuracies. Punching is generally done for small holes and in thin plates. In the third method which is called sub-punching, the holes are punched about 3 mm less in diameter and reamered (enlarged with a drill) thereafter to the required size. Holes in purlins, side sheetings, runners, packing plates, lacing bars, and holes in light framing with the exception of joint holes, may be punched full size. Holes are made larger than the diameter (shank) of the rivet—5.3 mm for 5 mm dia. of rivet, 6.3 for 6 mm, 8.4 for 8 mm, and 10.4 for 10 mm diameter. (See also Table at page 4M/13.)

When the number of plates or sections to be riveted together exceeds three, or when their total thickness is 90 mm or more, holes shall be drilled

or reamered in position after the members are assembled. Before riveting or bolting up finally, the members shall be taken apart after drilling and all burrs removed.

The parts assembled for riveting shall be in close contact with each other and shall be firmly bolted and rigidly held together while riveting. Riveting shall be done by hydraulic or pneumatic process. Where however, such facilities are not available, hand riveting may be done. Each rivet should completely fill the hole and form a head of the standard dimension. For rivets of all diameters of 16 mm and upward, the diameter of the rivet before being heated shall not be more than 1.5 mm less than the diameter of the hole it is intendent to fill. The heads of the rivets should grip the members firmly. Caulking and recupping should not be done. Rivets of diameter less than 10 mm may be driven cold.

Rivets shall be properly heated to red heat for the full length of the shank and quickly driven in the hole. Sparkling or burnt rivets must not be used. Much over-heating is detected by the burnt scaly appearance of the surface of the shank ; rivets should not be over 20 minutes in the fire. Riveting gangs consist of a holder-up, two riveters and one or two boys for heating and supporting riveters. Riveting hammers vary from 1 to 3 kg in weight, the holding up hammer or "dolly" from 5 to 18 kg ; 90 to 100 rivets can be put in by one gang in one working day of 8 hours.

Bolts, nuts and washers shall be thoroughly cleaned and dipped in double boiled linseed oil before use.

Before riveting is commenced, every alternate hole in the joint plate should be tightly service bolted so as to ensure tight riveting. The service bolt should be retightened frequently as the riveting proceeds.

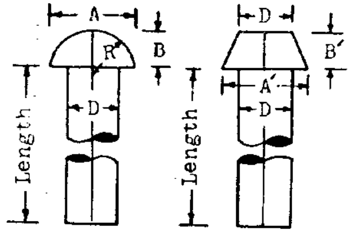
Loose rivets can be detected by holding a finger on one head and tapping the other with a small hammer of about 1/4 kg weight. All rivets should be tested. Strike rivet head with several good blows of the hammer to see if it can be "floated" or moved up and down. Slack or loose rivet gives a hollow or dull sound and a jar. When a loose rivet is removed, it may loosen adjoining rivets ; for cutting out of loose rivets the head or point must be divided into four by means of two cross cuts before it is cute off sideways and rivet shank punched out.

The surface of all joints must be thoroughly scraped and then painted with a thick coat of red lead and boiled linseed oil before joining up which should be done while the paint is still wet. All field rivets, bolts, nuts, washers, etc., are to be dipped into boiling linseed oil.

Where the heads and nuts bear on timber, square washers having the length of each side not less than three diameters of the bolt and the thickness not less than one-quater of the diameter should be provided. Steel or wrought iron tapered washers should also be provided for all heads and nuts bearing on bevelled surfaces.

For permanent bolted connections, washers not less than 6 mm thick shall be used under the heads and nuts.

Screwed ends and eyes of tie-rods should not be welded on but should be formed from the solid bar. They should not be upset but stayed up in a die and afterwards annealed.



Standard Proportions:

A = 1.75 D
B = 0.75 D
R = 0.885 D

A′ = 1.6 D
B′ = 0.7 D

5. *Minimum Thickness of Steel Metal in Structural Works :*

(*i*) Where the steel is directly exposed to weather and is fully accessible for cleaning and repainting, the thickness shall be not less than 6 mm ; and where it is not accessible for cleaning and repainting, the thickness shall be not less than 8 mm. These provisions do not apply to the webs of rolled steel sections or to packings.

(*ii*) Where the steel is not directly exposed to weather, the thickness of steel in main members shall be not less than 6 mm ; and the thickness of secondary members not less than 4.5 mm.

6. *Weights of Steel Structure :*

Average weight of a structure = nominal weight of the sections + 10 per cent for cleats, rivets, bolts, etc.

(10 per cent is taken for wastage for bolts, nuts, washers, and rivets, etc., for field works.)

Average weight of a beam = weight of section including cleats + 2.5 per cent for rivets.

Average weight of stanchions = ditto. + 5 per cent.

Average weight of plate girder = ditto. + 10 per cent.

Weight of steel stairs 0.91 m wide, industrial type = 167 kg/m run.

Allowable Stresses in Axial Compression/Values in kg/sq. cm

Type of Steel

Le/R	Mild Steel	Hy-Ten 31	Hy-Ten 36	Hy-Ten 42
0	1250	1537	1785	2083
20	1239	1520	1762	2055
40	1203	1464	1685	1946
60	1130	1347	1522	1710
80	1007	1155	1255	1352
100	840	920	960	1005

See page 10/2

SECTION 11
ROOFS

(For Timber Roofs, see under "Timber Structures", Section 9)

Thatch Roofing : (Page 11/21) Thatches generally consist of reeds, palymrah, rice straw or coconut leaf (most generally used). If thatch is treated with bituminised ('cut-back') non-erodable mud plaster, this not only renders the thatch fire-resistant but water resistant as well and enhances its life from one to about 8 years. If copper sulphate dissolved in water is sprayed on both sides of the thatch or thatch is dipped in the solution and dried it will prolong its lifespan from one year to four years The chemical fungicide is neither washed by rain nor degraded by sunlight and makes the thatch also fire-proof. An untreated thatch catches fire in as short a time as 30 seconds.

1. GLOSSARY OF ROOFING TERMS

Barge Board. Wooden planks fixed on the gable end of a (sloping) roof. They connect the ends of ridge, purlins and wall plates.

Bay. The distance between two adjacent trusses.

Cleats. Small block of wood, angle iron or steel which are fixed on principal rafters of trusses to support the purlins to prevent them from sliding down.

Eaves. The lower edge of an inclined roof.

Eaves Board. A wooden board fixed to the feet of the common rafter at the eaves.

Gable. The triangular upper part of a wall at the end of the ridge (in a sloping roof construction).

Hip. The outer angle (more than 180 deg.) formed by the inclined ridge between two intersecting roof slopes.

Flashing. A strip of lead or copper sheet let into the joints of a wall or worked in the roof slates or tiles around dormers, chimneys, etc., to lap over gutters so as to exclude water from the junction between a roof covering and anotner part of the structure, to prevent leakage.

Panel. Distance between two adjacent purlins in a roof truss.

Pitch. The ratio of rise to span—it is expressed either as a ratio or in degrees. *Pitch of Roof*—is the angle of inclination of the roof with the horizontal plane. *Pitched Roof*—a roof of which the pitch is greater than 10 deg. to the horizontal—a sloping roof. There are various types of pitched roof.

Types of roofs are also defined in section 9.

A *Lean-to-roof* is a verandah roof. A *Pent-roof* is a roof with slope on one side only.

Purlins. Horizontal members laid on principal rafters, spanning between the adjacent trusses.

Verge. Edge of sheets, slates or tiles which project beyond the gable end of a sloped roof - that on the horizontal portion being called *eaves.*

Verge board—often corrupted into barg board ; the board under the verge or gables.

Weather Boarding. Board lapped over each other to prevent rain and sun from passing through.

Weathering A slight fall or slope on the top of cornices, window-sills, etc. to throw off the rain.

Sag tie. A member connecting centre of the tie with the ridge to avoid sagging of the tie.

2. LOADINGS FOR DESIGN OF ROOFS

The following loads shall be taken for design of roofs—

(i) *Dead Loads* due to the weight of the roofing materials, such as beams, slabs, trusses, purlins, battens, roofing sheets, ceiling or any other fixtures.

(ii) *Superimposed Loads.* Live loads, snow loads and wind loads, etc. (All loads other than dead loads)

Live Loads (IS Code)

(a) Flat or sloping roofs with slopes up to 10 deg., where access is provided other than that necessary for cleaning and repairs (i.e., a roof which is to be used for sitting or sleeping), the minimum live load shall be 150 kg/sq. metre measured on plan, subject to a minimum load of 375 kg per metre width of the roof slab uniformly distributed over the span, and 900 kg uniformly distributed over the span in case of all beams. (That is, min. load should be 375 kg per metre width of slab for all spans up to 2.5 metres. The load of 150 kg/sq. metre becomes operative at spans of 2.5 metres).

Flat roofs which are to be used for incidental social gatherings, the minimum live load shall be taken at 400 kg/sq. metre or more as the case may be. (See also under "Superimposed Loads on Floors at page 6/50).

(b) For roofs where access is not provided except for repairs, the minimum live load shall be half of that recommended in (a) above.

(c) For sloping roofs with slope greater than 10 deg., live load shall be 75 kg/sq. metre less one kg/sq. metre for every deg. increase in slope over 10 deg. up to and including 20 deg , and 2 kg/sq. metre for every deg. increase in slope over 20 deg., subject to a min: of 40 kg/sq. metre.

Any member directly supporting a roof covering shall be able to sustain a point load of 90 kg at the most unfavourable position on the member.

Snow loads

Snow weighs 125 to 190 kg/cu. metre when fresh and 250 to 800 kg/cu. metre when compacted. Snow loads where applicable shall be taken at 2.0 kg/sq. metre per cm depth of snow anticipated. Where snow is likely to occur, a minimum value of 125 kg/sq. metre of the horizontal covered surface shall be taken for all slopes up to 20 deg., this load may be reduced one kg for every two deg. of slope above 20 deg. No allowance for snow need be made for roofs slooping 1 to 1 or steeper.

(A roof subjected to snow load is designed for actual anticipated load due to snow plus that provided for in (b) above.)

3. SPECIFICATIONS FOR DIFFERENT TYPES OF FLAT TERRACED ROOFS

Several specifications are adopted for the construction of flat terraced roofs according to the climate and availability of construction materials. The most common are : (A) RCC or RB slabs, (B) (i) One or two layers of brick tiles or common building bricks, (ii) Stone slabs, (iii) Wooden planking, etc., all supported on wooden or RC battens, or Tee irons. (C) Jack arches supported on RS joists. Lime concrete is put over Jack arches with tiles on top. Mud is also used instead of lime concrete over tiles and slabs after giving one or two coats of bitumen for water-proofing, and finished with tiles again on top.

RCC and RB roofs are described in Section 8.

Safe Loads on Kail-wood Planking

Span in metres	Thickness in millimetres: 25	40	50	65	75
	Safe Loads in kg/sq. metre				
0.46 (1.5′)	2000	—			
0.61 (2′)	900	3000			
0.76 (2.5′)	450	1500	3600	—	
0.91 (3′)	250	900	2000	4100	—
1.07 (3.5′)	—	550	1500	2600	4500
1.22 (4′)		370	900	1750	3000
1.37 (4.5′)		250	650	1150	2000
1.52 (5′)		—	450	900	1500
1.67 (5.5′)			350	650	1150
1.83 (6′)			250	500	900

The table applies to good class ordinary timber.

For loads on other kinds of timber multiply by the following factors :—Deodar—1.33 ; Sal—1.83 ; Chir—1.42.

Diameter and Spacing of Ballies for
500 kg/sq. metre of Roof Load

Span in metres	Deodar Ballies—Dia.		Sal Ballies—Dia.	
	Spacing 30 cm c/c	Spacing 45 cm c/c	Spacing 30 cm c/c	Spacing 45 cm c/c
1.2 (4′)	75 mm	90 mm	70 mm	75 mm
1.5 (5′)	90 ,,	100 ,,	80 ,,	80 ,,
1.8 (6′)	100 ,,	115 ,,	95 ,,	100 ,,
2.1 (7′)	115 ,,	130 ,,	110 ,,	120 ,,
2.4 (8′)	120 ,,	140 ,,	115 ,,	125 ,,
2.7 (9′)	135 ,,	150 ,,	125 ,,	140 ,,
3.0 (10′)	140 ,,	165 ,,	135 ,,	145 ,,
3.3 (11′)	160 ,,	170 ,,	150 ,,	165 ,,
3.7 (12′)	165 ,,	185 ,,	160 ,,	180 ,,

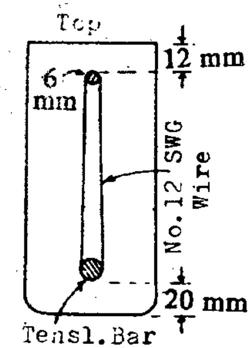
Wooden & RCC Battens for Flat Roofs
for Roof Loads of 500 kg/sq. metre, 30 cm centre to centre

Clear span in metres	1.5 (5′)	1.8 (6′)	2.1 (7′)	2.4 (8′)	2.7 (9′)	3.0 (10′)
Wooden Battens (good class ordinary timber)						
Size in mm	50×95	60×110	65×120	75×135	80×145	85×160
RCC Battens (1 : 2 : 4)						
Concrete section in mm	50×125	60×140	65×150	70×165	75×190	90×200
Tensile reinforcement Bar dia.—one no.	10 mm	10 mm	10 mm	12 mm	12 mm	14 mm

One 6 mm dia. bar is provided on the compression side of every batten and both the tensile and compression bars tied with No. 12 SWG wire.

Where brick tiles are supported on steel Tees, the Tees may be of size 50×50 mm laid 30 cm apart, on a span of 1.25 metres, between joists.

RCC Battens for Flat Roofs



Lime Concrete Terracing. Before the terracing is laid, the surface of the slab should be thoroughly cleaned with a rag soaked in kerosene oil and treated with two coats of hot bitumen.

Lime concrete of the following mix shall be laid, in a single layer 7.5 cm to 12 cm thick :

(i) 2 lime : 2 surkhi : 7 brick ballast 25 mm gauge (for Bombay and Calcutta regions)

(ii) 2 lime : 1.5 surkhi : 2.5 brick ballast 20 mm gauge (for Madras region)

(iii) 1 lime : 2 surkhi and 25 mm gauge brick ballast (for Delhi, Punjab, U.P., and Rajasthan regions). Lime is slaked lime (lime putty).

The volume of wet mortar used is 50 per cent of stacked volume of brick aggregate. Brick aggregate shall be thoroughly soaked in water for not less than 6 hours before mixing. For water-proofing, 12 kg of bar soap and 4 kg of alum dissolved in water shall be added to each cu. metre of concrete. Thickness of compacted concrete shall not be less than 75 mm.

During compaction by hand beating, the surface shall be liberally sprinkled with lime water and a mixture of gur (jaggery) and boiled solution of bael fruit in the proportion of 3 kg of gur and 1.5 kg of bael fruit boiled in 100 litres of water.

Porous brick ballast or showing signs of saltpetre shall not be used. Min : slope of roof is 1 : 48.

Cement-Cinder Concrete Terracing—Cinder should be of 12.5 mm nominal size, i.e., passing through IS sieve 212-micron and 70 to 90 per cent retained on IS sieve 100-micron. Cinder obtained from brick kiln shall not be used. Cement and cinder are mixed in the ratio of 1 : 15 so that the cinders are uniformly and completely coated with neat cement. The cinder shall be soaked in water before mixing. The finished surface shall be cured for at least 7 days. Min : slope of roof is 1 : 48 (CWPD Specifications)

Painting of Roof Slabs or Tiles with Hot Bitumen

Bitumen may be applied in two coats on a dry and clean surface and blinded with dry clean sand when the bitumen is still hot. The quantity of bitumen used is 120 kg. for the first coat and 70 kg. for the second coat and that of sand for blinding 0.6 cu. metre per 100 sq. metres of surface. The bitumen coat shall be continued along the parapet walls up to the drip course. Mud (or earth) filling or plastering shall be done when the bitumen has cooled down and is no longer tacky. Coal tar can also be used in place of bitumen (maxphalte). Primer to be applied to the roof surface if necessary.

Mud mortar shall be made from good 'brick-earth' free from grass, roots, gravel, kankar, etc. The earth shall not contain any efflorescent salts. The earth shall be reduced to a fine powder and mixed with water to form stiff mortar. Old 'bhusa' at the rate of 7 to 8 kg per cubic metre of puddled clay shall be mixed with it and allowed to weather for 4 days. Small quantity of sand (10 to 20 per cent) may be added with the earth to reduce cracking while drying. The mud mortar is laid to a thickness of 7.5 to 10 cm over the tiles and given the requisite slope towards the outlets.

Mud plaster. After laying the mud phuska, the surface is given a coat of mud plaster 25 mm., or two coats of 15 mm and 10 mm thick and the plaster allowed to dry and crack. The earth for mud plaster is reduced to fine powder and mixed with water in a pit, adding fibrous reinforcing materials such as chopped straw (bhusa), not longer than 2 cm, in the proportion of 30 to 35 kg per cu. metre of earth. The mixture is allowed to mature for 7 days during which period it is worked up from time to time so as to make it into a homogeneous mass.

After the mud plaster has dried, the surface is given a coat of gobri (cow dung) leaping, with equal quantities of fresh gobar and finely sieved clay. The quantity of gobar used in gobri leaping shall not be less than 0.03 cu. metre per 100 sq. metres of plaster area. The cow-dung should be free grass, straws, seeds, etc. A coat of plaster with 3 parts mud and 1 part gobri can also be used.

Laying of brick tiles over mud mortar. After the gobri leaping has dried, tiles are laid using the minimum amount of plain mud mortar (without bhusa) as bedding. Care should be taken to see that mud mortar does not rise into the vertical joints of the tiles more than 12 mm. The thickness of the joints in the tiles should not be less than 6 mm or more than 12 mm in width. After the tiles are well set and the bedding mortar has dried, joints of the tiles shall be grouted with cement mortar slurry 1 : 3. Tiles should be inserted into parapet walls 4 cm. The brick tile work shall be allowed to dry for 24 hours before grouting of joints. Cement mortar may be mixed with 5 per cent of crude oil by weight of cement for water-proofing. The tiles or bricks shall be soaked in water for at least two hours before use.

Two layers of tiles 30 mm thick each are laid on battens covered with 25 mm layer of mud plaster, and 100 mm layer of earth. The tiles are laid in lime mortar after they have been well soaked in thick white lime water. The second layer of tiles is laid over 12 mm bed of lime mortar over the first layer breaking joints in both directions with the layer underneath. If only one layer of tiles is to be laid, the thickness of the tiles is 50 mm instead of 30 mm. The spacing of battens is generally 30 cm centre to centre. The necessary slope in the roof should be formed by sloping the beams or the battens.

Repairs to Porous Roofs

0.7 kg of bar soap, 0.3 kg of alum, each dissolved separately in 10 litres of boiling water, brushed on the roof alternatively with lime water (white-wash), 10 hours to elapse between each coat.

Cement slurry pumped into the cracks, or cement grouting (preferably mixed with some water-proofing compound) over the surface will stop leakage.

Water-proofing Porous Flat Roofs

Lime Concrete Roofs :

One or two coats of oxide of iron paint should render a porous roof water-proof. Coal tar or bitumen can also be used if not otherwise objectionable.

Water-proof Mastic Cement for Cracks :

1 part red lead 4 parts ground lime 5 parts sharp sand	*Or*	1 part red lead 5 parts whiting 10 parts sharp sand	mixed with boiled oil

Asphalt felting and Polythene (PVC) films are now used for waterproofing roofs and also Shalimar Tarfelt. See also under "Stabilized Soils" in Section 7.

Repairing of Cracks in Old Terraced Roofs

Cracks should be cleaned out and formed into V-shaped cuts at least 6 mm wide at the top. When repairing is indicated in cement, the cracks shall be thoroughly flooded with water, water allowed to soak in, and grouted with cement and sand slurry (1 : 3). When repairing is indicated with bituminous mixture the cracks shall be dried and cleaned and filled solidly with hot mixture of bitumen and sand (1 : 1) by weight.

Roof Slopes. For porous type of roofs, give a slope of 1 in 20 in heavy rainfall regions and 1 in 36 in comparatively dry locations. Where the roofs are made of RCC or RB slabs duly water-proofed or with a layer of bitumen or tar with 7.5 cm of earth on top, a slope of 1 in 40 and 1 in 60 respectively will be sufficient. The necessary slopes may be provided by either two-way or four-way from the centre of the roof outwards. Any slope exceeding 40 mm should be taken up by the height of masonry walls, or beams and battens.

Sandstone Slabs Roofing

The stone slabs shall not absorb more than 10 per cent of their dry weight of water after 24 hours immersion. The width of the slabs shall not be less than 40 cm. The max : spacing of wooden or RCC battens supporting the slabs shall not exceed the figures given below for various thickness of slabs :—

Thickness of slab—mm	40	45	50
Max : spacing c/c—cm	52.5	60	68

The size of the battens should be calculated for the weight of the whole roof.

The bearing of the slabs over the battens shall not be less than 30 mm. Where a batten supports a slab from one side only, the bearing of such slab shall be for full width of the batten. The stone slabs shall be jointed in cement mortar 1 · 4 after having been soaked in water for two bours.

Madras Terraced Roofing

Madras terrace consists of a course of thoroughly burnt terrace bricks of 6" × 3" × 1" (15.2 cm × 7.6 cm × 2.5 cm) size laid in lime mortar 1 : 1.5 mix, the bricks laid on edge diagonally across the joists. After 10 days or when this brick-on-edge course has set, a layer of bricks jelly lime concrete, 4" (10.2 cm) in thickness, is laid over and well beaten to 3" (7.6 cm) thickness with wooden hand beaters. After 6 days or after the concrete laid has hardened, 3 courses of flat tiles of 6" × 6" × 0.5" (15.2 cm × 15.2 cm × 1.3 cm) size are laid in lime mortar 1 : 1.5 mix, diagonally and breaking joints. The top surface and the ceiling are then plastered with 3 coats of lime mortar 1 : 1.5 mix and rubbed to a polished surface.

Alternatively, instead of 3 courses of flat tiles in lime mortar 1 : 1.5 mix with 3 coats of plastering on top in lime mortar 1 : 1.5 mix, 2 courses of flat tiles may be laid in cement mortar 1 : 3 mix, mixed with 10% crude oil by weight of cement, the first course diagonally and the second course square to the joists breaking joints. The ceiling is plastered in this case with one coat of cement mortar 1 : 3 mix, 15 mm in thickness, instead of 3 coats of plastering in lime mortar 1 : 1.5 mix.

JACK ARCH ROOFING

Size of Rolled Steel Beams for Different Spans

Size of Beam mm	Weight kg/m	Max : clear span in metres up to which the beam may be used with a spacing of : 0.91 m	1.22 m	1.37 m	1.52 m	1.68 m	1.83 m
150 × 80	14.2	2.8 m	2.6 m	2.4 m	2.2 m	2.1 m	2.0 m
175 × 90	16.7	3.0 ,,	2.8 ,,	2.7 ,,	2.6 ,,	2.4 ,,	2.1 ,,
200 × 100	19.8	3.4 ,,	3.2 ,,	3.0 ,,	2.9 ,,	2.7 ,,	2.6 ,,
225 × 100	23.5	4.2 ,,	4.0 ,,	3.8 ,,	3.6 ,,	3.3 ,,	3.2 ,,
200 × 100	25.4	4.4 ,,	4.2 ,,	4.0 ,,	3.8 ,,	3.6 ,,	3.4 ,,
225 × 110	31.2	4.9 ,,	4.7 ,,	4.5 ,,	4.3 ,,	4.0 ,,	3.8 ,,
275 × 140	30.0	5.6 ,,	5.3 ,,	5.0 ,,	4.7 ,,	4.4 ,,	4.2 ,,
325 × 165	43.1	6.3 ,,	6.0 ,,	5.8 ,,	5.5 ,,	5.2 ,,	5.0 ,,
350 × 165	49.5	6.7 ,,	6.4 ,,	6.1 ,,	5.9 ,,	5.7 ,,	5.5 ,,
350 × 140	52.4	7.7 ,,	7.3 ,,	6.9 ,,	6.6 ,,	6.2 ,,	5.9 ,,
400 × 165	56.9	7.8 ,,	7.3 ,,	7.1 ,,	6.8 ,,	6.5 ,,	6.2 ,,
1—75 lbs. Rail		3.2 m	3.0 m	2.9 m	2.8 m	2.7 m	2.6 m

The above spans are calculated for a total load of (dead+live) 750 kg/sq. metre. Area of concrete filling below the crown of Jack arches, for estimating, is taken = span × depth from top of crown to top of springing × 1/3.

For Jack arch roofs, RCC precast beams can also be used instead of rolled steel beams. Such beams can be made of a splayed shape somewhat like the top of a pillar of an arched bridge for the ends of the arch to rest on the sides of the beam. Jack arches are supported either on the lower flanges of the joist or on top of the upper flange. Steel beams may also be encased in cement concrete.

For design of arches and tie rods, see under "Arches" in Section 7 "Mansonry Structures".

Construction of Khurras (Spouts for roof rainwater outlets) :

The khurras should be constructed before the brick masonry work in parapet wall is taken up, and it should be 45 cm × 45 cm and formed of cement concrete 1 : 2 : 4. A PVC sheet of size 1 m × 1m × 400 micron should be laid under the khurra and then cement concrete laid over it 3 cm thick with its top surface lower than the level of the adjoining roof surface by not less than :

(*a*) 20 mm for roof surface finished with lime concrete terracing ; 50 mm for roof surface finished with mud pushta with brick tile covering ; and 70 mm for roof surface finished with lime concrete terracing covered with brick tiles. See also 11/39

The concrete should be laid to a size greater than the stipulated size of the khurra in such a way that the adjoining terracing should overlap the concrete on its three edges by not less than 7.5 cm. The concrete to slope uniformly from the edges to the outlet. The size of the finished outlet opening should be 10 cm wide by 20 cm high.

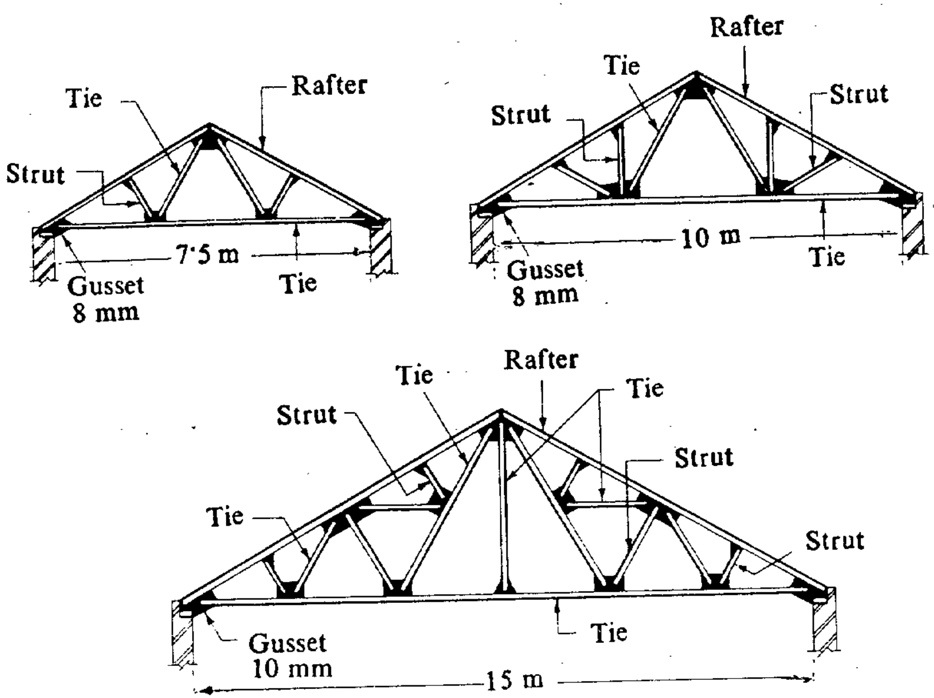
In case where the rainwater is to be disposed off through rainwater pipes, iron gratings should be provided at the outlet of overall size 20 cm × 25 cm, with an outer frame of 15 mm × 3 mm mild steel flat to which 4 mild steel bars of 10 mm dia. are welded in a vertical direction keeping equal clear spacing of 2.5 cm.

Cement Concrete Gola. A chase of 75 mm wide by 75 mm deep should be cut in the parapet wall at the junction of mud pushta or lime concrete with parapet wall and filled with cement concrete 1 : 2 : 4 with a slope of 1 : 0.75 and the exposed surface of gola plastered with cement mortar 1 3. Expansion joint at every crossing and 3.5 to 4.5 metres provided and filled with bitumen filler prepared by mixing bitumen, cement and coarse sand in the ratio of 80 kg of hot bitumen, 1 kg of cement, and 0.25 cu. m of coarse sand.See also 11/39

4. INCLINED OR SLOPING ROOFS - Pitched roofs

The overall cost of a roof structure is made up of the following main items :

(*a*) Sheeting or Roof Coverings ; (*b*) Trusses ; (*c*) Purlins and Tie Runners ; (*d*) Bracings ; (*e*) Supporting Columns. With different spacings of trusses the cost of items (*a*) and (*d*) do not vary much, but cost of (*b*) and (*c*) vary.



Wind Pressure

Description of Wind	Velocity in km/hour	
Strong breeze	45	
Moderate gale	60	The cyclone storm which ripped through the coastal areas of Orissa on 3.6.82 had a gale wind speed of 160 km/hr. With gusts reaching up to 240 km/hr.
Strong gale	85	
Whole gale (trees uprooted, structures damaged)	100	
Storm (wide spread damage)	115	
Hurricane	125	
Violent hurricane	160	

Highest wind velocity in Delhi has been 159 km/hour in 1965.

Wind speed is measured by an instrument called 'anemometer'.

The relationship between wind pressure and velocity is : $p = KV^2$, where p is the pressure expressed in kg/sq. metre, V is the velocity in km/hr., and K is a coefficient the value of which depends upon a number of factors such as wind speed, shape of the structure.

When calculating wind pressure on tall structures, the total pressure to be adopted should be worked out in successive slabs of 3 metres in height for the full height of the structure.

No wind pressure on buildings need be allowed if the height of the building is less than three times its effective width, and where adequate stiffening is provided by cross walls or floors. In costal areas, however, where the height exceeds twice the effective width, wind pressure should be considered.

Wind pressure near the ground is less due to friction and is more on small areas than on large areas. It is not advisable to design for excessive wind loads at heavy extra cost ; exceptional wind loads will be taken by the factor of safety. Increase the wind pressure on small areas by 1/3. Structures designed with the wind loads mentioned in the above table should have factor of safety of 2 against overturning.

Wind Pressure on Inclined Roofs

Duchemin's formula :

$$Pn = P \times \frac{2 \sin \theta}{1 + \sin^2 \theta} \; ; \; Pv = Pn \times \cos \theta \; ; \; Ph = Pn \times \sin \theta.$$

P = wind pressure in kg/sq. m of vertical surface,

Pn = corresponding normal pressure per sq./m of roof surface,

Pv = vertical component of Pn, Ph = horizontal component of Pn,

θ = angle of roof slope with the horizontal.

Pressure of wind on roofs : (based on the above formulae).

Slope of Roof θ	1 in 3 18°—26′	1 in 2.5 21°—48′	1 in 2 26°—34′	30°	1 in 1.5 33°—41	1 in 1 45°	60°	75°
Pn=P×	.575	.653	.745	.800	.848	.943	.990	.999
Pv=P×	.545	.606	.667	.693	.706	.667	.495	.259
Ph=P×	.182	.242	.333	.400	.471	.667	.857	.965

Wind Pressure in kg/sq. metre

P	100	120	150	200	250	300
Pn	75	89	112	149	186	224
Pv	67	80	97	133	167	200
Ph	33	40	48	67	83	100

Wind Pressures in kg/sq. metre at Different Heights for Basic Pressure of 100 kg/sq. metre

Height in metres	Wind pressure in kg/sq.m	Height in metres	Wind pressure in kg./sq m	Height in metres	Wind pressure in kg./sq m
30	100	50	111	100	127
35	104	60	115	120	132
40	105	70	118	150	138
45	110	80	122	—	—

For basic pressures of 150 or 200, increase the figures accordingly.

(Wind pressures for the design of bridges given by IRC is different)

A maximum wind pressure of 200 kg/sq. metre perpendicular to the direction of the wind is considered for violent storms along the Indian coasts and about 100 kg/sq. metre for the central planes and protected locations ; 150 kg/sq. m for coastal areas, exposed positions and hills (that will cover for the weight of workmen and 60 cm of snow).

For the design of roof coverings, battens, purlins and rafters, the above loads (both superimposed and dead) should be taken acting normal on the member.

Dead load is taken for the full truss and the wind or live load is considered acting on one side of the truss at a time. (Either wind load or live load is taken, whichever is greater, both loads are not taken together). On the leeward side there will be suction force effect which may be reversal of the stresses and to safeguard against the same tension members should be designed to resist equivalent compression as well. The max. stresses in the members obtained by ignoring suction on the leeward side would appear to be on the safe side.

When wind load stress is included in the total stress, the working stress may be increased by 33.3 per cent. This increase should not be made

for structures in which the effect of wind is a predominating factor in the design.

Uplift. Design roof members of open sheds, factories, hangers, armouries, etc., which have large open interiors for a minimum uplift of 120 kg/sq. metre.

Dead Loads

Weights of roofing materials are given in Section 9.

Weight of steel roof trusses for slope of 1 in 2.

For steel trusses in general

(*i*) $W=0.40(4.42\sqrt{L}+L)$; (*ii*) $w=L/3+5$ kg

w=Weight of truss/sq. m for a spacing of 4 m—for other spacings of trusses, proportional values may be taken.

W=Weight of truss in kg/sq. m of horizontal covered area ;

L=Span of truss in metres.

The following self weight of the truss may be taken approximately for estimating purposes :—

Span of truss in metres	7.5	9.0	12.0	15.0	18.0	24.5	
Approx. weight in kg./sq.m of horizontal area	10	12	13	15	20	25	spacing 3 metres
	7.0	7.5	8	11	15	16·5	spacing 4.5 metres

To the above weights add weights of bearing plates, anchor bolts, anchor plates and purlins, etc.

Corrugated Galv. Iron Sheet Roofing

Ordinarily 22 gauge (0·80 mm thick) sheets are used for common roofs. These roofs are not generally laid at a slope flatter than 1 in 5. The normal slope adopted is 1 vertical to 2 horizontal.

Wooden or steel purlins, as per design, are fixed over the principal rafters. The spacing of purlins should be according to the standard lengths of sheets to be used to avoid cutting, but shall not be spaced at more than the following distances for different thicknesses of CGI sheets :

Thickness of CGI Sheets	Spacings of Purlins
0.63 mm—24 gauge	1.60 metres
0.80 mm—22 ,,	1.80 ,,
1.00 mm—20 ,,	2.00 ,,
1.25 mm—18 ,,	2.40 ,,
1.60 mm—16 ,,	2.80 ,,

The sheets shall be laid with a min: lap of 15 cm at the ends and two ridges of corrugations at each side. This min: end lap of 15 cm shall apply to slopes of 1 veritical to 2 horizontal and steeper slopes. For flatter slopes the min: end lap shall be 20 cm. The min: lap of sheets with ridges, hips and valleys shall be 20 cm. Commence by laying the bottom sheets and work up towards the ridge.

Sheets shall not generally be built into gables and parapets ; they shall be bent up along their side edges close to the wall and the junction shall be protected by suitable flashing or by a projecting drip course, the latter to cover the junction by at least 7·5 cm.

Sheets shall be fixed to the purlins or other roof members such as hips or valley rafters with galv. J or L hook bolts and nuts, 8 mm dia. with bitumen and GI 'limpet' (dome) washers. The bolts shall be sufficiently long so that after fixing they project above the top of their nuts not less than 12 mm. The grip of J or L hook bolts on the side of the purlin shall not be less than 25 mm. There shall be minimum of three hook bolts placed at the ridges of corrugations in each sheet on every purlin and their spacing shall not exceed 30 cm. Coach screws shall not be used for fixing sheets to purlins.

Where slopes of roofs are less than 1 vertical to 2.5 horizontal, sheets shall be joined together at the side laps by galv. iron seam bolts and nuts 25×6 mm size, each bolt with a bitumen and GI limpet washer. As the overlap at the sides extends to two corrugations, these bolts shall be placed zig zag over the two overlapping corrugations, so that the ends of the overlapping sheets shall be drawn tightly to each other. The spacing of these seam bolts shall not exceed 60 cm along each of the staggered rows.

The top sufaces of purlins should be painted before sheets are fixed. Embedded portions of wooden purlins should be coal-tarred two coats.

Deodar wood shall not be used in contact with galvanized sheets as it has destructive effect on zinc (coating). Galvanizing also quickly disappears near sea and chemical works.

For painting galv. iron roofs see under "Paints and Painting." GI roofs are found about 5 degrees cooler when painted silver grey than when left of their natural colour.

Holes in CGI sheets shall be drilled and not punched in the ridges of the corrugations, and shall preferably be made on the ground. The holes in the washers shall be the exact diameter of the hook bolts or seam bolts. The nuts shall be tightened from above. Sheets can be secured to wood framing by means of glav. cone headed screws, drive screws or jagged nails, 65 to 75 mm long, at intervals not exceeding 30 cm on every bearer.

Wind ties shall be of 40×6 mm flat iron section fixed just above at the eaves ends of the sheets in continuous lengths, bolted down every 1.2 metres. The fixing shall be done with the same hook bolts which

secure the sheets to the purlins. Slot holes are cut in the wind ties to allow for the expansion and contraction due to temperature changes.

Projection of eaves is generally 40 to 60 cm horizontally.

Ridges and Hips of GCI roofs shall be covered with ridge and hip sections of plain 20 gauge GI sheets with a min: of 22.5 cm lap on either side over the GCI sheets. The end laps of the ridges and hips and between ridges and hips shall also be not less than 22.5 cm. The ridges and hips shall be of 60 cm overall width plain GI sheets, 0.6 mm or 0.8 mm, properly bent to shape. Ridges, hips and valley sheets shall be fixed to the purlins and other members below with the same 8 mm dia. GI hook bolts and nuts and bitumen and GI limpet washers which fix the sheets to the purlins. At least one of the fixing bolts shall pass through the end laps of ridges and hips on either side. If this is not possible extra hook bolts shall be provided. The end laps of ridges and hips shall be joined together by GI seam bolts 25×6 mm size each with a bitumen and GI washer. There shall be at least two such bolts in each end lap.

Valleys shall be 90 cm wide overall, plain GI sheet 1.6 mm thick, bent to shape and fixed. They shall lap with the CGI sheets not less than 22.5 cm width on either side. The end laps of valleys shall also be not less than 22.5 cm. Valley sheets shall be laid over 25 mm thick wooden boarding if so required. Same thickness of sheets may be used as for eaves gutters, but prefer 18 gauge.

Flashings shall be of plain GI sheets of 38 cm overall width, 1.25 mm or 1.00 mm thick, bent to shape and fixed. They shall lap not less than 15 cm over the roofing sheets. The end laps between flashing pieces shall not be less than 22.5 cm. Only one edge of flashing is fixed and the other end is left free for expansion. Against a wall, the flashing will be slightly tilted and turned up 15 cm, the upper edge being left free and covered by an apron overlapping it by about 10 cm, the upper edge of which should be tucked 5 cm deep into the joint of the masonry and filled with mastic or cement mortar.

Eaves Gutters shall be made of plain GI sheets 18 to 22 gauge (1.25 mm to 0.80 mm thick) bent semi-circular with edges rounded, and twice the diameter of the down pipe. The longitudinal edges shall be turned back to the extent of 12 mm and beaten to form a rounded edge. The ends of the sheets at junctions of pieces shall be hooked into each other and beaten flush to avoid leakage.

Gutters are fixed 2.5 cm below the edge of the roof and laid with a min: fall of 2.5 cm in 3 metres ; the fall should be steeper for the portions which are not straight. Gutters shall be supported on and fixed to flat iron brackets of size 40×3 mm or 25×6 mm and bent to shape. Not less than one bracket should be fixed to each 1.5 metres length, and one bracket to each angle nozzle piece, drop end, etc.

For connection to down-take pipes, a drop end or funnel shapped connecting piece shall be made of GI sheet and riveted to the gutter, the other end tailing into the socket of the rain-water pipe.

Gutters and flashings shall be so arranged as to give free play for expansion or contraction in any direction.

Down-take Pipes. If of GI sheeting, may be of 18,20 or 22 gauge. Usual size is 7.5 to 15 cm dia. Brackets are fixed about 1.8 metres apart. Pipes may be provided about 6 metres apart. See below.

Rain-water Pipes for Roof Drainage

A 75 mm dia. down pipe will serve a roof area of about 35 sq. metres and, a 100 mm dia. down pipe about 65 sq. metres in dry regions (for flat roofs), and about 45 sq. metres in heavy rainfall regions (sloping roofs). Where no down-pipe is fixed, the outlet pipe or channel should project 30 cm outside the wall (for flat roofs). Min : size of a rain-water pipe is 75 mm dia. Drain outlet to be not less than 100 mm. Min: Weight of cast iron rain-water pipes:

Description	Nominal bore of pipe in mm 75	100	150
Min :	Weight	of one pipe	in kg
Pipe of 1.8 m overall length	11.0	14.0	26.0
Pipe of 1.5 m effective length	9.5	12.5	23.5

CURVED ROOFS

Curved roofs can be made up to 9 metres span with 6 metres radius of 18 gauge (1.25 mm thick) corrugated iron sheets without trusses with :—

Tie rods——20 mm dia. or 50 mm × 10 mm flats, 2 metres apart.

King rods——20 mm dia. Eaves stiffened by L irons 60 × 60 × 6 mm, and top strengthened by an L iron 45 × 45 × 6 mm running along the under surface of corrugated iron sheets on each side of the king-rods.

Asbestos Cement (AC) Corrugated Sheet Roofing

AC sheets may be corrugated or semi-corrugated, and are available in lengths of 1.5, 1.75, 2.0, 2.25, 2.5, 2.75 and 3.0 metres, and thickness of 6 mm and 7 mm.

Dimensions of Corrugated AC Sheets

Pitch of Corrugations mm	Depth of Corrugations mm	Overall Width mm	Effective Width mm
146	48	1050	1010

Dimensions of Semi-Corrugated AC Sheets

338	45	1100	1014

Fixing Manual is available with the manufacturers, that should be obtained while ordering sheets.

AC sheets are laid in more or less the same manner as CGI sheets. The weight of laid roofing is about 158 to 170/ten sq. metres, while CGI sheets roofing weighs about 130 kg/ten sq. metres. Weight of AC sheets is 12 to 15.6 kg/sq. metre, (inclusive of fixing accessories).

Maximum spacing of purlins under the sheets shall be 1.6 metres in the case of 7 mm thick sheets and 1.4 metres in the case of 6 mm thick sheets. Ridge purlins shall be fixed 7.5 cm to 11.5 cm from the apex of the roof.

AC roofing sheets shall be capable of withstanding a distributed test load of 390 kg/sq. metre.

The sheets should be laid with smooth side upwards, with side lap of half corrugation (45 mm) and min : end lap or 15 cm. The free overhang at the eaves, measured as the length of sheet from its lower end to the centre of the bolt holes of the sheets shall not exceed 30 cm for 6 mm thick sheets and 40 cm for 7 mm thick sheets.

Galv. iron 'J' type hook or cranked hook bolt and nut with galv. iron washer and bitumen washer shall be used for fixing sheets on angle iron purlins, and 'L' type hook bolts and nuts shall be used for fixing sheets on RS joists, timber or concrete purlins. Galv. iron coach screws with galv. iron flat washers and bitumen washers shall be used for fixing sheet on timber purlins. Bolt and coach screws shall not be less than 8 mm in diameter. The length of 'J' bolt or cranked bolt shall be 75 mm longer than the depth of the purlin for single sheet fixing and 90 mm longer where two sheets overlap or where ridge or other accessories are to be fixed. (The grip of the hook bolt on the side of the purlin shall not be less than 25 mm.)

Wind ties to be 40 × 6 mm flat iron section as described for CGI sheeting.

Asphaltic corrugated roofing sheets could be used in places where the temperature does not go beyond 43°C.

Asbestos Cement Semi-Corrugated or Trafford Sheet Roofing

Specifications for laying are similar to those of fully corrugated sheets except that the side laps provided are one corrugation (9 cm) instead of half corrugation, the left hand small corrugation of each sheet being covered by the right hand large corrugation of the next sheet. These sheets are laid from right to left starting at the eaves. For fixing, along each line of purlin there will be a hook bolt in every vertical side lap corrugation and at the two verges and there will be an additional hook bolt through one of the two intermediate corrugations on each sheet. When sheets are supported on intermediate purlins as in the case of lengths over 1.75 metres for 7 mm thick sheets and over 1.5 metres for 6 mm thick sheets, fixing accessories are required on the intermediate purlins, through each side lap and the verges only.

To avoid fracture, sheets must not be rigidly fixed and screws must not be too tight. A hammer must never be used. AC sheets are brittle and break in handling.

The nuts or screws should be screwed lightly at first and then tightened when a dozen or so sheets have been laid. Expansion joints are provided at every 45 metres (or 45 sheets) length of the roof.

AC roof cannot take heavy stresses, therefore, care should be taken while fixing purlins not to exceed the safe spans. Additional joists should be fixed where excessive loads are likely to be put on, such as for repairs of ventilators and chimneys. Cat ladders (also called duck ladders) or roof boards should be used for fixing sheets or for repairs so that no damage is done to the sheets when the workmen walk over the roof.

AC sheets are laid from left to right starting at the eaves. AC sheets are cut with a woodsaw. Holes for hook bolts shall be drilled (and not punched) through the ridges of the corrugations while the sheets are on the roof, in their correct positions. The diameter of holes shall be 1.5 to 2 mm larger than the diameter of the fixing bolts. No hole shall be drilled nearer than 40 mm to any edge of a sheet or an accessory.

The minimum length of square head coach screws for timber purlins shall be 120 mm. Galv. iron flat washers are generally 25 mm in diameter, and 1.6 mm thick, and bitumen washers 35 mm in dia. and 1.5 mm thick.

Each hook bolt shall have a bitumen washer and a galv. iron washer placed over the sheet before the nut is screwed down from above. On each purlin there shall be one hook bolt on the crown adjacent to the side lap on either side. Bitumen washers are put under the galv. iron washers (lead limpet washers are not used).

Where it is proposed to use AC sheets, particulars and specifications should be obtained from the manufacturing firms and work designed and executed accordingly. They have published a "Fixing Manual".

Slate Roofs

The pitch of the roof is kept according to the size of the slates as follows :

60 cm × 30 cm......22 deg. 40 cm × 20 cm... . 33 deg
50 cm × 25 cm......27 deg.

The lap should not be less than 7.5 cm. Only slating nails of copper or of non-rusting compositions should be used. The battens should not be less than 40 mm × 40 mm.

All slates should butt close to each other with the rough side uppermost. Every nail should be covered by the covering slate except in the eaves course.

The top edge of the slates on each side of the ridge should rest on the ridge plate the top of which should be splayed to the roof slope. On apex formed by the edge of the slates a roll not less than 7.5 cm in dia. and made of 1:2:4 cement concrete should be formed with its centre coinciding with the apex formed by the slates. Where lead ridging is to be used, the slates should butt against the ridge plate the top of which will be flush with the top of the slates ; lead tacks 50 mm wide and 38 cm long will be nailed across the ridge at 60 cm centres. A 7.5 cm dia. ridge roll will then be secured to the ridge plate by double pointed nails. Over the ridge will be dressed the lead covering resting 15 cm on the slates on each side and held down by turning up the lead tacks.

Eternit Slate Roof Covering

The slates are fixed over wooden battens 40 mm × 25 mm nailed 235 mm apart centre to centre, the rafters being spaced 60 cm apart centre to centre. The slates are of three sizes, the main slate is 400 mm × 400 mm out of which any size can be cut. Overlap is about 70 mm. The first two layers at the eaves are fastened by galv. iron wire or copper nails and the chief slates by nails and copper cramps. (The cramps are copper discs with coppe pin. The disk is slipped under the edges of the slates at points of contact and the pin passed through and turned down over the three, binding all together).

About 85 slates cover 10 sq.m of area. These slates are generally made in three colours—grey, black and red. Eternit slate roof covering is not affected by weather, it is fire resisting, light and cool.

Allahabad Tile Roofing on Wooden Battens

This is double tiling or single tiling. Wooden battens of size 30 × 45 mm or 25 × 50 mm are fixed with nails over the common rafters at 30 cm centre to centre. The lowest (eaves) battens are thicker and bigger, of size 45 × 60 mm, than the battens above so that the slope of the tiles from ridge to the eaves is continuous. The spacing of common rafters shall not be more than 90 cm.

The minimum length of battens to be used should be such that the battens cover at least three spacings of rafters and the joints in battens come over the rafters and not in-between. The top surfaces of battens on which the tiles are to be placed shall be coal-tarred two coats.

A dry tile should not absorb more than one-sixth of its weight when immersed in water for one hour.

Single tiling consists of a layer of flat trough shaped tiles 30 × 37.5 to 40 cm in size, laid on batten, the edge of every two adjacent tiles being covered with a half-round tile 15 × 35 to 36 cm in size slightly tapering towards one end for covering the side joints.

Eaves tiles are laid in lime mortar. Screwing eaves tiles to battens is done with 50 mm iron screws with washers, provided one in each tile after drilling holes. Unless special eaves tiles with closed ends are used the ends of each row of semi-hexagonal shall be stopped with mortar for half their lengths.

All ridges and hips shall be laid in lime mortar. Tiles to come in contact with mortar shall be soaked in water mixed with cowdung for at least six hours before laying.

Valleys are made of GI sheets.

The roof slope shall not be less than 1 in 3 or more than 1 in 2.

Double tiling consists of two layers, the bottom layer consisting of a layer of flat tiles laid on battens and the side joints of every two adjacent flat tiles is covered by semi-hexagonal tiles. Over these semi-hexagonal tiles is laid an upper layer of flat tiles and the adjacent edges of every two of these flat tiles covered with half-round tiles.

Eaves and Barge Boards

The barge board should be about 50 to 65 mm higher than the eaves board for a single layer and about 125 to 150 mm higher than for a double layer of tiles so as to cover the end row of tiling.

Manglore Tiling

The roof should not be pitched at a slope of less then 1 in 3 or more than 1 in 2. Each tile of size about 40 cm × 24 cm covers 32 cm × 21 cm and allowing for laps, about 160 tiles are required for each 10 sq. m of roofing. 1000 tiles weigh about 2.5 tonnes. (Sizes of tiles differ ; exact size should be fixed locally). Each ridge tile is about 41 cm in length.

The tiles are laid breaking joint, *i.e.*, the left channel of the upper tile should lie in the right of that below, and should fit properly one to another, the "catches" resting fully against battens, 50 mm × 25 mm or 45 mm × 25 mm fixed 32 cm centre to centre to the upper surface of the rafters, or when planking is used, battens are 25 mm × 25 mm and exactly parallel to the eaves. The lowest batten, that nearest to the eaves, should be fixed about 25 cm from the one immediately above, and should have double the ordinary thickness. Special tiles are used for ridges, hips and valleys ; all these and tiles at gable ends should be set in cement mortar. Tiles to be set in cement should be immersed in water for four hours before laying. Tiles must not absorb more than one-sixth of their weight when immersed in water for one hour. In exposed situations and at all gable ends, eaves and places where the tiles are not readily accessible, they should be secured to the battens by No. 18 gauge galv. soft iron wire passed through the holes provided for the purpose in the underside of the tiles.

A tile should have a breaking strength of 90 to 100 kg for class AA tiles and 68 to 70 kg for class A tiles applied at centre of span when supported on battens at 34 cm centres.

For **Nanital Pattern Roofing** the roof coverings are made from 24 gauge, and rolls and ridges from 22 gauge galv. iron plain sheeting. The sheets are laid on wooden boarding with battens at side joints of sheets covered with rolls.

Insulation Against Hot Weather of Corrugated Iron or Asbestos Roofs

Fixing Mangalore or Country Tiles Over CGI Sheet Roofing

CGI sheets are covered with tiles for insulation of heat.

Pitch of the roof should be about 2 : 9

Sloping wooden battens 40 mm × 40 mm are laid over the ridges of the corrugations 46 cm to 54 cm centre to centre securely screwed through the sheets into the purlins below. Between battens the corrugated sheet should be fastened down to the purlins by two 75 mm screws. The horizontal battens 50 mm × 12 mm for country tiles and 25 mm × 20 mm for Mangalore tiles should be laid in the usual manner. When country tiles are laid they should be protected at the eaves by wind ties of flat iron 40 mm × 3 mm secured by 10 mm bolts to the corrugated sheets below.

Ridges and hips can be of the tiles laid in mortar or of GI sheets or zinc sheets laid as detailed before. Battens may not be used in the case of country round tiles.

Put on the top of the sheets a layer of about 7.5 to 10 cm of paddy husk mixed in lime mortar (1 lime, 2.5 to 3 of sand) and coat it with a layer of rich lime plaster. Paddy husk to be free from rice grains, otherwise these sprout and produce cracks. Mix 1 lime mortar to 2 of paddy husk by volume ; wet the paddy husk for a day and mix lime mortar while husk is wet. Lay it on the roof uniformly and tamp gently with a trowel, leaving about 7.5 cm of the roof sheets from the edge free. Finish with about 10 to 12 mm lime plaster. Where paddy husk is not available, groundnut husk or any other suitable vegetable husk may be used.

Grass Roof Ideal Insulation. People in extremely hot and extremely cold climates have used grass roofs in Scandinavia and Africa for centuries. The roof is clad in plastic sheeting and 7 to 10 cm of turf cut on the spot. The plastic sheeting prevents the roof from slipping and rain from seeping through too fast. The water collected works as a storage heater, the air between the blades of grass insulates the roofing from the influence of outside temperature. The grass roof provides natural air conditioning.

Thatch Roofing See page 11/1

Thatch is laid 15 cm thick in two layers. 25 mm dia. bamboos are placed 22.5 cm apart, or 40 dia. bamboos 30 cm apart, tied to the purlins. Split bamboo jaffri 15 cm mesh is laid on the top of the bamboo rafters and firmly tied to the purlins and rafters with string. Operation of laying the thatch begins at the eaves working upwards, tied firmly to the bamboo jafri. When the eaves row of poolas has been laid, a bamboo split into half is laid on top at about 30 cm removed from the eaves line and parallel to it. This bamboo split is tied down pressing the straw under it. Bamboos shall be dipped in crude oil before use.

The second and subsequent rows of poolas is laid in exactly the same manner tying each poola to the supports and securing each row with split bamboo fillets. Each row of fillets is placed at 30 cm intervals and concealed by the lower edge of poolas directly above. The process of laying is repeated working upwards towards the ridge.

The slope of thatch roof is limited between 30 deg. and 70 deg. 45 deg. slope is common.

Plastering both sides of the thatch with 2 per cent bitumeu stabilized mud will make it fire proof.

Materials required for 10 sq. metres of roof surface : Grass poolas (bundles)—166 nos., Bamboos—40 nos., Rope—5 kg., Chattai (Mat)—12 sq. metres ; 2 thachers and 2 mazdoors are required.

5. GENERAL PRACTICAL CONSIDERATIONS FOR MAKING AND FIXING TRUSSES

Numbers of panels in a truss should be so designed that the principal rafter is not longer than 3 metres between any struts. Panel points on rafters are usually kept at about 2-metre centres.

Spacing of Trusses

For different spans of trusses there is an optimum spacing of trusses which will give the minimum total cost of the structure. To achieve maximum economy trusses should be provided at optimum spacing. The cost of the supporting columns or pillars reduces with increase in spacing of the trusses as the number of columns required will be less with the same covered area. For most of the trusses, the most economical spacing, taking the above factors into consideration, will be between 3.5 metres to 4.5 metres. For spans 7.5 metres to 15 metres, 3.5 metres has been found to be the optimum spacing. Wider spacing results in heavy purlins and uneconomical sizes of members. Spacings may be increased for light roofs with CGI sheets up to 5 metres. With AC sheets, 3.7 metres spacing is most economical.

Proportions of Roof Inclinations

Proportion of height to half span		Proportion of length of Rafter to height	Proportion of length of Rafter to half span
1/1	45°—0′	1.41	1.41
1/1.5	33°—41′	1.80	1.20
$1/\sqrt{3}$	30°—0′	2.00	1.15
1/2	26°—34′	2.24	1.12
1/2.5	21°—48′	2.69	1.08
1/3	18°—26′	3.16	1.05
1/4	14°—2′	4.13	1.03
1/5	11°—20′	5.09	1.02

Rise of Trusses. The rise of a steel truss is generally kept 1/4th to 1/5th of the span, but it depends mostly on the roof coverings and climatic conditions. Regions subject to heavy rains need steeper slopes. (See Section on "Timber Structure" under Roof Slopes). Pitch of roof on West Coast is usually 1/3.

Minimum Size of Truss Members : (IS Code)

Angles—50×30×6 mm or 50×50×5 mm

Flats—50×6 mm ; Rods—16 mm dia.

Min : thickness of gusset plates shall be 6 mm.

Min : size of Rivets—16 mm.

Min : number of Rivets at ends—2.

Max : Slenderness ratio of compression members—180.

For compression members, when double angles are used, they shall be unequal angles with longer legs outstanding, and when single angles are used they shall be equal angles. Double angles are placed back to back and between which gussets are fixed.

Angle sections are most commonly used for truss members as angles effectively resist both compression and tension stresses, and can be conveniently attached. Angles ensure stiffness and rigidity of frame work and minimize bending and distortion during handling and transport. Flats and rounds will not resist compression and tend to become buckled. However, for small sections in tension, flats may be used.

When the stresses are known the sizes of the members can be worked out ; a tie being given such an area that the net area of the tie (the area of the bar minus the area of the rivet or bolt in it) × the working stress (see page 10/2) equals the total stress carried ; and a strut being given such an area that the area of the strut × the working stress obtained from buckling formula (Section 10) is equal to the total stress carried. Use tables given in Section 4M.

Small trusses are fabricated (riveted or bolted together) at the works and transported to the site. Large trusses are fabricated in parts at the works and assembled together on the job. Trusses are erected by a crane (sheer legs) and connected by holding—down bolts to the building.

Gusset Plates at the shoes, ridges and junctions of heel and tie and upper tie will be 10 mm thick and for other joints 6 mm thick. For big spans say 15 metres, thickness of the gusset should be increased to 12 mm. All gussets are single.

Rivets shall be in the standards gauge lines of angles as shown at page 4 /23. Generally 16 mm dia. rivets are used for spans less than 7 metres and 20 mm dia. for bigger spans, at 75 mm centres—the pitch of the rivets shall not be less than 3 times the dia. of the rivet. Maximum pitch is 15 cm for compression members and 20 cm for tension members. The minimum distance from the centre of the rivet to edge of plates or end of angles shall not be less than 25 mm.

All joints are designed for shop rivets except joints at apex of truss, and one-third points of bottom tie, portions of which are likely to be field-riveted in cases where the trusses cannot be transported in one piece.

A saving in weight of about 10 to 13 per cent can be obtained by the use of welding instead of riveting although there may not be a corresponding saving in cost.

Weights of Bracings in the planes of the upper and lower chords will vary from 2.5 to 5 kg/sq. metre of the area covered.

Thickness of Walls or Piers in cement mortar for supporting steel trusses

Span 4m to 7m	—2 bricks	See also under "Timber Structure" at page 9/14
,, 8m to 10m	—2.5 ,,	
,, 12m	—3 ,,	

Check for "slenderness ratio" (see page 7/11)

Bearings. Ends which are to be fixed in masonry should be properly anchored down with bolts of lengths varying from 75 to 120 cm and minimum dia. of 16 to 20 mm. For open sheds, anchorage has to be more secure and may be doubled. Two anchor bolts are provided on each end for ordinary trusses.

Camber. For small spans, the tie is made in one piece for the whole span and kept level, but in large spans the tie is cambered to reduce the effective lengths of the struts. Where tie is to be cambered, it is 1/30th to 1/40th of the span. Where the main tie is to be kept horizontal, a camber of 1/480 of the span is given to avoid appearance of sagging, or alternatively sag ties may be provided for spans above 9 m of same section as struts.

Provision shall be made for expansion and contraction of the trusses due to temperature changes. For small spans of say up to 7 metres, slotted holes are made in the truss base and the sole plates so that ends are free to slide. Or, one end may be fixed and the other end made sliding. For big spans say 20 metres or above, ends may be built on rollers. Free ends are made away from the direction of the prevailing wind, and the end from which direction the prevailing wind blows is made fixed.

Load reactions at the ends of the trusses are vertical on free ends and inclined on fixed ends.

Mild steel bed plates (also called bearing plates, base plates or sole plates) of size 30 cm×30 cm×10 to 15 mm thick are fixed over a stone template or a concrete block 30 cm thick under the truss ends with shoe angles of size 50×50×6 mm or 65×65×6 mm. Two angles are on each end fixed with the gusset plate. Plates are fixed with rag bolts where stone is provided.

Lead sheet 3 mm thick is provided under the plate on the free end. Slotted holes 5 cm long and of width 1.5 mm bigger than the diameter of the anchor bolts are made in the bed plate and anchor bolts passed through them, in the free end. In the fixed end, holes for anchor bolts are round and not slotted, and the lead sheet is omitted and replaced by cement grout.

For a series of trusses, wind ties, diagonal braces between the two end trusses should be provided on either side to prevent general distortion of the roof due to wind action.

Ties. Should not have slenderness value of more than 250 (max: 350). Deduction is made for the rivet holes from the cross-sectional area = t (d+4 mm) for each rivet hole. t is thickness of tie, d is dia. of rivet. See page 4 /13.

The stress is the main tie decreases towards the centre, the middle portion consists of smaller angles.

Design of Purlins

The spacing of purlins depends on the safe span of the sheeting materials, but as far as possible, purlins should be fixed at the joints (nodes or panel points) of the truss to avoid bending in the main rafter of the truss, otherwise heavier sections shall have to be provided for the rafters. But sometimes such an arrangement is unavoidable where roofing sheets of some particular size have to be fixed.

Purlins are fixed with angle cleats (or clips) to the principal rafters. Slotted holes should be made in the joints of the purlins, say at about 15-metre intervals, and arrangements made for their movement.

Design of Principal Rafters. Design for compression as struts. Effective length is taken 0·75×length of panel (between nodes) with Le/R less than 180. (See Section 10).

Angle sections are generally used for purlins. Minimum depth is 1/45 and width 1/60 of the length of the purlin. Where RS joists are used as purlins, the depth of such joists shall be not less than 1/40 of the span. Bending moment for the design of a purlin is taken as WL/10, where W is the total uniformly distributed load *normal* to the roof on the purlin. This is dead load (weight of sheets + self weight of purlin) + live load—according to para (c) page 11/2. Find out modulus of section (z) with permissible stress as given at page 10/2—which is 1650 kg/sq. cm.

Self weight of purlins is 6 to 15 kg/sq. m of the area covered, depending on the spacing of trusses and the weight of the roof coverings. It is generally taken 6 kg/sq. m for CGI sheets and 10 kg/sq. m for AC sheets and 15 kg/sq. m for tiles.

For wind-maximum suction (upward component) on the roof is taken 0.7 of wind pressure of roof area normal to roof.

Example :

Roof covering—AC sheets @ 20 kg/sq. m with fixtures & fittings.

Spacing of trusses—4.5 m (span of purlin)

Slope of roof—1 : 2=26°—34′

Spacing of purlins—1.5 m

Self weight of purlin—10 kg/m

Live load=75−(10×1+6.5×2)=52 kg/sq. m

(vide para(c) page 11/2)

Weight of AC sheets per purlin per metre

=20×1.5=30 kg

Live load normal to roof per purlin per metre

$$=52\times1.5\times\frac{1}{1.12}$$ (1.12 is vide Table at page 11/22)

for 1 : 2 slope)=65 kg.

Total dead load=30+10=40 kg/m

Dead load+live load=40+65=105 kg/m

(When wind load is considered, either the live load on the purlin can be decreased by 50% or the permissible stress in bending can be taken 1.33 times the usual permissible stress).

$$BM=\frac{WL}{10}=\frac{(105\times4{\cdot}5)}{10}\times4{\cdot}5=212{\cdot}6\text{ kg m}$$

$$Z\text{ required}=\frac{212{\cdot}6\times100}{1650}=12.9\text{ cm}^3$$

Min : width of purlin=L/60=450/60=7·5 cm

Min : depth of purlin=L/45=450/45=10 cm

Angle—100×75×6 @ 8 kg/m with Z=14·4 cm³ will do.

6. DESIGN OF STEEL ROOF TRUSSES

Size of Members for Steel Roof Trusses of Various Spans

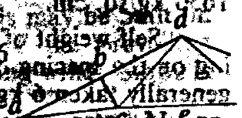
Size of L members in mm ; Roof Slope 1 in 2

5-metres span-Light Loading-4 pannels at 1.4 metres

Truss spacing		3–m centres	4–m centres	5–m centres
Member–P.R.	ad	2Ls–50×30×6	2Ls–50×30×6	2Ls–50×30×6
,,	dc, bc, ce, ac	1L–ditto–	1L–ditto–	1L–ditto–

5-metres Span-Medium Loading-4 panels at 1.4 metres

Member–P.R.	ad	2Ls–50×30×6	2Ls–50×30×6
Tie	ac	1L–ditto–	1L–65×45×6
Member–dc, bc, ce		1L–ditto–	1L–50×30×6



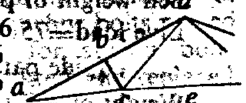
Dia. of Rivets–16 mm ; Thickness of Gussets–10 mm and 6 mm

6-metres Span-Lighting Loading-4 panels at 1.8 metres

Truss spacing		3–m centres	4–m centres	5–m centres
Member–P.R.	ad	2Ls–50×30×6	2Ls–50×30×6	2Ls–50×30×6
Tie	ac	1L–ditto–	1L–ditto–	1L–50×50×6
Member–dc, bc, ce		1L–ditto–	1L–ditto–	1L–50×30×6

6-metres Span-Medium Loading-4 panels at 1.8 metres

Member–P.R.	ad	2Ls–50×30×6	2Ls–50×30×6
Tie	ac	1L–50×50×6	1L–65×45×6
	dc, bc, ce	1L–50×30×6	1L–50×30×6



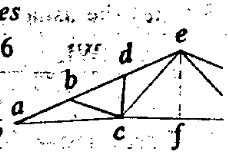
Dia. of Rivets–16 mm ; Thickness of Guessts–10 mm and 6 mm

7.5 metres Span Light Loading–6 panels at 1.4 metres

Truss spacing		3–m centres	4–m centres	5–m centres
Member–P.R.	ae	2Ls–50×30×6	2Ls–50×30×6	2Ls–50×30×6
Tie–(end)	ac	1L–ditto–	1L–65×45×6	1L–75×50×6
,, (central)	cf	1L–ditto–	1L–50×30×6	1L–50×30×6
Member	bc	1L–50×50×6	1L–50×50×6	1L–50×50×6
,,	ec, dc	1L–50×30×6	1L–50×30×6	1L–50×30×6

7.5-metres Span Medium Loading – 6 panels at 1.4 metres

Member–P.R.–ae		2Ls–50×30×6	2Ls–50×30×6
Tie (end)	ac	2Ls–ditto–	2Ls–ditto–
,, (central)	cf	1L–ditto–	1L–ditto–
Member	bc	1L–50×50×6	1L–50×50×6
,,	ec, dc	1L–50×30×6	1L–50×30×6



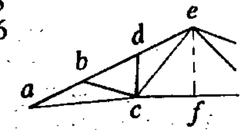
Dia. of Rivets–16 mm ; Thickness of Gussets–10 mm and 6 mm

9-metres Span Light Loading–6 panels at 1.68 metres

Truss spacing		3–m centres	4–m centres	5–m centres
Member–P.R.–ae		2Ls–50×30×6	2Ls–50×50×6	2Ls×65×45×6
Tie (end)	ac	1L–65×45×6	1L–75×50×6	2Ls–50×30×6
,, (central)	cf	1L–50×30×6	1L–50×30×6	1L–ditto–
Member	bc	1L–50×50×6	1L–50×50×6	1L–50×50×6
,,	ec, dc	1L–50×30×6	1L–50×30×6	1L–50×30×6

9-metres Span Medium Loading–6 panels at 1.68 metres

Member–P.R.–ae		2Ls–65×45×6	2Ls–65×45×6
Tie (end)	ac	2Ls–50×30×6	2Ls–50×30×6
,, (central)	cf	1L–ditto–	1L–65×45×6
Member	bc	1L–65×45×6	1L–ditto–
,,	ec, dc	1L–50×30×6	1L–ditto–



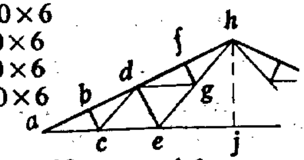
Dia. of Rivets–16 mm ; thickness of Gussets 10 mm and 6 mm

12-metres Span Light Loading–8 panels at 1.68 metres

Truss spacing		3–m centres	4–m centres	5–m centres
Member–P.R.	ah	2Ls–50×30×6	2Ls–65×45×6	2Ls–75×50×6
Tie (end)	ae	2Ls–ditto–	2Ls–50×30×6	2Ls–50×30×6
,, (central)	ej	1L–ditto–	1L–50×50×6	1L–65×45×6
Member	fg, he	1L–ditto–	1L–ditto–	1L–ditto–
,,	de	2Ls–ditto–	2Ls–50×30×6	2Ls–50×30×6
,,	dc, dg, bc	1L–ditto–	1L–ditto–	1L–ditto–

12-metres Span-Medium Loading–8 panels at 1.68 metres

Member–P.R.	ah	2Ls–75×50×6	2Ls–75×50×6
Tie (end)	ae	2Ls–50×30×6	2Ls–50×50×6
,, (central)	ej	1L–65×45×6	1L–75×50×6
Member	fg, he	1L–75×50×6	1L–90×60×6
	de	2Ls–50×30×6	2Ls–50×30×6
,,	dc, dg, bc	1L–ditto–	1L–ditto



Dia. of Rivets–16 mm ; Thickness of Gussets–10 mm and 6mm

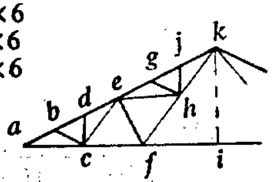
15-metres Span Light Loading–12 panels at 1.5 metres

Truss spacing		3–m centres	4–m centres	5–m centres
Member P.R.	ak	2Ls–65×45×6	2Ls–75×50×6	2Ls–90×60×6
Tie (end)	af	2Ls–50×30×6	2Ls–50×30×6	2Ls–50×50×6
,, (central)	fi	1L–75×50×6	1L–75×50×6	1L–90×60×6
Members	fk	1L–ditto–	1L–ditto–	1L–ditto–
,,	bc, dc, gh, jh	1L–50×30×6	1L–50×30×6	1L–50×50×6
,,	ef, ce, eh	2Ls–ditto–	2Ls–ditto–	2Ls–ditto–

Dia. of Rivets : 16 mm ; Thickness of Gussets–10 mm and 6 mm

15—metres Span Medium Loading—12 panels at 1.5 metres

Member P.R. ak 2Ls—75×50×6 2Ls—90×60×6
Tie (end) af 2Ls—65×45×6 2Ls—75×50×6
„ (central) fi 2Ls—50×30×6 2Ls—50×30×6
Members fk 2Ls—ditto— 2Ls—ditto—
„ bc, dc, gh; jh 1L—ditto— 1L—ditto—
„ ef, ce, eh 2Ls—ditto— 2Ls—ditto—



Dia. of Rivets : 16 mm ; Thickness of Gussets—12 mm and 8 mm

P.R. is principle rafter.

Truss members for various spans and spacings given in the tables have been worked out with the following assumptions :—

Light Loading Roofing : Covered with Corrugated Iron sheets or Asbestos Cement sheeting and steel purlins. Total load for design is taken at 100 kg/sq. metre (inclusive of wind load at 150 kg/sq. metre), normal for roof coverings, vertical for truss members (total weights divided by 1.33).

Medium Loading Roofing : Covered with single tiles or slates and steel purlins with ceiling. Total load for design is taken at 150 kg./sq. metre (inclusive of wind load), all as for Light loading

Roof slopes 1 in 2 Camber = span/28

Panels equal and equally loaded, spans centre to centre of bearings will be about 30 cm more.

The following variations will require no alterations in truss sections :

(*a*) Variations of ± 10 per cent in the span and spacings of the trusses.

(*b*) The omission or inclusion of a boarded ceiling.

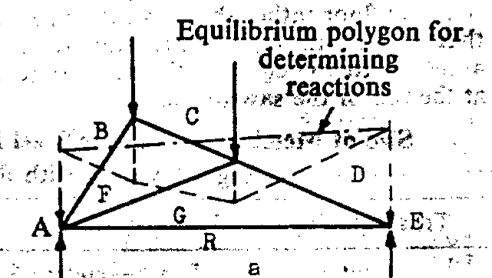
(*c*) Inclusion of ventilator at the ridge.

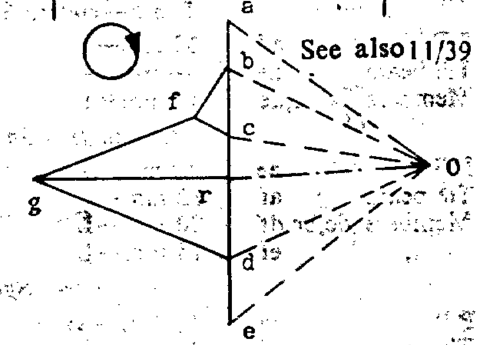
(*d*) If the truss is not cambered or is given a slight camber of say S/360, the truss spacings can be increased up to 15 per cent.

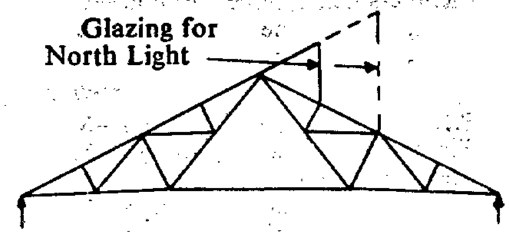
(*e*) If the panels are reduced from 6 to 4 or from 8 to 6, there will not be appreciable difference but the principal rafter lengths (of panels) will be increased necessitating a little heavier angle since it acts as a strut. This can be computed from the tables given earlier.

Trusses should be lifted at the nodes. Trusses above 12 members should not be slinged at the apex. They should be lifted by slinging at two mid-points of the rafters.

WORKSHOP AND FACTORY ROOFS
NORTHLIGHT ROOFS







Where it is desirable to provide north light without direct sunlight a "sawtoothed" roof is suitable. In this form of roof profi.e the north oriented face is pitched at an angle of between 50 deg. and 90 deg. and is covered with glass, usually in the form of patent glazing. The south oriented face of the roof is pitched at an angle of between 20 deg. and 30 deg. and is covered with some light-weight roof covering—CGI or AC sheeting, attached to purlins. The disadvantages of this pattern are that unless the roof slope is flattened (which is very undesirable on account of the risk of leaks), the truss is much deeper and consequently more expensive than the symmetrical truss. On prolonging the south roof slope of the fink truss beyond the ridge as far as necessary, a substitute for the ordinary saw tooth roof with a north light can be provided at a lesser cost, which is generally just as suitable and is often preferable. The interior of the roof can be painted white if necessary. These trusses are seldom used on spans exceeding 12 metres. For larger spans a composite form of construction comprising lattice girders and north light trusses is used.

The above illustration shows outlines of a saw-toothed roof and details for working out stresses graphically. The truss weight is divided among the panels BF, CF and DG in proportion to the lengths of these panels. The apex load at the peak is one-half of the sum of the loads on panels BF and CF ; load CD is one-half of those on panels CF and DG, and loads AB and DE one-half of those on panels BF and DG respectively. The reactions for the loads shown in the figures are obtained by means of the force and equilibrium polygons. All of the members of the truss are in compression except the horizontal tie GR which is in tension.

Where the roof is extended to obtain extra light, it does not appreciably increase the stresses in the members.

To admit in the maximum light and to prevent cutting off the light by the saw-tooth immediately in front and to ensure diffusion of the light over the floor rather than on the underside of the roof, it is considered that the north light glasses should be so fixed that they make an angle of about 20 to 25 deg. with the vertical at the bottom and of about 90 deg. at the top of the saw-tooth.

Size of Members of Tubular Steel Roof Trusses of Various Spans

Light Loading with Roof Slope 1 in 3

Truss spacing		3—m centres	4—m centres	5—m centres
		5 to 6—metres Span—Fig. D		
P.R.	ad	32 mm—L	32 mm—L	40 mm—L
Tie beam	ae	—ditto—	—ditto—	—ditto—
Members	bc,dc,de	15 mm—L	15 mm—L	15 mm—L
		7.5—metres Span—Fig. C		
P.R.	ae	40 mm—L	40 mm—M	50 mm—L
Tie beam	af	32 mm—L	32 mm—M	40 mm—L
Members	dc,bc,df	20 mm—L	20 mm—L	25 mm—L
,,	ef	15 mm—L	15 mm—L	15 mm—L
		10—metres Span—Fig. F		
P.R.	ah	40 mm—M	50 mm—M	50 mm—M
Tie beam	aj	40 mm—L	50 mm—L	50 mm—L
Members	dc,dg,de,bc	20 mm—L	25 mm—L	25 mm—L
,,	he	32 mm—L	40 mm—L	40 mm—L
,,	hj	15 mm—L	20 mm L	20 mm—L
		12—metres Span—Fig. F		
P.R.	ah	50 mm—M	65 mm—M	65 mm—M
Tie beam	aj	40 mm—M	50 mm—M	50 mm—M
Members	dc,dg,de,bc	20 mm—M	25 mm—M	25 mm—M
,,	he	32 mm—M	40 mm—M	40 mm—M
,,	hj	15 mm—M	20 mm—M	20 mm—M
		15—metres Span—Fig. H		
P.R.	ak	65 mm—M	65 mm—M	65 mm—H
Tie beam	ai	65 mm—M	80 mm—L	80 mm—M
Members	bc,dc,gh,jh	25 mm—L	25 mm—L	25 mm—L
,,	ef,ce,eh	25 mm—L	32 mm—L	40 mm—M
,,	kf	65 mm—M	65 mm—M	65 mm—M
,,	ki	40 mm—M	40 mm—M	40 mm—M

P.R. is principal rafer. Light loading is with Corrugated Iron or Asbestos Sheets. "L" is light, "M" is medium. Size is "nominal bore"—See Table on next page.

Circular steel tubes are structurally efficient. Steel tube trusses are normally shop welded with flange plates bolted at site. Tube junctions involve machining of tube ends to a curved profile to saddle on to the larger chord tubes, with fillet welding.

Sizes and Properties of Steel Tubes for Structural Purposes

IS: 1161-1979

Nominal Bore	Outside Dia.	Class	Thickness	Weight	Area of Cross Section	Moment of Inertia	Modulus of Section	Radius of Gyration
mm	mm		mm	kg/m	cm^2	cm^4	cm^3	cm
15	21.3	L	2.00	0.962	1.21	0.57	0.54	0.69
		M	2.65	1.22	1.55	0.69	0.65	0.67
		H	3.25	1.45	1.84	0.77	0.73	0.65
20	26.9	L	2.35	1.42	1.81	1.38	1.02	0.87
		M	2.65	1.58	2.02	1.50	1.12	0.86
		H	3.25	1.90	2.41	1.72	1.28	0.84
25	33.7	L	2.65	2.04	2.58	3.14	1.86	1.10
		M	3.25	2.46	3.11	3.65	2.16	1.08
		H	4.05	2.99	3.77	4.22	2.51	1.06
32	42.4	L	2.65	2.61	3.31	6.57	3.10	1.41
		M	3.25	3.15	4.00	7.71	3.64	1.39
		H	4.05	3.86	4.88	9.07	4.28	1.36
40	48.3	L	2.9	3.27	4.14	10.70	4.43	1.61
		M	3.25	3.61	4.60	11.73	4.86	1.60
		H	4.05	4.43	5.63	13.90	5.75	1.57
50	60.3	L_1	2.9	4.14	5.23	21.59	7.16	2.03
		L_2	3.25	4.57	5.82	23.77	7.89	2.02
		M	3.65	5.10	6.50	26.17	8.68	2.01
		H	4.5	6.17	7.89	30.90	10.2	1.98
65	76.1	L	3.25	5.84	7.44	49.44	13.0	2.58
		M	3.65	6.53	8.31	54.65	14.4	2.56
		H	4.5	7.92	10.1	65.12	17.1	2.54
80	88.9	L	3.25	6.86	8.74	80.31	18.1	3.03
		M	4.05	8.48	10.8	97.38	21.9	3.00
		H	4.85	10.1	12.8	113.46	25.5	2.98
90	101.6	L	3.65	8.82	11.2	134.9	26.6	3.47
		M	4.05	9.75	12.4	147.9	29.1	3.45
		H	4.85	11.6	14.7	172.9	34.0	3.42
100	114.3	L	3.65	9.97	12.7	194.4	34.0	3.91
		M	4.5	12.1	15.5	234.3	41.0	3.89
		H	5.4	14.5	18.5	274.5	48.0	3.85
110	127.0	L	4.5	13.6	17.3	325.3	51.2	4.33
		M	4.85	14.6	18.6	347.7	54.8	4.32
		H	5.4	16.2	20.6	382.0	60.2	4.30
125	139.7	L	4.5	14.9	19.1	437.2	62.6	4.78
		M	4.85	16.2	20.5	467.6	66.9	4.77
		H	5.4	17.9	22.8	514.5	73.7	4.75

Nominal Bore	Out-side Dia:	Class	Thick-ness	Weight	Area of cross Section	Moment of Inertia	Modulus of Section	Radius of Gyration
mm	mm		mm	kg/m	cm²	cm⁴	cm³	cm
135	152.4	L	4.5	16 4	20.9	572.2	75.1	5.23
		M	4.85	17.7	22.5	612.5	80.4	5.22
		H	5.4	19.5	24.9	674.5	88.5	5.20
150	165.1	L	4.5	17.8	22.7	732.6	88.7	5.68
		M	4.85	19.2	24.4	784.5	95.0	5.67
		H	5.4	21.2	27.1	864.7	105.0	5.65
150	168.3	L	4.5	18.1	23.2	777.2	92.4	5.79
		M	4.85	19.6	24.9	832 4	98.9	5.78
		H1	5.4	21.7	27.6	917.7	109	5.76
		H2	6.3	25.3	32.1	1053	125	5.73
175	193.7	L	4.85	22.6	28.7	1284	133	6.68
		M	5.4	25.0	31.9	1417	146	6.66
		H	5.9	27.3	34.8	1536	159	6.64
200	219.1	L	4.85	25.7	32.6	1874	171	7.58
		M	5.6	29.4	37.6	2142	195	7.55
		H	5.9	31.0	39.5	2247	205	7.54
225	244.5	H	5.9	34.2	44.2	3149	258	8.44
250	273.0	—	5.9	38.8	49.5	4412	323	9.45
300	323.9	—	6.3	49.5	63.4	7992	493	11.2
350	355.6	—	8.0	68.3	86.5	13111	737	12.3

Steel tubes are designated by their nominal bore and are classified as "Light"—L; "Medium"—M and "Heavy"—H, depending on the wall thickness.

Permissible Stress in Axial Compression for Steel Tubes of Grade YSt 210
IS : 1161-1979

l/r	kg/sq.cm	l/r	kg/sq. cm	l/r	kg/sq. cm	l/r	kg/sq. cm
0	1250	70	970	140	540	210	243
10	1217	80	929	150	490	220	219
20	1175	90	876	160	432	230	198
30	1131	100	814	170	381	240	180.
40	1088	110	745	180	339	250	162
50	1046	120	674	190	304	300	106
60	1002	130	603	200	271	350	71

l/r is slenderness ratio ; l being the effective length and r the radius of gyration.

7. DESIGN OF STEEL ROOF TRUSSES BY STRESS CO-EFFICIENT TABLES

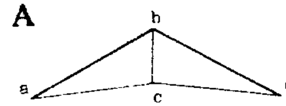
To find stress in any member:

(*a*) Multiply the total dead load carried by the truss including its own weight by the co-efficient for dead load.

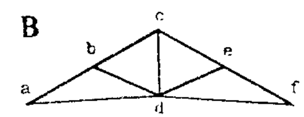
(*b*) Multiply the total wind load pressure (or the live load to be taken in lieu of the wind load) acting normal to its surface on one side of the roof by the co-efficient for wind pressure.

S=Span, R=rise, C=camber.

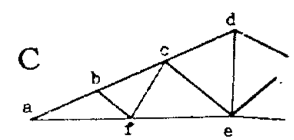
Member of Truss	Kind of Stress	Stress co-efficient: $R=\frac{S}{4}$ $C=\frac{S}{30}$ Dead load (vertical)	$R=\frac{S}{4}$ $C=\frac{S}{30}$ Wind load (normal to roof surface)	$R=\frac{S}{3}$ $C=\frac{S}{24}$ Dead load (vertical)	$R=\frac{S}{3}$ $C=\frac{S}{24}$ Wind load (normal to roof surface)
ab	+	.65	.70	.52	.58
bc	—	.08	.07	.07	.05
ac	—	.58	.49	.43	.27
ab	+	.97	1.05	.77	.70
bc	+	.64	.70	.51	.58
bd	+	.31	.69	.24	.58
ad	—	.87	1.13	.65	.80
dc	—	.33	.34	.32	.35
ab	+	1.08	1.24	.86	.86
bc	+	.94	1.10	.74	.80
cd	+	.65	.70	.51	.58
bf	+	.17	.37	.14	.30
ce	+	.25	.55	.21	.51
fc	—	.13	.30	.12	.30
de	—	.41	.44	.40	.45
fe	—	.77	.92	.57	.63
af	—	.96	1.35	.72	.98
ab	+	.97	1.05	.77	.70
bd	+	.85	1.05	.63	.70
bc	+	.22	.50	.21	.50
ac	—	.87	1.18	.65	.80
ce	—	.54	.45	.41	.28
cd	—	.34	.69	.26	.54

A


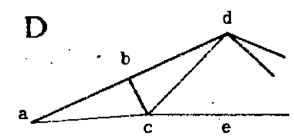
3 m

B


4.5 m

C


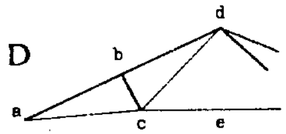
7.5 m

D


5 to 6 m

Member of Truss	Kind of Stress	Stress co-efficient $R=\frac{S}{3}$	$\frac{S}{R}$ 30°	$R=\frac{S}{4}$	$R=\frac{S}{5}$
ab	+	·70	·75	·84	1·01
bd	+	·54	·63	·73	·92
bc	+	·21	·22	·22	·23
ac	—	·56	·65	·75	·94
ce	—	·40	·43	·50	·61
cd	—	·19	·22	·25	·31
ab	+	·91	1·03	1·20	1·45
bd	+	·77	·91	1·06	1·35
bc	+	·21	·22	·13	·23
ac	—	·77	·91	1·06	1·40
ce	—	·45	·52	·60	·75
cd	—	·36	·42	·49	·63
ab	+	·75	·83	·93	1·12
bd	+	·59	·67	·76	·93
de	+	·57	·63	·78	1·00
bc	+	·16	·17	·18	·20
dc	+	·16	·17	·18	·20
ac	—	·63	·72	·83	1·04
cf	—	·36	·43	·50	·63
ce	—	·25	·39	·33	·41
ab	+	1·02	1·15	1·31	1·61
bd	+	·82	·94	1·08	1·35
de	+	·83	·98	1·16	1·48
bc	+	·17	·19	·21	·27
dc	+	·17	·19	·21	·27
ac	—	·85	1·01	1·18	1·50
cf	—	·45	·52	·60	·75
ce	—	·44	·52	·61	·78

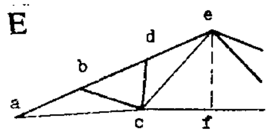
D



5 to 6 m
without camber

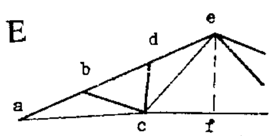
Ditto.
with camber=R/6

E



7 to 10 m
without camber

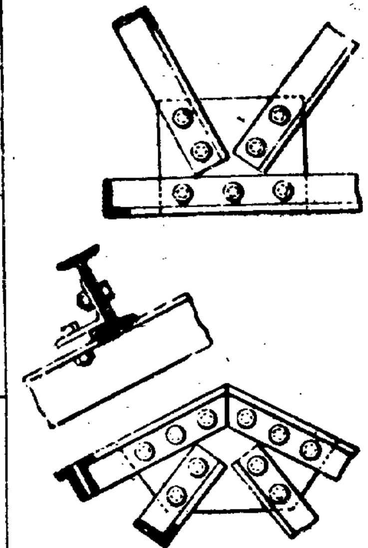
E



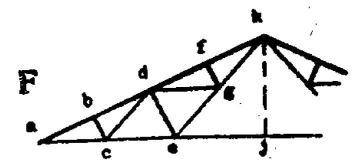
7 to 10 m
with camber =Rise/6

Add (*a*) and (*b*): This will give the total stress for which a member has to be designed either as a strut or as a tie. This total stress may be divided by 1·33 where wind and dead loads have been taken together.

Member of Truss	Kind of Stress	Stress co-efficient $R=\frac{S}{4}$ Dead load (vertical)	$C=\frac{S}{30}$ Wind load (normal to roof surface)	$R=\frac{S}{3}$ Dead load (vertical)	$C=\frac{S}{24}$ Wind load (normal to roof surface)
ab	+	1·13	1·34	·90	·93
bd	+	1·07	1·34	·83	·93
df	+	1·02	1·34	·76	·93
fh	+	·96	1·34	·69	·93
bc	+	·11	·25	·10	·25
fg	+	·11	·25	·10	·25
de	+	·22	·50	·21	·50
ac	—	1·01	1·46	·75	1·06
ce	—	·87	1·13	·64	·81
ej	—	·54	·45	·41	·28
cd	—	·15	·32	·11	·26
dg	—	·15	·32	·11	·26
gh	—	·49	1·01	·36	·80
eg	—	·34	·69	·26	·55
ab	+	1·08	1·24	·86	·86
bd	+	1·00	1·24	·81	·94
df	+	·75	·85	·60	·68
bc	+	·15	·33	·14	·34
de	+	·22	·50	·22	·52
dc	—	·59	·43	·18	·42
fe	—	·27	·55	·26	·57
ac	—	·96	1·35	·72	·98
ce	—	·77	·92	·58	·63
eg	—	·56	·47	·42	·28

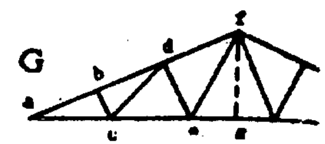


F



10 to 15 m

G

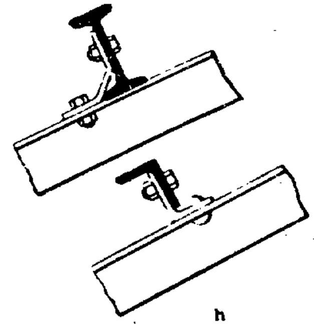


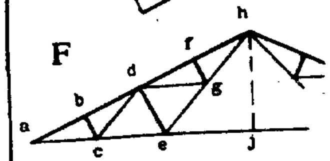
9 m

(*c*) Where only one stress co-efficient is given in the table it can be used for a case where both dead and live loads have been resolved to act as one single vertical load. (Wind load is considered to act only on one side of the truss at a time).

(The different tables might give slightly varying results but it will not make any appreciable difference in the final choice of a suitable section).

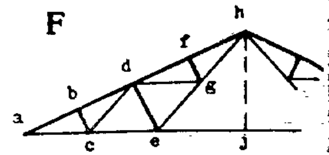
Member of Truss	Kind of Stress	Stress co-efficients $R=\frac{S}{3}$	$R=30°$	$R=\frac{S}{4}$	$R=\frac{S}{5}$	$R=\frac{S}{6}$
ab	+	.80	.90	.98	1.20	1.38
bd	+	.72	.81	.92	1.13	1.34
df	+	.65	.75	.87	1.09	1.30
fh	+	.58	.70	.81	.04	1.27
bc	+	.10	.11	.11	.12	.12
fg	+	.10	.11	.11	.12	.12
de	+	.21	.22	.22	.23	.24
ac	—	.66	.76	.88	1.09	1.30
ce	—	.56	.65	.75	.94	1.13
ej	—	.40	.43	.50	.63	.75
cd	—	.09	.11	.13	.16	.19
dg	—	.09	.11	.13	.16	.19
gh	—	.28	.33	.38	.47	.56
eg	—	.20	.22	.25	.31	.38
ab	+	1.06	1.20	1.37	1.56	1.74
bd	+	.99	1.14	1.31	1.38	1.59
df	+	.92	1.08	1.26	1.60	1.81
fh	+	.85	1.02	1.20	1.55	1.78
bc	+	.10	.11	.11	.12	.12
fg	+	.10	.11	.11	.12	.12
de	+	.21	.22	.22	.23	.24
ac	—	.90	1.06	1.24	1.58	1.79
ce	—	.77	.90	1.06	1.35	1.54
ej	—	.45	.52	.60	.75	.87
cd	—	.13	.15	.18	.23	.26
dg	—	.13	.15	.18	.22	.26
gh	—	.49	.57	.67	.86	.95
eg	—	.36	.42	.49	.63	.70





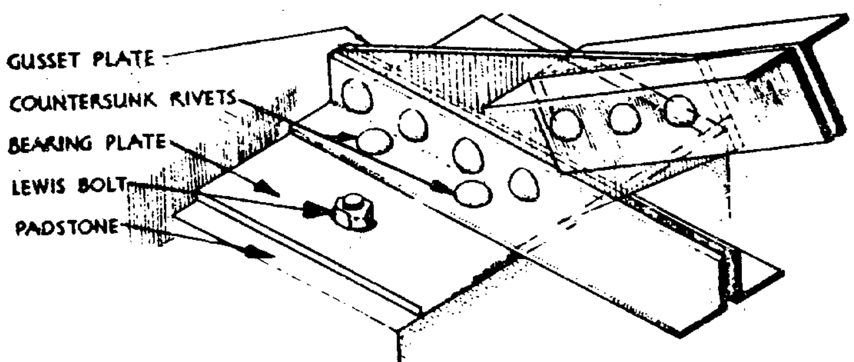
F

10 to 15 m
without camber

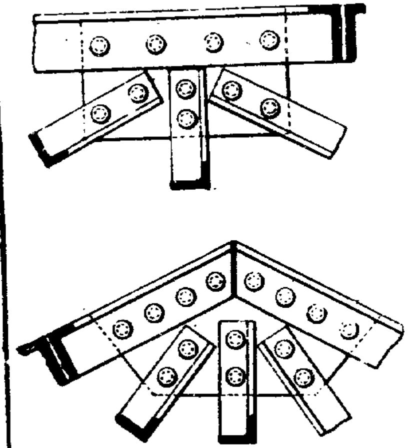


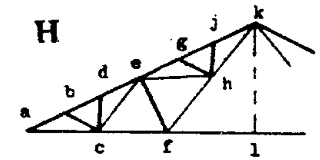
F

10 to 15 m
with camber=Rise/6



Member of Truss	Kind of Stress	Stress co-efficient Dead Load (vertical)	Wind load (normal to roof surface)	Length co-efficient
ab	+	1.03	1.21	.093
bd	+	.94	1.10	.093
de	+	.95	1.21	.093
eg	+	.91	1.21	.093
gj	+	.83	1.10	.093
jk	+	.84	1.21	.093
bc	+	.09	.20	.084
dc	+	.09	.20	.084
gh	+	.09	.20	.084
jh	+	.09	.20	.084
ef	+	.22	.50	.140
ac	—	.92	1.49	.156
cf	—	.75	1.12	.156
f l	—	.50	.56	.188
ce	—	.17	.37	.156
eh	—	.17	.37	.156
hk	—	.42	.93	.156
fh	—	.25	.56	.156

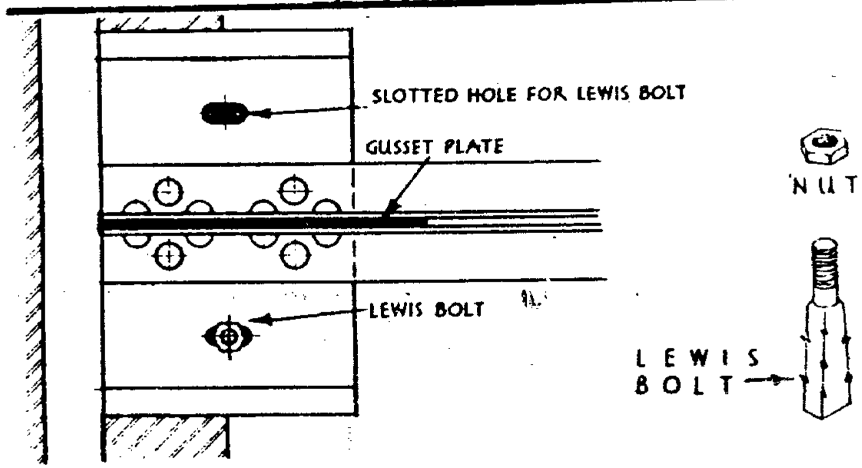


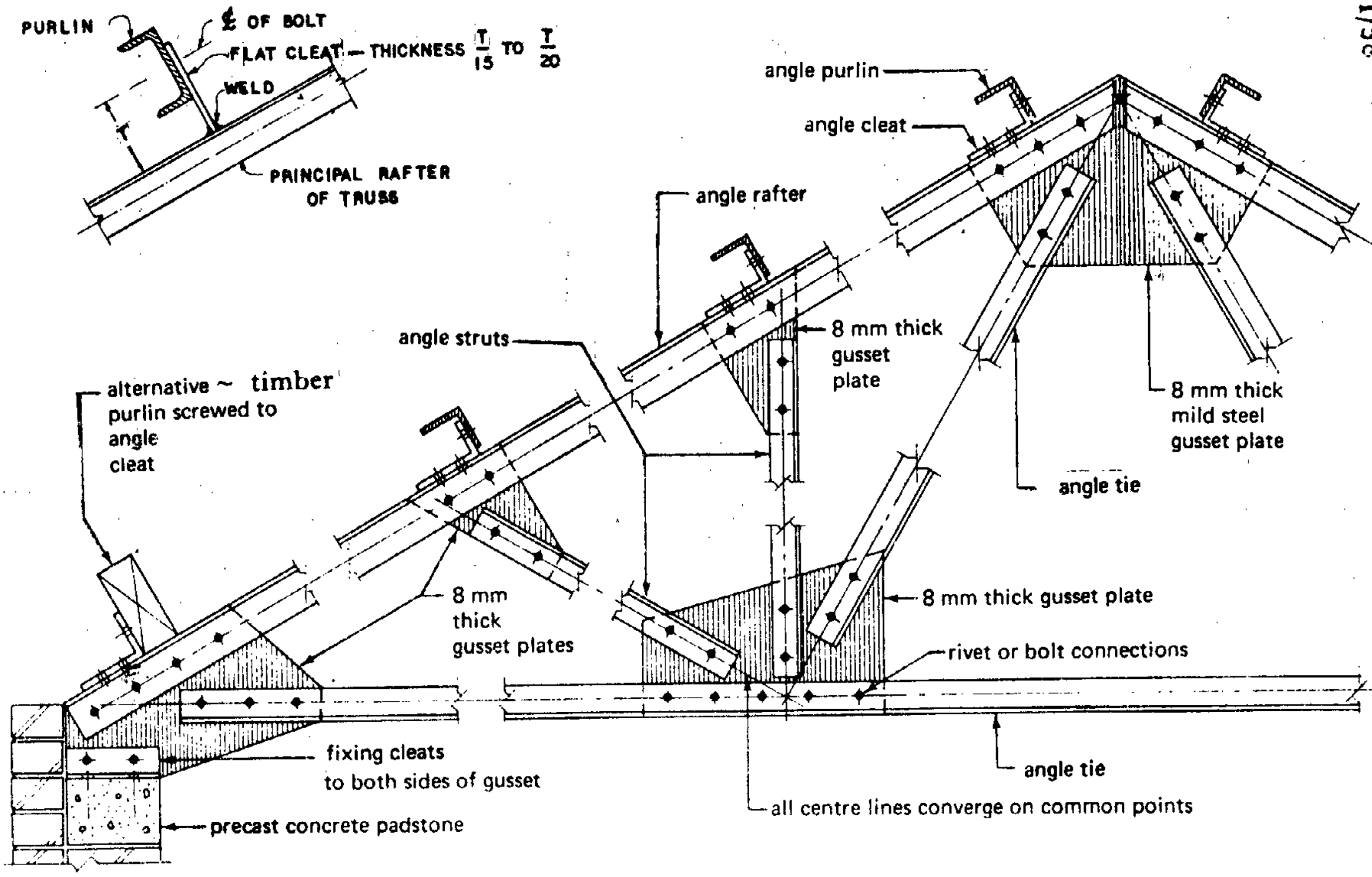


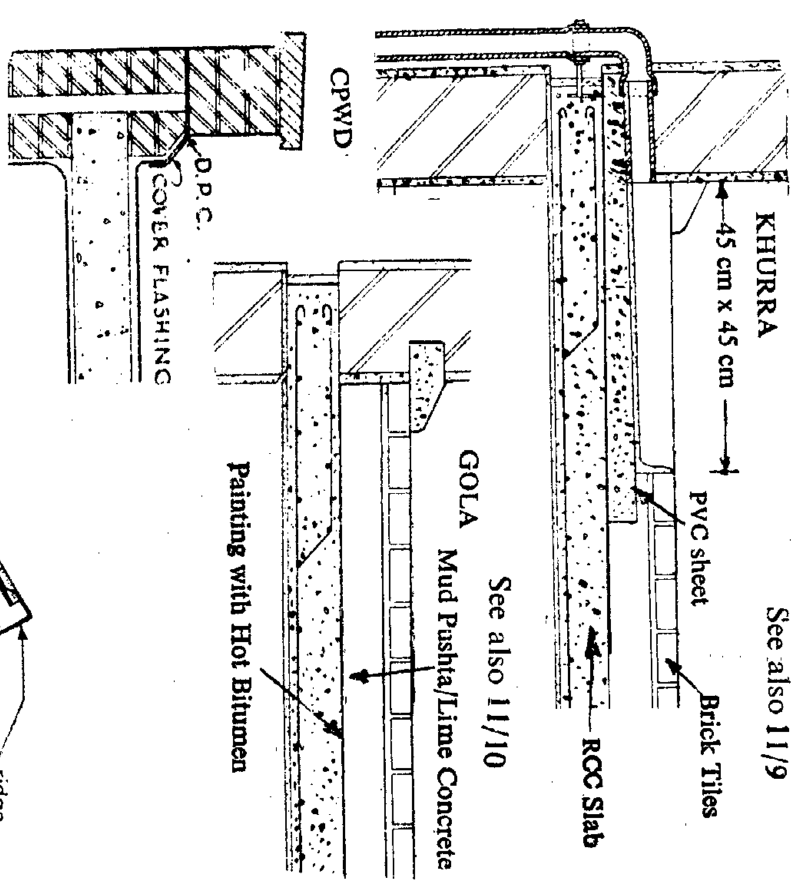
H

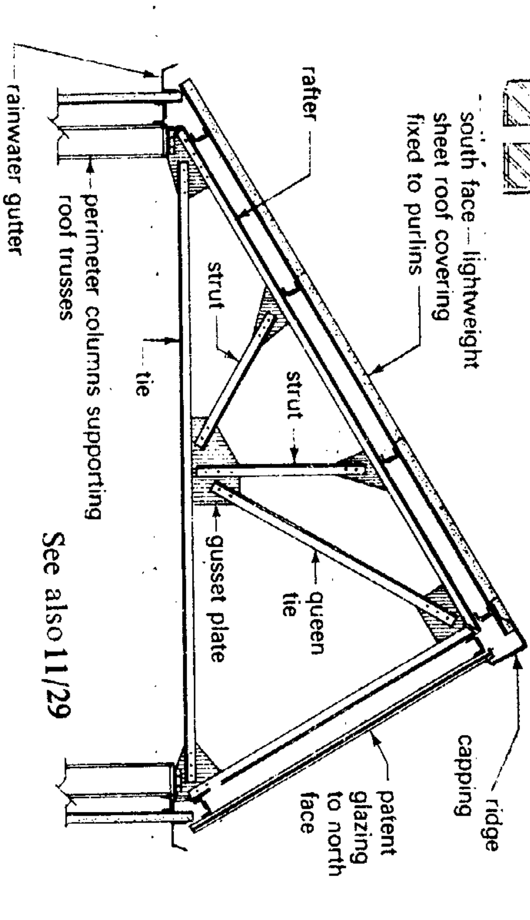
15 to 20 m

The above co-efficients are for trusses without camber. If camber is given, of say about span/28, loadings may be based on a span of about 20 per cent more to give increased stresses. The rise is: span/4.



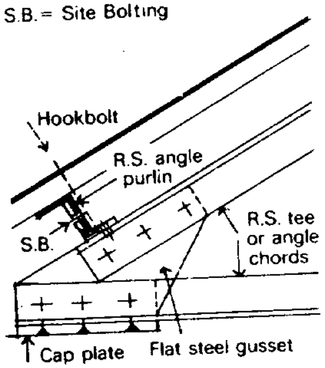




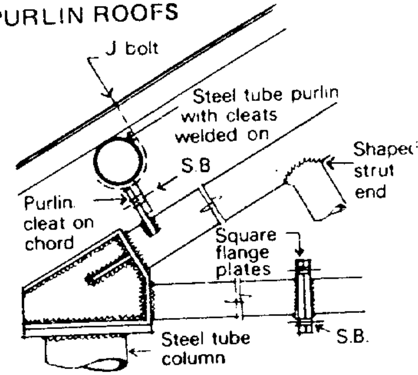


Single span steel northlight roof truss

PURLIN ROOFS

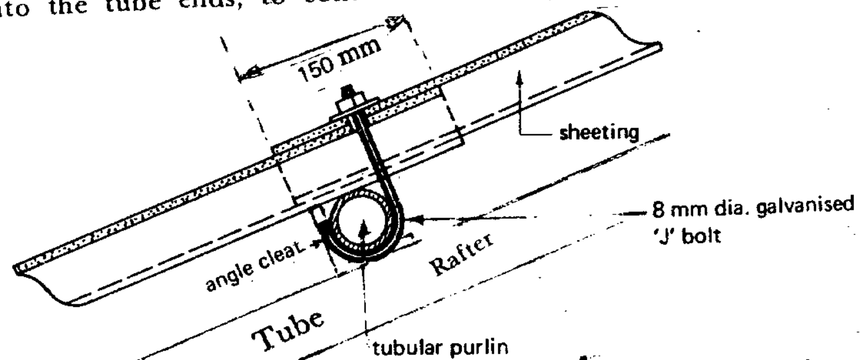


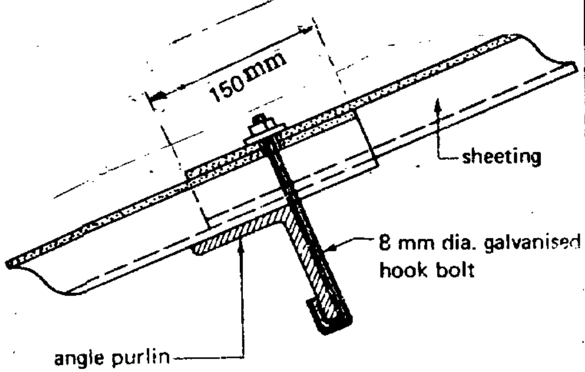
Rolled steel truss and purlin

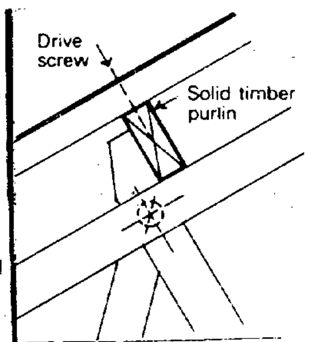


Steel tube truss and purlin

Tubular purlins meet at upstand cleats, fillet welded to the top chord of the truss. End-to-end butt joints, although very clean in appearance, are expensive, requiring precise splay machining to the tube ends, and a backing ring inserted into the tube ends, to contain the weld metal in the vee groove







Solid timber purlin on bolted connectored truss

ADDENDA

(See pages 11/6 to 11/9 and 11/39)

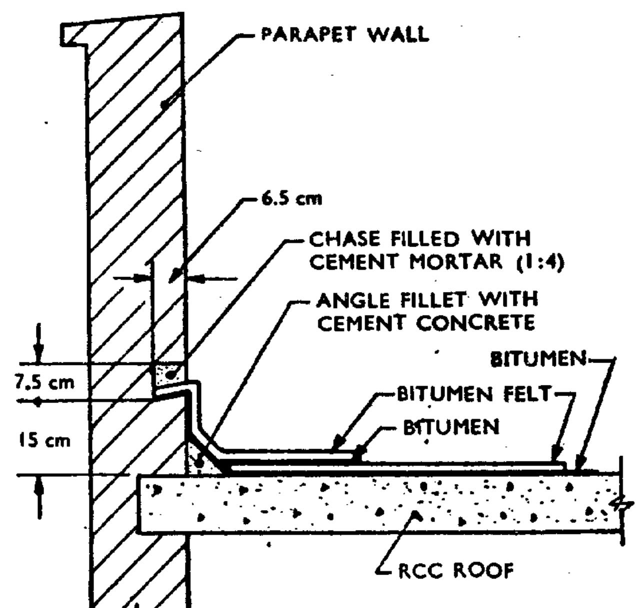
WATER-PROOFING ROOFS

Illustrations show the Sequence of Operations

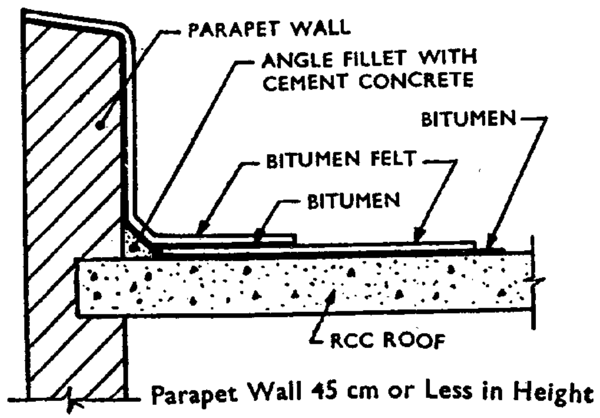
(A) With Shalimar Tarfelt (Bitumen Felt)

A coat of hot bitumen (adhesive mastic) is laid at the rate of 1.2 kg./sq. m (min:) over a prepared smooth roof surface and Tarfelt is laid immediately, over which another coat of hot bitumen is applied. While the bitumen is still hot the roof is then covered with coarse sand, grit, gravel or tiles or any conventional roof surfacing.

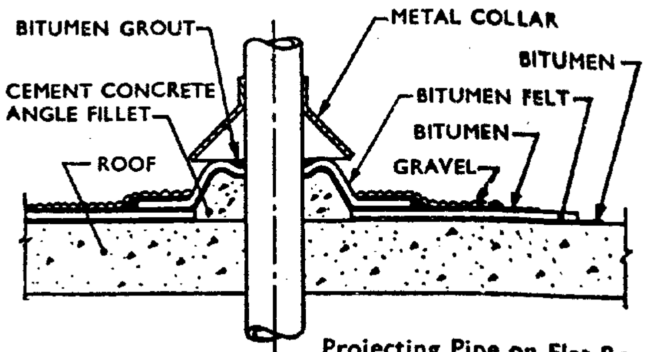
For normal works, one layer of Tarfelt and two coats of hot bitumen are required and for heavy works, two layers of Tarfelt and three coats of hot bitumen are needed. The bitumen is heated to 170 deg. C — 180 deg. C.



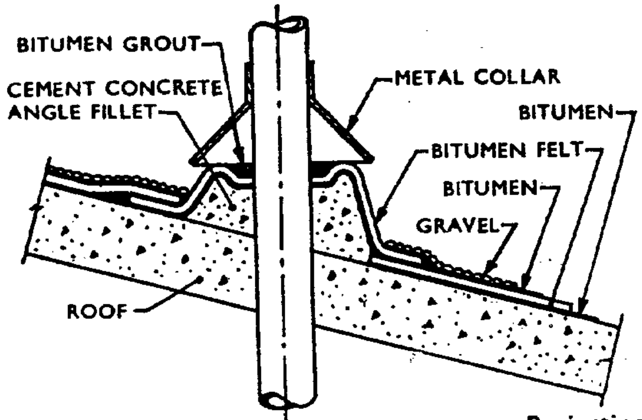
Parapet Wall Over 45 cm in Height



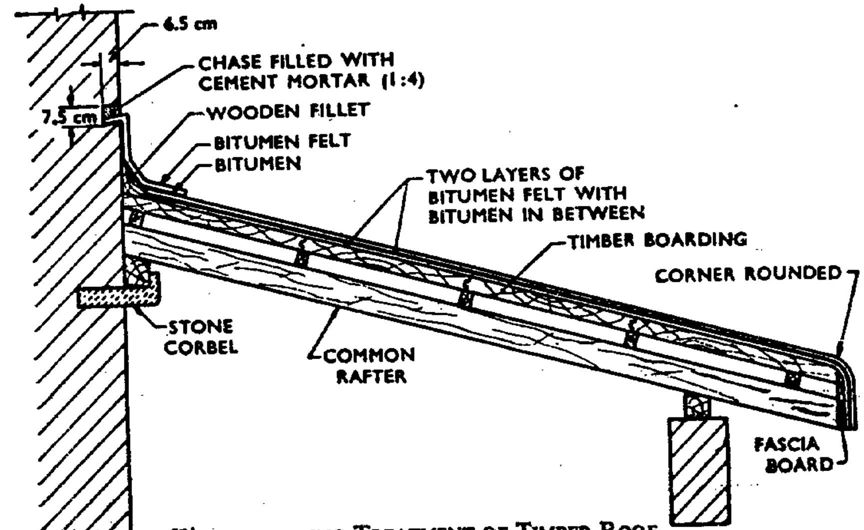
Parapet Wall 45 cm or Less in Height



Projecting Pipe on Flat Roof



Projecting Pipe on Sloping Roof



Waterproofing Treatment of Timber Roof

(B) Using Polyethylene Film

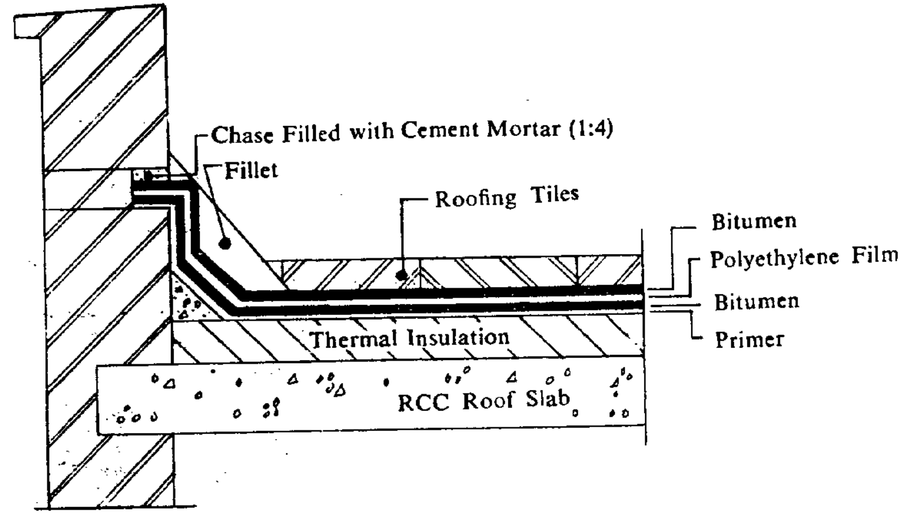
(a) *Under Mud-Pushta* (Fig. at page 11/39)

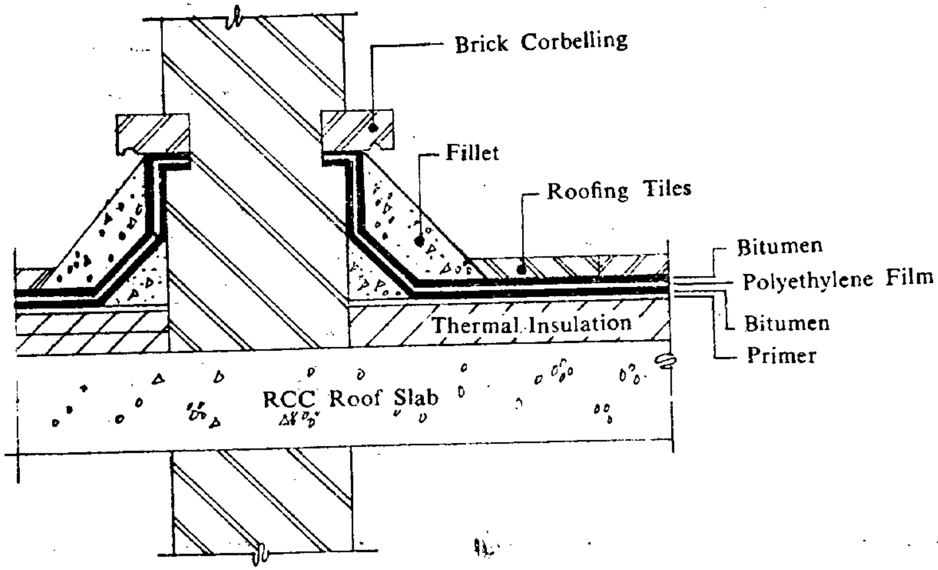
A coat of hot bitumen at the rate 1.2 kg./sq. m (min:) is laid over a prepared smooth roof surface. Polyethylene film of 100 to 150 mic. thickness is laid over the warm tack coat of bitumen. An 80 to 150 mm layer of mud pushta is laid over the film.

Prior to the actual water-proofing treatment, the roof surface is prepared smooth; dust and dirt are removed and a coat of primer is applied to bind the residual dust.

(b) *A modified method* involves the use of a polyethylene film/kraft paper laminate. The kraft paper provides the necessary regidity to the film, making it possible to lay the film free of wrinkles

Hot bitumen is applied at the min: rate of 1.2 kg./sq. m. The polyethylene film/paper laminate with the film face downwards is applied and pressed down over the warm tacky coat of bitumen. The overlap joints of min: width 75 mm are sealed with the help of hot bitumen. Hot bitumen is mopped over the paper surface of the laminate at the min: rate of 2.5 kg./sq. m. Over this roofing tiies, or any conventional roofing material is applied.





SECTION 12

BUILDING MATERIALS

Properties and Uses

1. BRICKS

Standard sizes for 9-inch bricks as prescribed by various Public Works Departments in India :—

$9'' \times 4\frac{1}{2}'' \times 2\frac{1}{2}''$	$9\frac{1}{2}'' \times 4\frac{3}{4}'' \times 2\frac{3}{4}''$	Indian Standard (Modular)
$8\frac{7}{8}'' \times 4\frac{1}{4}'' \times 2\frac{3}{4}''$	$9'' \times 4\frac{3}{8}'' \times 2\frac{3}{4}''$	19 cm × 9 cm × 9 cm
$8\frac{3}{4}'' \times 4\frac{1}{4}'' \times 2\frac{5}{8}''$	$9'' \times 4\frac{1}{4}'' \times 2\frac{3}{4}''$	19 cm × 9 cm × 4 cm
$8\frac{7}{8}'' \times 4\frac{1}{4}'' \times 3''$	$9'' \times 4\frac{1}{4}'' \times 2\frac{1}{2}''$	
10-inch bricks :		With mortar joints the sizes taken for brickwork are :
$10'' \times 4\frac{7}{8}'' \times 3''$	$9\frac{3}{4}'' \times 4\frac{3}{4}'' \times 2\frac{3}{4}''$	20 cm × 10 cm × 10 cm
$10'' \times 5'' \times 3''$	$9\frac{1}{2}'' \times 4\frac{1}{2}'' \times 2\frac{3}{4}''$	20 cm × 10 cm × 5 cm
$10'' \times 5'' \times 3\frac{1}{2}''$	$9\frac{3}{4}'' \times 4\frac{3}{4}'' \times 2\frac{7}{8}''$	

Brick moulds are about 1/10th larger than the required size of the brick to allow for shrinkage on burning. The length of the brick is equal to twice the width plus one mortar joint and three times the height plus two mortar joints.

Common size of bricks :

in England—$8\frac{3}{4}'' \times 4\frac{1}{4}'' \times 2\frac{5}{8}''$; $8\frac{3}{4}'' \times 4\frac{1}{4}'' \times 2\frac{3}{4}''$

in America—$8'' \times 3\frac{3}{4}'' \times 2\frac{1}{4}''$; $8'' \times 4'' \times 2\frac{3}{8}''$.

General Physical Characteristics

A good first class brick should be sound, hard and well burnt with uniform size, shape and colour (generally deep red or copper), homogeneous in texture and free from flaws and cracks. A fractured surface should show a uniform compact structure free from holes, lumps or grit. The surface should not be too smooth as otherwise mortar will not stick to it. Arrises should be square, straight and sharply defined. No dimension of a first class brick to vary more than 3 mm from the standard size. A brick should give a metallic ring when struck with a small hammer or another brick. A good brick should not break when struck against another brick or when dropped flat from a height of about 1.2 to 1.5 m on the ground. It should have a surface so hard that cannot be scratched by the finger-nail. A first class brick should not absorb more than 1/6th of its weight when dry, and a second class brick not more than 1/4th, after immersion in in water for 1 hour. Bricks of low porosity have greater strength. (Absorption test is only very rough and depends upon the clay used.)

All (burnt) bricks should be soaked in water for at least one hour before use with lime or cement mortars. The cessation of bubbles through the water is an indication of saturation being complete. (Some departments prescribe soaking in water for six hours but tests show that bricks absorb no further water after 15 minutes soaking). A "frog" or "kick" is made 6 mm deep and bricks are usually laid frog up which affords a key for the mortar.

A small proportion of lime, not exceeding 5 per cent, which must be in a fairly divided state and not in lumps, is useful in brick earths. Oxide of iron lends the bricks its peculiar red colour, while magnesia gives a yellow tint. A small amount of alkali is useful as it has an influence on plasticity of clay which can be mixed and kneaded well. A high percentage of alkali produces *efflorescence*. The bricks should show no signs of efflorescence after soaking in water and drying in shade. Overburning the bricks reduces efflorescence.

Iron pyrites, salts, pebbles, nodules of kankar, gravel and tree roots in the clay are harmful. Presence of lime can be detected by pouring a few drops of an acid on the earth when effervescence will be formed.

Test for Good Clay for Bricks

Make a few bricks and let them dry in the open air. If the bricks cracks, it shows the earth is too plastic and needs mixing sand. If they break easily when thrown down on ground, they are brittle and have too much sand and are apt to fuse in burning and become "Jhama". Too much water also makes a brick brittle.

It rarely happens that earths occurring in nature are suitable for making first class bricks. Pure clay requires the addition of loam, sand, etc., whilst loams may require the addition of clay. Sand in brick earth prevents shrinkage cracking and warping of bricks, but too much of sand makes the bricks brittle. Clay content should range between 10 and 20 per cent. No earth or clay should be taken from localities where salt water is found and any containing the slightest trace of salt must be rejected for brick making. Presence of salt can be detected by the formation of efflorescence on the sides of fresh excavations if moist. Dry earth can be tested by moistening it a little with water and drying it. (For brick burning or brick kilns, see under "Estimating.")

Strength of Bricks :

Crushing strengths of Indian bricks are very variable, and may vary from 30 kg/sq. cm to 150 kg/sq. cm for hand-made burnt bricks, while heavy duty bricks machine pressed (also called engineering bricks) may have compressive strength as high as 450 kg/sq. cm, and even 500 kg/sq. cm. The minimum crushing (or compressive) strengths of burnt bricks tested flatwise prescribed are : (*i*) Common building bricks—35 kg/sq. cm, (*ii*) Second class bricks—70 kg/sq. cm, (*iii*) First class bricks—105 kg/sq. cm, (*iv*) Bricks with crushing strength not less than 140 kg/sq. cm are graded as AA class. The strength of a brick decreases by about 25 per cent when soaked in water.

Strength of sun-dried (un-burnt) bricks is from 15 to 25 kg/sq. cm.

Water absorption of bricks after 24 hours' immersion :

First class bricks—20%, Second class bricks—22%, Third class bricks—25%. Heavy duty machine made bricks should not absorb more than 5% of their weight.

Breeze fixing bricks—Composition :

1 Cement
4 Coke breeze screened to 6 to 20 mm or screened coal ashes } Will readily receive screw or nail

Fire-Bricks

Fire-bricks are made of fire-clay or refractory clay, burnt at a high temperature in special kilns. Fire-bricks are generally of a white or yellowish white colour. The colour of fire-clay is generally grey, and is somewhat greasy to the touch. A fire clay with a coarse open grain is found to be more refactory than the one with a close even texture. Fire-bricks are used for the lining of furnaces, boilers, combustion chambers and chimney flues, where great heat is developed.

Fire-bricks should show no signs of fusion when heated to a temperature of about 1600 deg. C. Crushing strength of fire bricks should be not less than 125 kg/sq. cm. Fire-bricks weigh about 2400 kg/cu. m and absorb water from about 5 to 10 per cent.

Fire-bricks are always set in a mortar of fire-clay and not in lime or cement. About 0.06 cu. m (75 kg) of fire-clay is required for laying 100 bricks. The joints of the fire-brick lining should be made as fine as possible. The fire-bricks should be merely dipped in well puddled fire-clay mixed with water to the consistency of paste, so that there is no appreciable thickness of joint between the fire-bricks. The fire-brick linings should be laid one course of headers followed by two courses of stretchers. Lime stone is to be avoided in construction intended to be fire-proof.

Terra-cotta

Terra-cotta is a kind of earthenware which is generally used as a substitute for stone in the ornamental parts of buildings. It is made like clay products burnt at very high temperature; usually contains : 8 parts sifted dry clay, 3 parts crushed pottery, 2 parts white sand, 1 part ground glass. Porous terra-cotta is made by adding saw dust or ground cork, and is very light but weak structurally. It can be sawed and nailed very easily.

Terra-cotta is vitrified on the surface and made into various colours and patrons. Hollow blocks are made for walling etc. It is fire-proof and is unaffected by the atmosphere and can be jointed with cement mortar.

Brick Ballast

Ballast should be made of first class well burnt or slightly over-burnt brick-bats to 40 mm gauge for foundations and floor concrete and 25 mm gauge for roof concrete. No under-burnt or "jhama" bricks (over-burnt porous) should be used. About 390 to 420 9-inch bricks and 420 to 440 Indian Standard Modular (IS) bricks make one cu. m of brick ballast. 280 to 350 9-inch bricks and 300 to 370 IS bricks made one cu. m of brick-bats. About 125 cu. m of brick-bats make 100 cu. m of ballast.

Hollow Clay Blocks (other than used for non-load-bearing partition walls) should each contain a proportion of solid material not less than one-half of the gross over-all volume, so disposed that the aggregate width of solid material (measured horizontally at right-angles to the face of the block as laid) shall nowhere be less than one-third of the total overall width of the block. No web should be less than 16 mm thick.

Hollow blocks have certain advantages over bricks : they are only about 1 3rd of the weight of the same number of bricks and they can be laid about four times as rapidly, and are of ample strength for all purposes for which ordinary bricks are used except under concentrated loads. They have the advantages of hollow walls as regards insulation against heat and sound.

Earthenware and Stoneware

Earthenware is made from ordinary clay, similar to that used for bricks, burnt at a low temperature and the articles are usually porous and weak. Stoneware is made from refractory clay mixed with crushed pottery and stone, burnt at a high temperature. Stoneware products are hard, compact and impervious to moisture. These products are usually salt-glazed to make them impervious.

2. SAND

Sand should be clean, sharp, angular (gritty to the touch), hard and durable ; free from clay, mica and soft flaky pieces. River or pit sand should be used and not sea sand except under special circumstances as it contains salt and other impurities which will affect the structures. All sands must be well washed and cleaned before use. Generally 4 to 6 per cent of clay and silt are permitted in sand as they improve the mortar to some extent but this percentage must not be excessive. A well graded sand should be used for cement work as it adds to the density of the mortars and concretes. Sand required for brickwork needs to be finer than that for stonework. For ordinary masonry work, for concrete, and first coat of plaster, the sand should pass through a sieve of 2 mm × 2 mm mesh, and for fine works, pointing, or second coat of plastering, sand should pass through No. 14 or 16 BS-1 mm sieve but should not have too much of finer sand. (See also under "Mortars and Concretes"). Crushed stone is not so satisfactory as natural sand since crushed stone contains a lot of fine particles like dust.

Sand which contains 90% of particles of size greater than 0.06 mm and less than 0.2 mm is fine sand. Sand which contains 90% of particles of size greater than 0.6 mm and less than 2 mm is coarse sand.—(C.P.W.D.)

The object of mixing sand in mortars is :

(*i*) To prevent excessive shrinkage and cracking of mortars in setting, especially in the case of fat limes which shrink very much while drying. Cements also shrink to some extent.

(*ii*) To improve the setting power of fat limes.

(*iii*) To improve the strength of a mortar as sand has greater crushing strength.

(*iv*) To increase the bulk and reduce the cost, especially in the case of cement mortars.

(More details are given under "Soil Mechanics," "Reinforced Concrete" and "Mortars".)

3. SURKHI

(Called *Trass*, or brick-dust in England.)

Surkhi is used as a substitute for sand for concrete and mortar, and has almost the same function as of sand but it also imparts some strength

and hydraulicity. Surkhi is made by grinding to powder burnt bricks, brick-bats or burnt clay ; under-burnt or over-burnt bricks should not be used, nor bricks containing high proportion of sand. When clay is especially burnt for making into surkhi, an addition of 10 to 20 per cent of quick-lime will improve its quality ; small clay balls are made for burning.

Surkhi for plaster may be made from slightly under-burnt bricks, and ground very fine ; this will improve the hydraulicity of fat lime. Surkhi must be well mixed with lime, preferably in a mortar-mill. (Test for under-burnt bricks is that they should not dissolve in water). Surkhi mortar gains strength if left immersed in water. Surkhi is not suitable for plaster exposed to weathering and humid conditions.

Experiments carried out at the Government Test House, Alipore, (Calcutta), and in the Punjab on lime/surkhi mortar showed that slightly over burnt bricks are the best. The surkhi in the experiments was ground so fine as to pass through a BS sieve No. 30 (500 micron).

Pozzolana—Is a volcanic substance found in a number of places but named from the deposits at Pozzuoli near Naples in Italy. It is mixed with lime to produce mortar. It is manufactured artificially in India by burning some types of clays and grinding it very fine. It is more or less like Surkhi. It can be used in all situations where surkhi can be used to give superior performance.

Surkhi makes cement mortars and concretes more water proof, more resistant to alkalies and to salt soluations than those in which no surkhi is used. Surkhi mixed in cement concrete has been used in some of the big dams and other massive works in India. This admixture is known to reduce the temperature rise during hydration in a mass cement concrete and reduce cracking. It is also useful in sea-water construction, in structures which are subject to attack from aggressive ground waters or industrial waters, and in hydraulic structures where water tightness is the first consideration. Surkhi mixed cement concrete is more plastic, bleeds less and segregates less as compared to ordinary cement concrete. A surkhi concrete of 25 to 50 mm slump is just as readily placed as a correrponding straight cement of much higher slum. The proportion of sunkhi recommended is 10 to 30 per cent of cement (cement will be proportionately less) but it must be ground as fine as the cement. Surkhi should not be added to aluminous cements. Surkhi is added both in mortar and concrete. The addition of surkhi is accompanied by a slight reduction in strength as surkhi attains its full strength only after one year. Surkhi concrete is subject to a slightly higher shrinkage than ordinary concrete. Surkhi is not a standerdized produce and its properties are widely variable. This is still in experimental stage in India and should be used with caution for important work. (See under "Extracts from some of the General Specifications adopted for the Lining of Bhakra Canals" in Section 17.)

4. LIMES

Lime is classified as fat, pure or rich lime and hydraulic lime. Fat lime is so called because it increase in bulk to two to three times its original volume when slaked. Fat lime is nearly white and it does not set under water but dissolves, while hydraulic lime sets and hardens under water.

Fat lime which is also called stone-lime or white lime is high calcium lime with about 6 per cent material insoluble in acid, chiefly obtained by burning (called *calcination*) in a kiln pure limestone, chalk or sea shells, etc. (calcium carbonate). By burning calcium carbonate, carbon dioxide is driven off as a gas leaving calcium oxide or *quick-lime* in the form of lumps. When water is poured over quick-lime it almost immediately cracks, swells and falls into powder with a hissing and creaking sound, slight explosions and considerable evolution of heat and steam. The process is called *slaking* or *hydration*, and the powder produced is called *hydrated lime* or *slaked lime* (calcium hydroxide). Quick-lime should be slaked as early as possible after it is burnt in a kiln. Over-burnt or under burnt pieces or lumps should be picked out and removed before slaking.

Quick-lime if left exposed to the air will absorb moisture and carbon dioxide and become an inert powder of calcium carbonate or chalk having no cementing power. Therefore, lime should be stored in an enclosed space in large heaps and air excluded as far as possible. Unslaked lime kept in air tight vessels, and slaked lime packed in gunny bags and stored in dry place will keep sound for months. All lime that has been in any way damaged by rain, moisture or dust should be rejected.

Test for freshness of white lime. Unslaked lime weights about 1050 kg/cu. m, expands on slaking and then weighs 640 kg/cu. m when fresh, increasing to about 800 kg/cu. m after 10 days.

Hydraulic lime is obtained by burning *kankar* or clayey limestones. Lime is considered to be hydraulic when it sets under water within 7 to 30 days. Lime is called feebly hydraulic, moderately hydraulic or eminently hydraulic according to its readiness to set under water and its properties which depend upon the proportion of clay in the lime, which varies from 5 to 30 per cent. The larger the proportion of clay, the more sluggish the slaking and the greater the hydraulic property. Hydraulic lime slakes very slowly taking several hours or even days depending upon its composition, and without producing much heat, noise or change in bulk. Slaking is done in the same manner as for fat limes but only just enough water is added for hydration and lime is turned over with spades. Excess of water will harden it and make it useless. Slaking action is accelerated if lime is initially pulverised in a grinding mill.

Hydraulic lime should be slaked just before use and not immediately after burning, and then passed through a 3.35 mm sieve and stored in a compact heap in an air-tight dry place.

Hydraulic lime is suitable for works under water and for all positions where strength is required as it has much less tendency to shrink or crack than fat lime, and addition of a small proportion of sand improves its qualities. It has to be a ground to a very fine powder for plaster work.

Methods of Slaking lime

Platform slaking. Lime is spread over a dry nonporous platform in a 15 to 23 cm layer and water poured over it generously through a nozzle, and heaps turned over and over between each application of water until the lime disintegrates to a fine powder.

Tank slaking. The lime is placed 30 cm deep in a drum or a tub with about 90 cm of water and allowed to stand for about 24 hours or such longer period as may be necessary to slake the lime completely. It is better to add lime to the water and not water to the lime. The mixture should be well stirred.

Lime is considered to be completely slaked when the temperature of the lime and the water ceases to rise and any further addition of water also produces no further chemical action or heat, but as a precaution, water should be allowed to stand on for 12 hours or more for white limes and until the normal temperature is restored. A vigorous slaking with heat and noise indicates a high calcium content. After slaking the lime should be screened through a 3.35 mm sieve or kept in excess of water to form lime putty according to the requirements. Limes must be thoroughly slaked especially for plaster work which is also ground very fine, any unslaked particles left will produce "blisters".

Setting and Suitability of Limes. In the case of pure lime, the hardening takes place partly by the absorption of carbon dioxide from the air and partly by drying which is facilitated by dry condititions, and the setting action is very slow. Slaked fat lime has a great tendency to absorb carbon dioxide from the air when it dries and hardens but it shrinks and cracks on drying. This lime is mixed with large quantities of coarse sand, up to two to three times it volume, in the preparation of motar which makes the mortar porous and increases the absorption of carbon dioxide for the hardening process, and also prevent shrinkage. Mortar from a mixture of fat lime and sand will set in thin wall joints and under heavy pressure. In thick-wall construction, the mortar in the interior very often never sets or hardens but crumbles into a friable powder and does not acquire any strength. As such, fat lime is suitable only for thin masonry wall joints and for interior plaster and not for works in wet foundations or under water as it dissolves in water and does not weather well in exposed positions.

Hydraulicity and setting properties of fat lime can be improved by the addition of surkhi and grinding the mixture in a mortar-mill, (see under Surkhi). An addition of 10 to 15 per cent of cement to a fat lime mortar also improves its quality considerably (see under cement/lime mortars in Section 7).

The hardening of hydraulic lime does not depend on the absorption of air; the setting of hydraulic limes and cement is facilitated by the presence of water. The setting action of hydraulic lime is much quicker than that of fat lime. Only eminently hydraulic lime is suitable for under-water works but it should not be immersed within 48 hours.

Burning of Lime. Limestone is burnt in clamps or kilns. Fuel used is generally, coal-dust or fire-wood. Cowdung or litter should not be used with kankar. A clamp consists of a heap of limestone and coal stacked in alternate layers and is used for burning only small quantities of lime as it is a wasteful method.

Out of 100 parts of pure limestone burnt, only 56 parts of lime are left behind.

After burning kankar lime it should be ground without delay as it deteriorates rapidly if left unground during the rains. The limes (both kankar and white) after being ground dry should be carried to the site of work in gunny bags, and not dumped or stacked on the ground.

Poor lime, also called Meagre or Lean lime, contains from 10 to 40 per cent impurities insoluble in acids such as sand and stones, takes longer to slake, does not increase in bulk, to such an extent (less than twice) as pure lime and has inferior plasticity ; colour may not be white.

Shell lime freshly slaked is used for polished plaster and white washing. It comes under the catagory of 'fat limes'.

Grinding and Mixing of Lime Mortars. Lime mortars require grinding to slake the unslaked particles and to make an intimate mixture of the materials. Lime/surkhi mortars are ground in two operations, first only the lime and surkhi are ground together and then sand is added and again ground. For big jobs grinding is done in a bullock-driven mortar-mill (or machine). In a mortar-mill the diameter of the track should not be less than 8 m. No piece larger than 6 mm should be introduced in a mortar mill. For small jobs grinding can be done by ponding the ingredients in a small pit and mixing. Small quantities of mortar can be mixed by repeatedly turning over the materials with a shovel, and afterwards with a trowel, so as to mix them very thoroughly.

Artificial hydraulic lime successfully used for works under water can be made as follows :—Five and a half parts by volume of ordinary slaked lime in paste is mixed thoroughly with one part of clay and the mixture made into balls, when thoroughly dried, are burnt in a kiln. The burnt balls are ground into powder and mixed with sand in the proportion of 1 : 2 for mortar for superstructure work and in the proportion of 1 : 3 for mortar in concrete and footings. Hydraulic lime can also be made by burning an intimate mixture of 4 parts of slaked fat lime and 1 part of clay. It has a tensile strength of about 14 kg/sq. cm.

Deposits of pure limestone and chalk are white or whitish brown or of grey colour. Clayey and silicious limestones have light brown to dark brown colour and a dull earthy appearance indicating the presence of clay. Good hydraulic limestones show a bluish or yellowish brown colour and compact texture. A freshly fractured surface when damped with water has clayey taste and an earthy smell. Pure white limes dissolved in dilute hydrochloric acid, leave very little residue and have vigorous action (effervescence). The quantity of sand and clay in lime could thus be ascertained. White lime which gives a residue of more than 10 per cent by weight of impurities should be rejected. The hardest varieties of limestones can be scratched by a pen-knife.

Tests for Strength of Lime Mortars

Two bricks are joined flat in a cross fashion one over the other with 12 mm mortar joint. Bricks are thoroughly soaked in water before joining and cured for 7 days after they are jointed. Load required to separate them at joint gives the adhesive strength of mortar which should not be less than 1.4 kg/sq. cm for 1 lime to 2 sand mortar. For testing tensile strength, briquettes are made (as for cement test), and cured for 24 days by immer-

sion in water. A good hydraulic lime should have an ultimate tensile strength of at least 7 kg/sq. cm and a fat lime 2.8 kg/sq. cm, with 1 : 3 lime : sand mortar. The compressive strength should be 4 to 5 times its tensile strength. Mortar consisting of 1 part good quality kanker lime and 1 part sand should develop a compressive strength of over 50 kg/sq. cm after 3 months and twice that after two years. A pure surkhi mortar gives a breaking strength of about 5.6 to 6.3 kg/sq. cm if left in dry air, and 21 to 25 kg/sq. cm if left immersed under water.

Compressive strength of 1 : 4 cement-sand mortar after 3 months is over 210 kg/sq. cm.

Simple method of testing the strength and suitability of a particular limestone or kankar is to burn the limestone, produce lime, mix it with the required proportion of sand and test the mortar as described above.

Plaster of Paris is calcined gypsum. Mixed with ordinary lime it is used for repairing holes and cracks in wooden or plastered surfaces, and for making moulds and ornamental works. When mixed with water its swells slightly and sets rapidly.

Gypsum is natural calcium sulphate and occurs as a soft stone which is from white to dark in colour. It is used mainly in the manufacture of cement.

Barium Plaster is employed as finishing coat to X-ray room walls. It consists of 1 part cement, 2 parts of fine and 2 parts of coarse barium sulphate powder. Thickness varies from 6 mm to 12 mm.

Kankar

Kankar is extensively used for producing hydraulic lime. The nodules should have a blue grey fracture, free of any sand grains or mud sticking to them, and broken to pass a 12 mm gauge before being calcined.

Kankar is a nodular variety of limestone which is of spongy nature, found in almost all parts of India containing some quantity of clayey and silicious matter. It is found either in layers or blocks, or in separate nodules. The block form occurs as solid deposits at various depths, and the nodular variety is generally found scattered on the surface or in small thicknesses about a metre or so below the surface in the low-lying portions of the catchments of nallas and rivulets. The nodules are of sizes varying from 10 mm to 100 mm. Nodular kankar is superior to block kankar but is not available in large quantities. Shining or glittering particles in a fresh fracture indicate presence of sand. The proportions of clay and sand can be determined by dissolving the sample in powdered form in dilute hydrochloric acid and determining the residue left. "Bichwa" kankar as known in the Punjab and U.P. is considered to be the best. (See also under "Kankar Roads".)

5. MISCELLANEOUS MATERIALS

Coke is used for smelting iron and in other industries connected with metallurgical works. It is produced as a "by-product" in the process of tar-coal manufactured from bituminous coal. It is soft coke or hard coke. Coke consists of dark grey, brittle, porous nodules of irregular shape containing 80 to 85 per cent of pure carbon, and slightly lighter than water.

One tonne of average Indian coal yields approximately : 0.5 tonne of coke, 55 litres of coal tar and oils.

Steam coal or soft coal is bituminous coal which burns with a smoky flame.

Asbestos is a fibrous mineral (which can be separated into fibres) of a white, grey or brown colour. It has excellent fire resisting properties and is used for heat, sound and electric insulation. It is also acid and water-proof. Asbestos paint is made for heat proofing and sound proofing. It does not shrink, swell, crack, crumble under heat or cold. The superior quality can be spun into coarse threads and woven into cloth.

Asbestos-Cement is a combination of asbestos fibres and cement ; about 15 p.c. of asbestos fibres are mixed with cement. Asbestos cement is very durable and possesses great resistance to transverse and tensile stresses. It is commonly used in the form of roof sheets and pipes. Asbestos-cement products can be cut with a hacksaw.

Clinker is the waste from furnaces and resembles burnt coal. It is used as an aggregate for inferior concretes and is much lighter and porous. Generally has ashes mixed with it.

Furnace Slag. Slag is a waste product obtaining from blast and cupola furnaces used for the manufacture of cast iron. It is crushed when cooled to make aggregate for concrete, railway ballast, and blast furnace cement, etc. The blast furnace slag when finely ground exhibits cementicious properties. Makes a good aggregate for fire-resisting purposes. It is most suitable for the manufacture of partition slabs or concrete blocks, but not for weight bearing structures as the slag is liable to contain sulphur.

Coke Breeze is a similar product to clinker, obtained from gas works and coke ovens, it is light in weight and porous and can be used as aggregate for slabs and blocks in wkich nails can be driven. Slabs or blocks of light weight concrete made from coke breeze, slag or cinders have heat insulating properties.

Furnace slag and coke breeze should never be employed for reinforced work.

Breeze is a general term for furnace ashes.

Gutta Percha is a substance somewhat like India rubber but stronger and less elastic and is used without any admixtures. It is obtained from the exudations of a larger number of species of trees growing in Malaya. It is tough and hard like wood when cold but becomes plastic when warmed when it can be moulded into varities of forms. It is superior to rubber and is generally used for the manufacture of cables under water and such type of works.

Carborundum is a polishing abrasive (stone) made artificially by mixing in certain proportions sand and carbon, and heating the mixture to very intense heat in an electric furnace. It is used for making grinding wheels for grey iron castings; hard alloys, stones and glass etc., and for polishing cement concrete floors.

Emery is a variety of carborundum stone and is very hard abrasive material. It is chiefly used for making grinding wheels, emery cloth and

emery paper for polishing glass and other hard materials. Emery ground to powder is glued to an ordinary cotton cloth or paper.

6. MIXTURES FOR MORTARS & CONCRETES

The function of a mortar is to (*i*) unite bricks or stones in the construction of brickwork or masonry ; (*ii*) form an even bed between different courses of masonry work and to distribute the load evenly on the lower layers ; (*iii*) form matrix to hold pieces of stones (aggregate) together and form a solid mass of concrete ; and (*iv*) cover exposed surfaces of walls and joints with plaster or pointing.

For works of importance where strength or hydraulicity is required, pure hydraulic lime may be used without any admixture of sand.

20 per cent of the cement used may be replaced by finally ground Surkhi where salts are found, and cement proportion should be high.

The following proportions are suggested which are by volume of dry materials ; cement is by weight or by bags :

Mortars for Foundation Concrete (All proportions by Volume)

Cement	Sand	Lime	Surkhi	Flyash	
					The quantity of mortar to be used will depend on the grade of aggregate.
—	1.5 to 2	1	—		(Type "A") for dry sub-grade, with sub-soil water level never within 2.5 m of the foundation level. Normally suitable for buildings not more than 3 storeys high.
—	1 to 2	1	1	—	
	—	1	2	—	
1	12	3	—		
1	3 or 4	—			(Type "B") for moist sub-grade with high sub-soil water usually 2.5 m or less below foundation level. The corresponding concrete mix will be 1:4:8 or 1:3:6.
1	3	—		1.5	
1	4	—	—	2	

Fat lime concrete is not suitable for use in large masses.

Mortars for Masonry in Foundation Plinth

1	3	—			(*i*) Type "B", for heavy and medium loading (40 to 80 tonnes/sq. m)
1	1	1	—		
1	6	1	—		Type "A", "B" and Ditto......
1	3	—	—	1.5	Type "B" and Ditto......
—	2	1	—		Type "A", lighter loading
--	1	1	1	—	Type "A" and Ditto......
	—	1	2	—	Type "A" and Ditto......
1	9	2	—		Type "A" and Ditto......

Mortars for Masonry in Superstructure

Cement	Sand	Lime	Surkhi	Flyash	Lime Putty	Cinder	
1	6	—	—	—	1	—	Very heavy loading 80-100 tonnes/sq. m
1	6.5	—	—	3.6	—	—	
1	3	—					(*ii*) Archwork and RC Brickwork
1	3	—	—	1.5	—		
1	9	—		—	2	—	Heavy loading 60-80 tonnes/sq. m
—	1	—	1	—	1	—	Medium loading 40 tonnes/sq. m
—	1	—	—	—	1	1	
		—	2	—	1	—	
				—	1	2	

Cement	Sand	Flyash	Lime Putty	
—	2	—	1	
1	9	—	2	
—	1	1	1	
1	6	—	1	Light loading below 40 tonnes/sq. m
—	3	—	1	Non-load bearing partitions

Note :— (*i*) Use of high alumina cement and puzzolanic cement is recommended.

(*ii*) 10% lime can be added for better workability.

Mortars for Plasters (Proportions by Volume)

Cement	Sand	Lime	Surkhi	Flyash	Lime Putty	
1	6	—	—	—	1	External plaster below Damp-Proof course.
1	9	—		—	2	Ditto. above Ditto.
—	2	—		--	1	Internal plaster on all walls.
1	10	3	—			
—	1	—	1	—	1	
	—	1	—	1	1	
		—	2	—	1	
			—	2	1	
1	—	9	—	—	2	
—	4 to 6	—			1	Under-coat
—	9	—			2	Finishing coat

Mortars for Pointing

Cement	Sand	Lime	Surkhi	Flyash	Lime Putty
1	3 to 6	—	—	—	1
1	10	—	—	—	3
1	9	—		—	2
1/8	—	—	1 to 2	—	1
—	1	—	1/2 to 1	—	1

Quick lime shall be supplied in the form of lumps when brought to site of work and not in powder form. Quick lime deteriorates rapidly and, therefore shall be used as quickly as possible. **Hydrated lime** shall be used within 3-4 months of its date of manufacture. It shall be stored in the same manner as cement. Slaked lime shall be screened through 3.35 mm sieve and the residue which do not pass through the sieve, shall be rejected. Slaked lime shall then be run into *putty* before use in mortar. Lime putty shall be kept wet till it is completely used. It can be stored without getting spoiled for about a fortnight provided it is protected from drying and stiffening. For maturity of putty at least 2 to 3 days shall be allowed in case of fat lime, but not more than 1 or 2 days in case of semi-hydrated lime.

Surkhi shall be ground so fine as to pass through 3.35 mm sieve with at least 50% of it passing through 1.70 mm sieve.

Cinder shall be obtained from furnace of steam boilers using coal fuel only. It shall pass through 3.35 mm sieve, with at least 50% of it passing through 1.70 mm sieve. Cinder obtained from brick kilns shall not be used. 10 cu. metres of coarse cinders give about 6.7 cu. m of cinder powder. More of cinder gives a leaner mix.

Flyash is finally divided residue resulting from the combustion of pulverised coal in boilers, received from Power Stations. It is meant for use as a part replacement of fine aggregate in mortar and concrete with a view to improving grading and to make use of its pozzolanic properties. The recommended magnitude of replacement is up to 20%.

The fineness or presence of coarse bottom ash shall be checked by mixing the flyash in a bucket half full of water and passing the resultant slurry through a 150 mesh sieve. If no residue is left, the flyash is treated as of requisite fineness. In case, the residue is left on the sieve, the flyash shall be rejected.

Lime mortar shall be used as soon as possible after mixing or grinding. As a rule mortar shall be used on the day it is made. If eminently hydraulic lime is present as an ingredient, the mortar shall be used within four hours after mixing. Lime mortar made with semi-hydraulic lime or fat lime and pozzolana as ingredients, shall be used within 36 hours of mixing and lime-sand mortar within 72 hours provided it is kept damp with wet sacks and shall on no account be allowed to dry till used. Gauged mortars with cement shall be used within 2 hours after the cement has been mixed wet.

Preparation of lime paste or putty. Lime is slaked in a tub, mixed with a large quantity of water, stirred and screened through a fine 25 BS or 600 micron screen or a coarse cloth. Again kept immersed in water for 3 days, when the excess water is poured off.

Cement/Lime Mortar. See also under "Plastering" in Section 7.)

A small quantity of hydrated white lime in a cement mortar increases its plasticity and makes it easier to use and at the same time reduces shrinkage cracks, and improves its qualities of water-proofing. It has been reported that experience gained over several years in Europe indicated that cement/lime/sand mortars are more successful as renderings than cement/ sand mixes. A proportion of 1 : 3 : 10 cemeat/lime/sand is usually recommended. Lime/cement mortars should be mixed in quantities small enough to be used within 2 or 3 hours, before the cement has set.

Wood float finish is preferred to steel trowelling.

The addition of not more than 20% of hydrated lime slightly increases the strength of the concrete but this concrete is not suitable for positions where water is met with.

General Remarks for Preparation of Lime Mortars. (*i*) Use thoroughly slaked white lime which has been kept in water for 24 hours for masonry work and 36 hours for plaster work after slaking, and then screened through a 3.35 mm sieve.

(*ii*) Fat lime or surkhi mortars, without an admixture of cement, are not suitable for exposed works. Fat lime is good for interior plaster.

It has been found that even 1 : 8 cement mortar is stronger than 1 : 3 lime mortar by about 50 per cent.

(*iii*) Mortars made with hydraulic lime should not be exposed to running water unless set.

If one kg of washing soda is mixed in 14 litres of water used for the preparation of cement/sand mortar, it will make a water-tight mortar.

(*iv*) Where surkhi and lime are used they should be thoroughly ground together. Surkhi must be ground very fine.

(*v*) Mortar for pointing is ground very fine.

(*vi*) **Sand** used for mortars for masonry work should consist of sharp angular grains. Rounded grains do not interlock sufficiently to produce a strong mortar. Coarse sand produces stronger mortar than fine sand ; fine sand requires more water than coarse sand and consequently the mortar is less dense. A well graded sand in which the percentage of voids is the minimum produces the best mortar. Use graded medium-fine sand with cement, and coarse sand with fat lime mortars. Coarse sand requires greater proportion than fine sand. For mortar and concrete, not more than 30% by weight should pass the 0.85 mm screen and not more than 60% should pass the 1.60 mm screen. (Size of sand particles is given in "Soil Mechanics".)

(*vii*) Fine sand is used for plaster and pointing, passing through No. 16 BS (1 mm) sieve, but not more than 10% should pass through No. 100 BS (150 micron) sieve. (This fine sand is not suitable for RCC works). Very fine sand will produce cracks.

(*viii*) Cement which has deteriorated in respect of quick-setting can be used in place of lime and makes better mortar than fat lime.

(*ix*) Chopped jute 6 to 12 mm long (or hair) is added to lime mortars for plastering in the proportion of about 5 kg to one cu. metre of slaked lime.

(*x*) Addition of a little powerded soap-stone increases the whiteness and polish of a lime plaster.

Cement mortar. The unit of measurement for cement is a bag of cement weighing 50 kg and this is taken as 0.035 cu. m. Sand is measured in boxes of size 25 × 35 × 40 cm on the basis of its dry volume. In case of damp sand, its quantity is suitably increased as explained in Section 8. Cement mortar shall be used as soon as possible after mixing and before it has begun to set, (and in any case within two hours, after water has been added to the dry mix), only that quantity of mortar which can be used within 30 minutes of its mixing shall be prepared at any time, care shall be taken not to add more wuter to the mixing than which shall bring the mortar to the consistency of a stiff paste.

In case the mortar has stiffened because of evaporation of water, it may be retampered by adding water frequently as needed to restore the requirements of consistency, but this retampering shall be permitted only up to two hours.

7. STONES

Geological Classification of Rocks

It is estimated that three-fourth of the land area of the globe is underlain by sedimentary rocks and the other fourth by igneous and metamorphic rocks. Main three formation groups :—

Igneous rocks : Are of volcanic origin, formed as a result of consolidation of molten materials either in the interior of the earth's crust or upon its surface. They represent a crystalline glassy or fused texture. Generally, igneous rocks are hard, tough, dense, impervious, strong and durable. Granite, Dolerite, Basalt, Trap, are examples. Form excellent concrete aggregate.

Acid rocks. Igneous rocks containing over 65 per cent of silica. Compared with basic rocks, acid rocks are of lighter colour and in coarsely crystalline varities ; free silica or quatz can be seen without the use of a lens. Granite is an acid rock. *Basic rocks* : Igneous rocks containing less than 52 per cent of silica. Compared with acid rocks they are of darker colour, and only rarely show free silica or quartz. Basalt and dolerite are basic rocks.

Sedimentary or Aqueous rocks : Are formed by the sediments deposited chiefly by water and to some extent by wind and ice, (sand, gravel, clay, cemented together by silica, lime, etc.) They represent a bedded or stratified structure in general, the individual beds lying one above another, often being distinguishable by differences in colour, texture or composition. May be close grained, compact or open textured. Sandstones, Limestones and Shale are examples. Gravel, sand, silt, clay and peat are considered as uncemented and unconsolidated sedimentary rocks.

Metamorphic rocks : Are either igneous or sedimantary in their origin but subsequently changed due to movements of the crust as a result of metamorphic action of heat and pressure. Metamorphic rocks have a foliated structure in general and also show layers of stratification which are not always uniform. These rocks are hard and durable. Slates, Schists Gneisses, Quartzite, some hard Shales and Marbles, etc., are formed in this way.

Definitions of Some Terms

Bed-rock. Any hard rock-bed underlying soft deposits.

Bedding plane. The plane of junction between adjacent strata in deposits of sedimentary origin.

Diluvium. Glacial deposits.

Drift. All superficial deposits of the earth's crust such as boulder clay, sands and gravels, alluvium.

Drys. Seams containing materials not thoroughly cemented together; *crowfoots* are veins containing dark-coloured uncemented material.

Eluvium. Superficial deposits formed of fragmental material from solid deposits which have not been transported by wind or water, but may have moved down hill slopes under the action of gravity when in a water-logged condition.

Fault. A dislocation of continuity of rock strata as a result of cracking of the earth's crush.

Fissure. A crack, break or fracture in rock mass.

Freestone. A rock used for building which can be quarried by splitting easily along certain bedding or joint planes. Also a rock of even texture which can be ornamentally carved for building.

Mineral. A homogeneous substance of definite chemical composition and constant physical character. Deposits forming the earth's crust may be composed of a mineral or of an aggregate of minerals.

Rubble. Irregular shaped (natural), but approximately cubical pieces of stones.

Quoins. Corner stones having two faces made plane.

Flag-stones and Paving sets. Flat stone slabs used for floorings or pavings.

Structure of Rocks

There are two main divisions :—

Stratified and Unstratified :

Stratified rocks are sedimentary rocks which were deposited in layers, and can be easily split along such layers, which are called "planes of cleavage". Unstratified rocks are of igneous origin and are stronger.

Natural Bed of a stone is the original position occupied by it during its formation which may be either horizontal or at an angle with the surface of the earth. For a metamorphic rock, the plane of cleavage or the plane of foliation is treated as a natural bed, but in igneous rocks, natural bed is difficult to be traced and is also of less importance. A stone is strongest when resting upon its natural bed. Stones should be placed in a building with their natural bed at right angles to the direction of the load or pressure. If the plane of foliation is parallel to the face the layers will tend to flake off one after another. In string courses and cornices, natural bed is kept vertical, while in arches it is radical and at right angles to the thrust of the arch.

Stratified rocks are often traversed by cracks which are called *veins* or *dykes*.

Seasoning of stones. Limestones, Sandstones and Laterites when freshly quarried contain some moisture called "quarry sap", and in this state they are softer and can be easily worked. As the moisture evaporates, the stones become harder. Therefore, these stones should be exposed to open air (and not sun) for two seasons before use in masonry.

CHARACTERISTICS OF THE PRINCIPAL BUILDING STONES

(For Specifications of Stone Masonry, see under "Masonry Structures.")

Granite : A hard rough-surfaced unstratified igneous rock that occurs in comparatively large masses that have been formed at considerable depth. The best forms of granite are amongst the strongest and most durable stones. The texture of granites vary from coarse grained to fine grained; fine grained is more valued and can be easily worked and polished. It is usually uniform in colour and texture (with mottled appearance) and weathers well. The colour varies from grey, green, brownish or reddish to black. Being a heavy stone, it is generally employed in very exposed and massive structures and those subjected to heavy loads, and for roads metal. Excellent building material.

Gneiss ; Is a metamorphic (may be either sedimentary or igneous) form of granite having the same mineral composition. It has a stratified structure and planes of foliation along which it can be easily split up. The principal minerals contained are quartz and felspar or feldspar), with a black or white mica, forming irregular streaks nearly parallel to one another composed alternatively of light coloured and black minerals. Quarrying is easier and yield good paving blocks and road metal. Because of its unclean appearance it is not very suitable for face work.

Schist : Is almost like gneiss ; a foliated metamorphic rock usually thinly laminated, in which the minerals are arranged in sub-parallel bonds of streaks and in which micas are prevalent. It can be split up in thin irregular plates.

Basalt : It a basic volcanic rock, fine grained, of glassy texture, very compact, hard and heavy which when of good quality breaks with a clean fracture and rings when struck with a hammer. It is hard to work and durable ; varies much in quality and is not obtainable in large blocks. Colour varies from greenish grey to dark grey and sometimes bluish black. Red and yellow varieties are softer. These stones are mostly suitable for road metal, flag stones, and aggregate for concrete although used for rubble masonry work as well where no carving or moulding is required. *Trap* is almost the same.

Conglomerate : Is a mass of sand, gravel, rounded pebbles, etc., of various rocks of sedimentary or volcanic origin embedded (cemented together) in a matrix of finer material.

Limestones : Limestones belong to the group of stratified rocks, consist essentially of carbonate of lime intermixed with certain impurities (silica, alumina, iron, etc.), and minerals. There are many varities of limestones differing largely in colour, texture, hardness and durability Limestones are easy to work, and are a good building stone as regards durability and weathering qualities if compact grained, dense and crystalline in texture ; soft qualities do not weather well. A knife scratch leaves a well marked line filled with white powder. Effervesces in contact with dilute solution of acid (which is not the case with dolomite), therefore, it is not suitable for use near industrial towns. Colour is generally white, grey, bluish yellow or even black, and some varieties take good polish. The crushing strength of limestones vary considerably because of varied composition and quality, therefore, the stone should be tested before deciding on a safe compressive stress.

Oolite is a variety of limestone.

Chalk is a white limestone composed of almost pure carbonate of lime. It is dry porous lime rock.

Marble : Is a compact crystalline carbonate of lime or limestone formed by the metamorphic action (re-crystallized by heat or pressure). A good marble is the strongest variety of limestone and is one of the most durable of all stones. It is obtainable in varieties of colours, plain or mixed, from white to black. Can be easily sawn and carved and takes a high polish ; is suitable for ornamental or superior type of building work.

Argillaceous or Clayey Limestones do not weather well but are most suitable for the manufacture of hydraulic limes and cement. *Kankar* is an example—has been described before.

Sandstones : A sandstone is a soft or moderately stratified (sedimentary) rock consisting of grains of sand or quartz cemented together by silica, alumina, iron oxide, etc. Strength and durability depends upon the material cementing the grains of sand together. There are varities of sandstones, fine-grained and coarse grained, compact and open-textured or porous. Best sandstones have very fine grains. The characteristics of strong and durable sand-stone are sharpness or fineness of grains, clear and shinig translucent appearance on a freshly cut surface, while a soft and parishable stone shows round grains, a dull matt surface on a fresh fracture, and a tendency to split into thin layer. Sandstones vary much in quality and many varities are too soft, friable and porous to be of much use ; those with least quantity of lime and iron are the most durable. Good sandstone is found in thick strata from which it can be quarried in large blocks, and is generally easy to work and dress. A sandstone must be used on its "natural bed".

A good accelerated test is to boil 6 mm size pieces in water. Rapid disintegration indicates a weak stone with a tendency to weather rapidly and is not suitable for engineering works.

Sandstones and limestones should not be used together in a structure as the chemical action formed in limestones due to atmospheric reactions will disintegrate the limestones. Sandstones exhibit different shades of colour such as white, yellow, light grey or brown or even red and pink. The colouring matter is chiefly iron. Sandstones resist heat well and are extensively used as local building materials, but are not generally suitable ... metal or railway ballast unless of the harder quality. They do not

Quartz : A metamorphosed mineral rock composed entirely of silica.

Quartzite (including Hornstone or Flintstone) : Is a from of silicious sandstone composed of quartz grains cemented together with silica. It is the hardest and strongest rock with stratified structure and crystalline texture, most compact and dense than sandstone. Quartzites are hard to work and break up into irregular cubes with uneven lustrous. Colour varies from brown red to yellowish white. Used for stone walling where quarried in large blocks, also for concrete aggregate and road metalling.

Laterite : Laterite is a soft sandy-clay stone with a porous or cellular structure, a mixture of red and yellow residual soils or surface products that have originated in situ from the atmospheric weathering of rocks, impregnated with iron, streaked red and yellow brown to black in colour. It is easy to work and can be quarried with the aid of pick axes. When freshly quarried the stone is wet and soft, and should not be used until seasoned for a month or two when it becomes harder. Is generally procurable in large slabs and can be easily cut into blocks. The slabs selected for work should be compact in texture. It is suitable only for light road and building work of inferior type.

Slates : Are metamorphic laminated clay rocks in which fine cleavage has arisen by earth pressure. The stone can be split into very thin slabs along the planes of cleavage, but they split also in directions other than that of bedding. A good slate should be fine grained, compact, light and should not absorb any water and should give a sharp metallic ring when struck. Colour is grey, blue, black, purple or greenish. A common test for roofing slates is to place one on edge so half its depth in water for 12 hours. If the water approaches the top of the slate, it should be rejected ; if it does not rise beyond 3 mm, may be considered as practically non-absorbent. Or alternatively, a good slate after 12 hours' soaking should not have absorbed more than 1/200th part of its weight. They are generally used for roofing, damp-proof courses and for flooring.

Shale : A compressed and laminated clay, or a kind of clayey stone not so hard as slate, with or without associated organic matter, splitting readily into thin plates.

Flint : Rock or boulder consisting of very fine crystalline silica and sometimes showing remains of sponges and other organisms. Has a conchoidal glassy fracture.

Mica Schist : Is a metamorphic composition of mica and quartz, crystalline and fine grained in texture. Colour is grey greenish, yellowish or brownish black. It is not suitable for measonry work but can be used as a road metal for light traffic roads.

Moorum : Is disintegrated granite or trap which has been weathered in situ : a gritty silicious material with lumps or stones not exceeding 20 mm in size and with a natural admixture of clay of calcarious or laterite origin. Used for top dressing of metalled roads or for filling floors or other such types of jobs. The upper surface of moorum deposit is softer than the lower and the total thickness is not generally more than about 1.5 m.

Pumice : Is a highly vesicular "lava froth" formed by escaping gases. It is a very light rock which can float on water. Pumice stone is used as aggregate for light weight concrete manufacture and for scrubbing concrete floors.

Requirements for Building Stones

The chief requirements of a building stone are : strength, density and durability combined with reasonable facility for working. A good building stone should be hard, tough, compact grained and uniform in texture and colour. Stones with uniform colour are generally found to be durable. Red and brown shades and mottled colour indicate the presence of injurious materials. Generally speaking, the heaviest and compact grained stones are the strongest and most durable ; a building stone should have a crushing strength of at least 100 kg/sq. cm. A crystalline stone is superior to a non-crystalline one and firmer the crystalline texture, the stronger it is. Igneous and metamorphic rocks are generally heavier and more durable than sedimentary rocks. A stone absorbing less water is stronger and more durable as it will have less action of rain water. A good building stone should be free from decay, flaws, veins, cracks and sand-holes.

The surface of a freshly broken stone should be bright, clean and sharp and should show uniformity of texture without loose grains, and be free from any dull chalky or earthy appearance. Stones showing mottled colour should not be used for face work. Free stones are useful for carved work. Stones are termed "freestones" if granular in structure with no planes of cleavage and therefore no tendency to split in any direction. Fine-grained freestones are used for carved or moulded work as it is possible to obtain much finer arrises than from the coarser grained varieties.

Stones should be properly seasoned by exposure to the air before they are put in a structure, as stones increase in durability after quarrying if well seasoned, especially limestones, sandstones and laterites. Stone newly quarried contain quarry-sap and can be more easily worked in this condition. The hard stones such as granite, are most durable with a rock-face finish, while the softer and more absorbent stones are usually most durable with a sawn or rubbed surface. The estimated life of granite, gneiss and good sandstone buildings is considered to be well over 200 years, while limestones and weaker types of sandstones hardly last for 50 years. Harder varieties of crystalline stones can be polished well.

The strength of a stone is greatly reduced under following conditions:

(*a*) Alternate wetting and drying, especially sand and limestones. Stones in wet condition show a lower crushing strength than when they are dry ; strength may be reduced by 30 to 40 per cent.

(*b*) Impact and intermittent loads as in the case of machine rooms and piers or abutments of bridges.

(*c*) Fire brings about rapid destruction of stones by disintegration.

Simple Field Tests for Durability of Stones :

(*a Crushing test* : The crushing strength of a stone greatly depends upon its texture and specific gravity. A stone of even texture and of specific gravity greater than 2.7 can take heavy loads. Safe compressive loads on stones should be taken not more than one-tenth of the crushing loads determined by cube test. Stones generally begin to crack or split under about half of their crushing loads.

(*b*) *Porosity or absorption test* : Porous stones such as coarse grained sandstones should not be used. A good building stone should not absorb more than 5 per cent of its weight of water after 24 hours immersion. Any stone absorbing more than 10 per cent or having specific gravity less than 2·5 should be rejected.

(*c*) *Structure test* : Small pieces of the stone are kept for about an hour in a glass of water and then shaken vigorously. If the water gets dirty it shows the stone particles are not properly cemented together.

(*d*) *Acid test* : A small sample is immersed into 1 per cent solution of hydrochloric acid and kept for about seven days. During this time the solution is frequently agitated. If the sample has still maintained its edges and corners as sharp as before, the stone will weather well. If a drop of weak sulphuric or hydrochloric acid on a piece of stone causes effervescence, the stone contains chalk and is poor in weathering qualities unless it is marble.

(*e*) Hardness may be tested by scratching with a penknife, which should not make an impression on a hard stone.

(*f*) Toughness may be tested by breaking the stone under a hammer. A hard and tough stone is required for road mental.

Chemical Classification of Rocks according to their chief constituent rock forming minerals :—

(*a*) *Silicious rocks* : These rocks have silica (and, quartz and flint) as their principal constituent and are very hard and durable, unaffected by weathering. Chief types of silicious rocks are Granities, Traps, Quartzites and Sandstones.

(*b*) *Calcareous rocks* : Calcium carbonate or lime is the main constituent of these rocks. Crystalline and compact types are hard and durable. Clay is very often found mixed in such rocks. Marbles and limestones are calcareous rocks.

(*c*) *Argillaceous rocks* : Rocks of the clayey types which are more or less composed of alumina mixed with small quantities of other minerals. Slates and laterites belong to this group.

Rock-forming Minerals in Building Stones :

Quartz-Silica, Felspar-Alumina, Lime and Magnesia are mineral earths. Silica and the silicates form the largest class of rock-forming minerals.

Silica is the most common mineral generally found as quartz, sand and flint. Silica is very hard and durable and is unaffected by weathering agencies and forms about 60 per cent of the constituent of common rocks, and alumina about 15 per cent. (sandstones, granite ; etc.)

Alumina is the chief constituent of clay.

Quartz is pure silica and is generally translucent with white opaque colour but other colours are also sometimes there. Most of the common sand is a form of quartz.

Mica is a source of weakness is a stone, and stones having more than 2 per cent of mica are unsuitable for structural purposes. Mica flakes are either white or dark brown or black and shine with metallic lustre.

Suitability of Type of Stones for Engineering Works

(*i*) Buildings in manufacturing town—compact sandstones and granites ; (*ii*) Buildings on sea-shore—granite and fine-grained sandstones ; (*iii*) Bridge piers, docks and breakwaters—granite and gneiss ; (*iv*) Road metal—granite and basalt ; (*v*) Railway ballast—sandstones, compact limestones and quartzite ; (*vi*) For fire resistance—compact sandstone.

Strength of Stones : (Pressure perpendicular to the plane of foliation, lamination or bedding)

Stone	Crushing load per sq. cm in tonnes			Tensile strength per sq. cm in tonnes		
	max:	min:	av:	max:	min:	av:
Basalt, Trap	1.76	0.55	1.34	—	—	0.10
Chalk	0.03	0.01	—	—	—	—
Granite, Gneiss	1.75	0.76	1.16	—	—	—
Lime-Stone	0.98	0.09	0.33	—	—	—
Marble	1.25	0.22	0.72	0.05	0.04	0.04
Oolite	0.27	0.09	0.19	—	—	0.05
Sandstone	0.84	0.17	0.55	0.09	0.02	0.06
Slate	1.41	0.72	1.16	—	—	0.09

Availability of Stones

Sand-stone : Agra, Bengal, Assam, Gwalior, Delhi, Eastern Ghats, Jubbulpore, Karachi, Mirzapore, Mysore, Pachmari, Punjab hills, Saugor, Trichinopoly.

Lime-stone : Bombay (Shahbad flags), Hyderabad, Jubbulpore, Madras, Sarai Kala, Trichinopoly, mountain border districts of Punjab.

Slate : Chamba, Dalhousie, Rajasthan. Slates of Southern India are of poor quality.

Marble : Gwalior, Jaipore, Kashmere and Southern India.

Granite : Ajmet, Bangalore, Dalhousie, Jhansi, Jubbulpore, Mysore, Madras, Secunderabad, Bihar, Orissa, Kutch.

Basalt or Trap : Bombay, Madhya Pradesh, Dehra Dun, Chakrata, Madras, Poona.

Laterite : Belgaum, Midnapore, Mahableswar, Orissa, Trichinopoly, West Coasts.

Kankar : All over India.

Stone Beams

Table of safe dead extraneous loads for beams of good building granite, 10 cm broad, supported at both ends and loaded in the centre :—

Depth in cm	Clear span in metres								
	0.3	0 6	0.9	1.2	1.5	1.8	2.1	2.4	3.0
	Safe load in kg								
2.5	18	9	—	—	—	—	—	—	—
5.0	75	36	23	18	—	—	—	—	—
7.5	162	81	54	36	31	—	—	—	—
10	288	144	90	72	54	45	36	—	—
12.5	450	223	144	108	90	72	63	72	54
15	648	324	216	162	126	108	90	108	81
18	882	441	288	216	171	144	117	135	108
20	1152	576	378	288	225	189	162	216	171
25	1800	900	594	450	351	288	252	315	252
30	2592	1296	864	648	504	432	360	—	—

The weight of beam allowed for in the above able is 2700 kg/cu. m. Factor of safety taken is 10.

If the load is uniformly distributed, the safe loads will very nearly be twice as great as those shown in the table.

For good slate on bed, the safe loads may be taken at about three times.

For good sandstone on bed, about half of the loads should be allowed.

For good marble or hard lime-stone on bed, the same loads as above should be allowed.

8. QUARRYING AND BLASTING

Quarrying of stone for small jobs is generally done by hand tools alone such as crowbars and wedges. In the case of large quarrying operations in hard rocks, rock drills are used.

There are natural joints and fissures in rocks and advantage is taken of these joints where existing in separating one block from the other. Fissures, cracks, planes of cleavage and bedding planes of stratification are all weak points in a rock. Where natural fissures or joints do not exist, artificial fissures can be made by drilling a line of holes (in rows), about 12 to 50 mm in diameter, 100 to 150 mm apart and about 150 to 200 mm deep with the aid of a chisel and hammer. In quarrying, holes are jumped or drilled along the desired line of cleavage and in each hole are placed two half-round pieces of steel with a conical wedge between them. (These are also called "feathers" and plugs"). If all the wedges are driven along together in succession with a hammer the rock will crack along the face of the holes. Instead of steel wedges, round plugs of dry hardwood are sometimes driven and kept soaked with water. The swelling of the wood will split the rock. Lighting and maintaining a fire on the surface of a rock causes the upper layer of the rock to expand and separate from the lower mass.

Blasting

Hard metamorphic rocks are difficult to be quarried and have to be blasted by means of explosives. Explosives should only loosen and break up the mass of the rock making it easy to be worked by tools and should not blow out the rocks violently, which may convert good workable stones into useless small pieces.

The explosives commonly used are—Blasting powder or Gun-powder, Dynamite, Guncotton or Blasting cotton, and Cordite. Ordinary blasting powder and cordite can be ignited by means of a fuse. The effects of any explosive can be greatly increased by percussion or detonation. Percussion caps or detonators contain fulminate of mercury which explodes on being ignited by an ordinary fuse or by an electric current. Guncotton and dynamite are fired by detonation.

Tools required for blasting : (*a*) Steel Jumper 1.8 to 3 m long. 40 mm diameter with chisel end of hard cast steel welded to it. (*b*) Tamping needle of brass or copper of slightly smalter diameter than that of the steel jumper, with a flat end. (*c*) Scraping spoon.

Materials Required for Blasting

Gunpowder or black powder : This is comparatively a weak explosive, slow in action, has great lifting power but no great shattering effect. Is easily ignited by means of safety fuse. Used for blasting soft rocks. Gun powder is available in powder form and explodes with a sound when in touch with a fire.

Blasting powder. Is a variety of gunpowder still slower in action and is available in crystalline form. It explodes and cracks the rock on all sides and the blocks have to be extracted by jumpers used in quarries.

Cordite. Cordite is supplied in the form of sticks or cartridges. It is comparatively slow burning explosive with effects similar to dynamite and three times stronger than those of blasting powder. It is ignited similar to blasting powder. It is effective under water subject to the charge being water-tight and is also more economical than dynamite or blasting powder.

Gelignite. This explosive is also very powerful and convenient to use and can work in wet conditions. One kg of gelignite with 12 detonators explodes about 6 cu. m of rock.

Detonators. They consist of small copper tubes with one end closed, containing an explosive priming substance, and are made to set off a high explosion under wet conditions. Detonators must be carefully handled and never left lying about ; if dropped they are liable to explode. Pressure should never be put on the fulminate end, and to bend it is extremely dangerous. Detonators should be stored separately from other explosives. Detonators explode when in touch with fire or struck with a piece of timber or stone or pressed hard in the hand. 12 detonators are required for about one kg of gelatine. They are fired either by fuse or by an electric spark and

are controlled from a fairly long distance. The sawdust in which the detonators are packed should be blown out with a dry blow of the mouth before using them.

Fuses. These are thin ropes of cotton impregnated with fine gun-powder, and burn at the rate of about 60 cm per minute for ordinary fuses and 30 m per second for instantaneous fuses. Usual length per charge whether used with gun-powder or with detonator, is 90 cm. In hot climates fuses deteriorate rapidly and are seldom reliable after the tin containing them has been open for 6 months. A batch of fuses should always be tested before use to ascertain the rate of burning length which must be accurately known, to enable a correct length being cut that will give sufficient time to the firer to reach a place of safety. Safety fuse burns under water. For ordinary quarry work a medium grade fuse is satisfactory. The charges are fired by lighting the fuse.

Capped Fuses. This is the name given to the length of safety fuse to which detonators are attached befor they are taken to the place of use.

Dynamite and Gelatine. Nitroglycerine is absorbed in porous solid, available in the form of cartridges, exploded by means of detonators. This has a high explosive value and is liable to shatter the rock to pieces. It is used for heavy work and can work under wet conditions or even in water. This explosive requires careful handling as it is detonated by a strong shock ; easily lighted but burns quietly in small quantities ; is very poisonous and causes violent headache through contact with the skin. Not suitable for very cold climates, and if frozen it is most dangerous.

Guncotton or Blasting-cotton. In dry form and under heat it is highly inflammable and easily detonated by concussion. Addition of water renders it non-inflammable and safe to handle and store without deterioration, and increases the explosive effect. When in a wet state it can be exploded by a primer of dry guncotton and a detonating fuse (or detonator plus fuse). Guncotton is a stable explosive in all climates. It is useful where cutting or shattering effect is required and is more powerful than dynamite and needs greater care in handling. Guncotton is supplied for use in the form of small discs used generally for exploding rocks under water.

Blasting with Dynamite. In blasting rocks with dynamite, the following data will be useful :—

Diameters of drills used for different depths of bore holes .

From 0.6 to 1.8 m depth—25 mm dia.
" 1.8 to 3.5 m depth—40 mm dia.
" 3.5 to 5.0 m depth—50 to 60 mm dia.

The depth of the bore hole should be about the same as length of the *line of least resistance* and if possible the bottom of the holes should never descend below the face of the rock. The bore holes should generally be not more than 1.5 m deep and the distance apart should be from 1.5 to 2 times their depth. The charge should always be placed in a sound piece of rocks and if possible not nearer to a crack or fissure than 30 cm.

One end (cut square) of a fuse is pushed into a detonator till it touches the white fulminate within it. The open end of the cap is then pinched in with pincers to attach it to the fuse, care being taken not to break the powder core of the fuse by pinching too tightly. A primer, *i.e.*, a dynamite cartridge used for priming, is then opened at one end and the detonator gently pushed into the dynamite leaving about 1/3rd of the copper tube exposed outside. The paper of the cartridge is then closed up and securely bound with wire or twine to prevent dislocation of the detonator. Avoid pushing the detonator too far into the cartridge otherwise there is a risk of the fuse burning up the cartridge releasing fumes. The premier, (*i.e.*, the cartridge with the detonator and attached fuse), is then gently inserted on the top of the charge. The space for about 20 cm above the charge is then gently filled with dry clay pressed home and the rest of the tamping is formed of any convenient material gently packed with a wooden hammer.

If it is desired to shatter rock, close connection between the dynamite and the rock is essential, and the points of contact should be multiplied as much as possible, therefore several bore holes of moderate diameters are preferable to one hole of a large diameter. In gently sloping rock with no face, dynamite should be used very much like powder is, only with fewer and shallower bore holes. As the line of least resistance is not so important in dynamite as in powder, the necessity for sloping the holes is not so great ; but if face is required on an almost level rock sloping holes must be used.

Blasting with Powder. Before filling the explosives it is quite essential that the holes should be thoroughly dry, and where water percolates it must be dried with oakum and quick lime, and the powder enclosed in a waterproof cartridge made either of thin plane iron sheet or water-proof paper of the necessary length so that the top of it is 15 cm above the top of the hole. (Explosives which are not affected by water should be used in wet situations).

When the hole has been bored proper proportion of the powder is poured into it by a funnel and copper tube so that none adheres to the sides. A fuse of sufficient length is inserted into the powder and taken outside to a sufficient distance according to the burning speed, as explained elsewhere. A wadding of hay, moss, or dry turf is placed on the powder and around the fuse and the remainder of the hole is filled in with sand and clay mixed, or soft moorum. 25 to 50 mm of the wadding is pressed down on the powder and the clay is tamped each time a little quantity is put to a small depth in the hole. Tamping in done with a copper, brass or wooden rod, until it becomes compact and no air hole is left around the fuse. If tamping is not done properly, explosion will take place along the line of the bore and the rock will not be blown out. Sometimes a priming needle (which is a thin copper rod of about 1.5 mm dia.) smeared with grease is inserted in the tamping materal which is subsequently-removed after the tamping is over, and the fuse is then introduced. Gunpowder can also be poured into the hole left by the needle and on the top of it the fuse is introduced. The fuse must be cut to the length required before being inserted into the hole. Joints in fuses should be avoided.

Quantity of blasting required approximately is 60 grams for 60 cm length of line of least resistance. One kg powder will loosen about

1.7 cu. m of hard rock and 2.8 cu. m of soft rock.

The proper charge of powder and the direction and spacing of the holes are very important in the case of blasting with powder.

Line of Least Resistance. Is taken as the shortest distance from the centre of the charge to the nearest surface of the rock. If there is any fissure or weak point in the rock its distance from the explosive, if shorter than the above distance, is taken as the line of least resistance. The line of least resistance must never be in the direction of the hole bored.

Boring Holes in Rocks : Bore holes are generally 25 to 75 mm in diameter and 30 to 120 cm deep for blasting with powder. The depth of bore-hole should be about the same as the line of least resistance and the bottom of the hole should never descend below the face of the rock. For blasting with dynamite the bore holes should be further apart than with powder, of similar depth but of smaller diameter.

Destruction of Blasting Explosives

Gunpowder : Should be thrown into water, preferably, hot water.

Nitrate of Ammonium : Should be scattered on damp soil.

Dynamite : Not more than 25 kg of dynamite should be destroyed at a time. A clear space of ground, about 100 metres all round should be selected, and a line of shavings or dry straw or grass laid down. On this the cartridges of dynamite should be placed in a continuous line not more than two abreast with the cartridge wrappers and any other available paper below them, and at an interval of an 25 cm between each two cartridges. Paraffin or other similar oil should then be poured over the shavings, straw or grass and cartridge, to accelerate the combustion. The line of shavings, straw or grass should be prolonged some distance beyond the dynamite (say 6 m) and lit with a short length of safety fuse and the operator should then retire quickly to a safe distance. The direction of the fire should be about an angle of 45 degrees to the direction of the wind and the fire should be ignited from the weather end.

Safety Fuse : Should be destroyed by burning in lengths in the open under suitable precautions.

Detonators : Should be disposed of by being taken to a deep river or sea ; or they may be soaked thoroughly in mineral oil for 48 hours and then destroyed one at a time, under suitable precautions, by burning.

Pracautions Against Mis-fires

Mis-fires are a source of great danger. In the case of doubts, allow sufficient time to elapse before entering the danger zone. Where fuse and blasting caps are used, a safe time of say, an hour should be given.

(*a*) The safety fuse (lighting end) should be cut in an oblique direction with a knife. (*b*) All saw-dust should be cleared from inside of the detonator by blowing down the detonator and taping the open end. (*c*) After inserting the fuse in the detonator it should be fixed by means of the nippers. (*d*) If the bore hole is damp, the junction of the fuse and detonator must be made water-tight by means of tough grease, white lead or tar. (*e*) The detonator should be inserted into the cartridge so that about one-third of the copper tube is left exposed outside the explosive. The safety fuse outside the detonator should he securely tied in position in the cartridge. Waterproof fuse only to be used in damp bore holes. (*f*) Only 10 holes may be loaded and fired at ope time, and the charges should be fired as far as practicable successively and not simultaneously. Bore holes must be thoroughly cleaned before a cartridge is inserted.

The withdrawal of a charge which has not exploded is under no circumstances to be permitted but the tamping and charge should be flooded with water, aud the hole marked in a distinguishing manner. Another hole should be jumped at a distance of more than 45 cm from the previous hole and fired in the usual way.

Precautions for Storage of Explosives

(*a*) All explosives should be protected from extreme heat or cold and moisture. (*b*) Packages containing explosives should not be thrown, dropped, rolled or pulled along the ground, but should be passea from hand to hand and carefully deposited. (*c*) Detonators, fuses and percussion caps should be kept in separate containers and should always be stored away from other explosives preferably in a separate building and never in the same container with detonators.

(*d*) No iron or steel tools should be used where liable to come in contact with explosives, but only wooden levers, wedges and mallets should be used. (*e*) Explosives should be stored in a pucca building separated from any dwelling house, public thoroughfare or any other building, by a distance of at least 45 m. (*f*) No person entering a room or building where explosives are stored, should have in his possession any matches, fuses or other appliances which can produce ignition or explosion. No person should be allowed to smoke or ignite any fire or light in the proximity of the building where explosives are kept. Nor any person wearing shoes with iron nails should be allowed inside the building. Persons with bare feet will, before entering the magazine, dip their feet in the water kept for the purpose in a tub and then step direct from the tub on to the clean floor. (*g*) A magazine on no account is to be opened during or on the approach of a thunderstorm, and no person should remain in the vicinity of the magazine during such storm. (*h*) Under no circumstances should a magazine be erected within half kilometre of any working kiln or furnace. (*i*) Two thoroughly efficient lightning conductors should be provided to a magazine, one at each end. (*j*) Should there be difficulty in keeping the magazine free from damp, fresh burnt quick lime, exposed in wooden trays, is recommended.

Blasting Operations :

(*a*) Red danger flags should be prominently displayed and all the work people, except those who have actually to light fuses, must stand away at a safe distance of not less than 150 m, before an explosive charge is fired. (*b*) For making the hole in an explosive catridge to take the detonator, only hardwood should be used and only wooden tempers for tamping explosive charges. On no account any metal implement should be used. (*c*) After firing an explosive charge sufficient time must be allowed to elapse before men are allowed to return to work within the danger zone

otherwise asphyxiation from carbon monoxide fumes may occur. (*d*) Some explosives are very dangerous in frostly weather. If work cannot be suspended during such a weather, special precautions must be taken to keep the explosive cartridge at a safe temperature. (*e*) When cartridges sweat they should not be handled with bare hands.

9. PAINTS & PAINTING

Paints essentially consist of the following :

Base of solid matter which is the principal constituent forming the body of the paint. White lead, Red lead, Zinc oxide (or zinc white), Iron oxide, and Graphite, are used.

White Lead is the base most largely used for all ordinary building works and is the cheapest. White lead ground in linseed oil and made into a stiff paste is the usual product on the market. White lead powder is also available. It is easily applied, works well, has a greater covering power than any other base, is dense and has a good body to obscure the surface and weathers well. White lead is poisonous (and care should be taken that it is not inhaled during mixing), is discoloured on exposure to the air, therefore, should be kept covered, and is not suitable for delicate work but is often used as undercoat with finishing coat of white zinc. It should be kept a considerable time before using as if used too fresh it acquires a yellowish tinge. White lead is not suitable for iron work as it dots not stop rusting.

Red lead is considered best as a primer (first coat) with oil for iron as well as woodwork as it sticks well and gives good protection against rust. It is a strong drier of linseed oil solidifying it in a short time. Red led is available in the form of powder, and is very heavy in weight.

Lead paints are poisonous and should not be used fresh. Precautions should be taken while scraping old dry painted surfaces or while painting with spray machines.

White lead is frequently adulterated with sulphate of baryta, whiting, etc., and red lead with brick-dust. The presence of such impurities can be detected by the addition of dilute nitric acid which dissolves lead paints.

Zinc Oxide or Zinc White is unaffected by weathering, is non-poisonous, but is costly ; less workable and less durable than white lead. It takes a fine polish, and is most commonly used for interior decorations. Whitest quality should be obtained and which should be completely soluble in dilute sulphuric acid without any effervescence. (See under "Zinc Paints".)

Lead driers should not be used with zinc paints.

White lead bases dry soft ; zinc white dries hard, and it is contended that a certain blend of the two will give better practical results for most purposes than either used separately.

Lithophone is a white paint (zinc pigment) used for interior work. It is cheap, non-poisonous but becomes yellow when exposed to day-light and is not very satisfactory.

Oxide of Iron is used as a base in paints chiefly for finishing coat on iron work.| It also prevents the formation of rust, and is comparatively cheap material. The tints obtainable vary from yellowish-brown to black.

Vehicle. The liquid vehicle helps to spread the base and colour pigment over the surface to be painted ; acts as a binder for the base and pigment and forms a tough elastic film when dried.

Linseed oil is the most widely used vehicle for all ordinary painting works. It is either raw or boiled.

Raw Linseed Oil is thin, pale in colour and transparent, sweet to the taste and has no smell. When exposed to the air it becomes hard and stiff. When spread in a thin film looks like varnish. For woodwork where the original colour and grain are to be preserved, or for delicate and interior work, raw oil is used by mixing it with a drier. It dries very slowly and is used for works not exposed to weather. The drying of raw oil may be improved by adding about 1 kg of white lead to 10 litres of oil and allowing it to settle for at least a weak. Addition of about one-third to one-fourth of boiled oil also assists drying for exterior work.

Boiled Linseed Oil: Boiling makes the oil thicker and darker in colour. During boiling a drier such as, red lead or litharge, is generally added. A boiled oil is more viscid than the raw oil, varying in colour from a deep amber to rich brown (having a reddish tinge). Dries up quicker than raw oil with a hard glossy surface, has more covering capacity, and is more durable. It is used for external work.

Pale boiled linseed oil is better than raw oil and is the same as ordinary boiled except that it is not dark in colour.

Double boiled linseed oil is as clear as raw linseed oil but smells slightly different, and dries quicker and gives better results. Pale boiled or double boiled oil is more suitable for painting plastered or metal surfaces. It generally requires a thinning agent like turpentine.

Linssed oil is readily soluble in turpentine, naphtha, mineral spirit (petroleum) and alcohol when in liquid form, but the dry film (painted surface) withstands this action and is also fairly water-proof.

Bad oil appears opaque, turbid and thick ; tastes acid and bitter to the tongue and its colour is rancid and strong. Good oil is mellow and sweet to the taste and has very little smell. Good boiled linseed oil spread in a thin film on glass or metal, should become quite hard (in dry weather) from in 12 to 30 hours, and it should be so dry in 24 hours that dust will not adhere, whereas raw oil may take from 2 to 10 days depending on the state of the atmosphere.

Inferior oils will frequently never really harden and will become sticky in damp weather. A sample kept in a bottle for 15 days should deposit no sediment whatever.

Linseed oil is subject to adulteration by the addition of cotton-seed, resin, mineral and fish oil. Adulteration can be detected by the smell by rubbing a few drops of the oil between the hands. As substitute, fish oil, cotton seed oil, are used.

When country linseed oil is used it should be boiled for about

3 hours with red lead and litharge in the proportion of 1 kg of each to 10 litres of oil.

Tung oil is much superior to linseed oil and is used in making superior paints and varnishes. *Poppy oil* is used for very delicate colours and is inferior to linseed oil as regards its drying qualities though its colour stands longer. *Nut oil* is nearly colourless, dries very rapidly, is not so durable, but is cheap.

Solvent or Thinner. A liquid thinner is used to thin prepared paints to the desired consistency and make them work more smoothly and evenly, and also helps penetration of porous surfaces. Spirits of *Turpentine* is the most common thinner used. Turpentine is inflammable, evaporates rapidly and dries the oil consequently. Use of a thinner in paint reduces the protective value of the coating, flattens colours and lessens the gloss of the linseed oil as the spirits evaporate leaving an excess of colour not mixed with the oil. When a dull "flat" appearance for indoor work, is desired only turpentine and no oil is used for the last finishing. At the most 5 to 8 per cent of the thinner might be added. A thinner is not generally used in finishing coats on exposed surfaces as it has a tendency to impair the firmness of the paint, but if the surface is to be exposed to the sun, turpentine is added to reduce the possibility of the paint blistering.

Turpentine has a pungent odour, is often adulterated with mineral oils and some of them have higher penetrating values but are otherwise inferior. Benzine and Naphtha are used as substitutes. Turpentine is a transparent, volatile liquid, obtained by distilling the resinous exudation of some varieties of pine trees.

Simple test for purity of turpentine: (*i*) When warmed gently, it should not smell of resin or coaltar; (*ii*) When shaken vigorously it should not froth ; (*iii*) On evaporation it should leave no residue. Paper coated with turpentine and left to dry should remain unstained, and should then take ink freely.

Driers. The function of a drier is to quicken the drying of the vehicle (linseed oil) in the paint and in consequence set a hard film. Driers are usually compounds of metals ; litharge (or oxide of lead), zinc sulphate, red lead, dissolved in a volatile liquid. *Litharge* is the most common drier in use (the proportion being 1/8 kg to 5 litres of oil) and *red lead* which is less powerful in its action than litharge, is next to it. Litharge is used especially for lead paints, but is not used for a finishing coat. *Zinc sulphate** is more costly. Driers should not be used unnecessarily, nor in excess, especially in the finishing coat as they have a tendency to destroy the elasticity of the paint and cause flaking of the paint. A drier should not be added until the paint is about to be used, nor more than one drier should be used in a mixture. Driers need not be used with pigments that dry well.

Pigments. Pigments form the colouring matter in a paint and are available in the form of fine powders in various colours and qualities. They

*Zinc sulphate is used as drier for zinc paints. No drier containing lead should be used for a paint with a zinc base as voltaic action will be set up.

are either powdered natural earths or calcined colours or metals, which must be thoroughly mixed with other constituents. Generally pigments of earthy or animal origin are less permanent than mineral colours.

Preparation of Paints

A good paint should have a high covering capacity and be fluid enough to be spread evenly in a thin coat and dry quickly forming a tough durable film without showing any brush marks or cracks.

To prepare a paint the base (white lead) is thoroughly ground in oil and mixed with the thinning agent (turpentine) to impart the necessary workability to the paint. The pigment (tinting colour) and drier (where desired) are also separately ground and are first mixed with linseed oil and then diluted with turpentine to a thin consistency, and mixed with the base that has already been prepared. The paint is then strained through fine canvas or a fine sieve. The paint should be used as soon after remixing as possible ; red lead is likely to harden after 24 hours. Paints are thinned by adding pure boiled linseed oil only. (Lead driers should not be used with zinc paints). See also "Estimating" Section.

For surfaces which are subsequently to be varnished, a minimum quantity of oil should be used in the paint.

When thinning of paint is required to produce a requisite consistency it can be done with the following mixture : Boiled linseed oil 14 parts, Turpentine 1 part.

For white paint only raw linseed oil should be used as boiled oil turns it yellow.

If paint has to be laid aside for a time in an open vessel, it should be covered with water to prevent oxidation and drying.

VARNISHES

The essential constituent of all varnishes is "resin" or rosin which is dissolved in oils, turpentine, or alcohol. The liquid dries or evaporates and leaves a hard transparent, glossy film on the varnished surface. There are various types of varnishes obtainable in the market each suited to a specific work. The preparation of varnishes is a difficult matter, and it is best to purchase ready made. Varnish dries quickly and gives a hard and tough coating. Painted surfaces are also varnished to brighten them.

Water varnishes are used for painting paper surfaces.

Oil varnishes are for interior or exterior works. Superfine Copal varnish is considered to be the best as it produces a higher gloss and smoother finish. Copal varnish is made from the fossil resins (the copals) which are found in several parts of the world and in many different grades of quality. English copal is considered to be the best. If the varnish is too thick, spirits of turpentine can be added.

Spirit varnishes : Shellac varnish and French Polish belong to this class.

Resins used for preparation of varnishes are generally obtained from gums of various trees. The most common being Shellac, Gum Arabic, Rosin, Amber.

Aluminium Paint. Aluminium paint has the advantage of being visible in the dark. It does not oxidize and fade. Aluminium and graphite paints have great covering capacity : 5 litres of paint covering 1000 to 1500 sq. m of surface. It protects iron and steel from corrosion due to sea water and acid fumes far better than any other paint and also resists heat to some extent. This paint is commonly used for painting electric and telegraph poles, oil storage tanks, hot water pipes, marine piers, etc.

Zinc Paints. Zinc pigments such as zinc oxide, zinc chrome, lithophone (described before), and zinc dust are now being used on an increasing scale for white paints for indoor and outdoor use, especially on metal works. Zinc oxides have great resistance to weathering and high hiding power. Zinc sulphide has an interesting application in the production of luminescent and fluorescent paints which are used for the illumination of maps and aircraft instruments in darkness.

Cellulose Paints. Are costly but much superior to ordinary paints and for special purposes such as motor cars, airplanes, and are commonly known as "spray paints." They possess greater hardness, smoothness and flexibility and can be washed and cleaned easily and stand heat and weather well.

Shellac or "lac" is made from the exudation of a kind of insects which grow on some varieties of trees. It is soluble in alcohol and an alkaline water solution. It is not soluble in turpentine and will withstand the action of acids. This immunity make it an ideal material for placing between the knots and sappy portions of timber ; these contain a crude turpentine which is a solvent for linseed oil and oil paints. Used for making varnish ; gives glossy finish.

Glue is made from bones. A good quality glue is clear in colour, transparent dark amber, free from spots or cloudy patches, without much smell. When immersed in cold water, good glue becomes soft and swells considerably but does not dissolve in it unless of inferior quality. For preparation of the glue, it is soaked in water and boiled in a double glue-pot specially made for the purpose. Glue is used in joining wood joints. Glue should not be used in exposed works as it absorbs moisture in damp weather and sets up decay, instead a mixture of 4 parts white lead, 1 part litherge and 8 parts boiled linseed oil is used. Size is made from superior glues. One kg of glue makes 10 litres size ; double size is of double consistency and one kg makes 5 litres. Size is used in white-washing, distempers, etc.

Clear Cole. Is a size coating applied to fill up the pores of wood or plaster preparatory to distempering or painting.

Preparing Woodwork for Painting

Whole success of the painting operation depends upon satisfactory preparation of the surface to be painted and the great majority of defects which occur are due to a faulty preparation. It is essential that the wood should be well seasoned and the surface to be painted perfectly dry.

The surface of woodwork to be painted or polished should be rubbed down perfectly smooth with medium and fine grade sandpaper, all rubbing to be done with the grain. Worked timber should be primed as soon as possible particularly on the cut end grain.

New woodwork should be knotted, primed and stopped before giving coats of paint.

Knotting. This process is done before the application of a priming coat to cover all knots in wood to prevent any exudation of resin, or any marks to show through the paint caused by the absorption of the knots. There are three common methods of knotting : (*i*) Lime knotting, (*ii*) Ordinary size knotting, and (*iii*) Patent knotting. Knots in deodar or other resinous woods must be painted over with hot lime and scrapped off after 24 hours, the knot primed with red lead and glue laid hot and one coat of knotting varnish applied ; the surface rubbed smooth with pumice stone or sandpaper.

Ordinary size knotting is applied in two coats. The first is made by grinding red lead in water and mixing it with strong glue size used hot. (Dries in about ten minutes). The second coat consists of red lead ground in oil and thinned with boiled oil and turpentine. *Patent knotting* consists of two coats of a varnish made by dissolving shellac in methylated spirit or naphtha. Knotting may be composed of 150 grams of pure shellac dissolved in 1/2 litre of methylated spirit and when thoroughly dissolved, 15 grams of red lead is stirred in. This is suitable for general purposes.

Stopping is filling up nail holes, cracks and other inequalities to bring the surface to a level. Stopping can be done with ordinary putty made of 2 parts of whiting (absolutely dead stone lime), 1 part of white lead, mixed together in linseed oil and kneaded, (100 grams linseed oil to 1/2 kg of whiting will also do), after the priming coat of paint has been applied and not before, as otherwise, the wood absorbs the oil in the stopping and so defeats the purpose. For high class interior work, the stopping can be of a mixture of 1/3 of white lead to 2/3 or ordinary putty. For varnishing, the wood surface can be stopped with hot weak glue size (one kg of glue making about 10 litres of size). When dry, the surface should be well sand-papered.

"Beaumontage" or stopping-out wax is a useful preparation for concealing all defects in floor and wood-work generally. It is made as fallows : Put a cupful of common shellac in an iron pot, add a tea-spoonful of resin, a piece of bees-wax (about 25 mm and a teaspooful of powdered lemon chrome or other colouring powdered matter. Heat until the whole is melted and mix all well. It should be applied hot. This wax will not take stains so it must be coloured to suit the finished work. It sets quite hard.

Priming Coat is the first coat applied to fill the pores of wood or any minute inequalities on the surface to be painted. It also prepares a smooth base for the subsequent coats of paints and accelerates their drying. A priming coat may be given of red lead, or of red and white lead mixed in double boiled linseed oil (3 kg of red lead, or 3 kg. of red and white lead, mixed with (3 litres of oil). When dry, all cracks or holes are filled up with putty and the whole surface rubbed down with pumice stone or sandpaper, and allowed to harden before applying paint. (Priming coat should have no turpentine.)

Woods with excess of resin or oils in them are unsuitable for polished and painted work *e.g.*, the resin of Deodar shows itself up in discoloured patches even through a number of coats of paint.

Second coat of the desired colour is laid on in exactly the same manner as the priming coat and when dry the surface is rubbed down with pumice stone or glass paper. This is followed by third coat. One kg of pumice is required for rubbing down about 200 sq. m of old surface.

Each coat is allowed to dry completely before the next is applied. The final coat should be carefully crossed. Paints should be applied in thin coats, thick coats take longer to dry and generally begin to flake off after sometime.

If lead paint has been used, the dry rubbing of lead painted surfaces must invariably be prohibited. Dry sandpapering of painted surface is the cause of lead poisoning among painters. Water-proof sand-papers or flint paper and cloth for rubbing are available.

When the work to be painted is subjected to a strong light and is not of very high finish, oil paintings show up every defect ; in such cases it is desirable to have the painting done in turpentine instead of oil the result being a flat instead of shiny surface. The proportions used are 1 kg of white zinc, 1/3 litre of turpentine and 1/6 litre of boiled linseed oil.

When white paint is specified, white lead should be used if the work is outside and likely to be exposed to the weather. White zinc should be used for inside works not exposed to the weather.

Blistering : Sometimes paints blister if the coat is too thick or if there is moisture in the paint of the wood painted.

Repainting Woodwork

If the old paint work is in a perished or cracked condition, no satisfactory job can be made other than by complete removal of the old film.

Old work can also be cleaned with lime-wash and rubbed with pumice, filling all holes with putty. Washing with soap and water is also effective.

Sodium Carbonate or *Washing Soda* : Diluted with water is the cleansing agent used in the preparation of old painted surfaces for repainting. Cleanses greases and fats.

Paint Removers. Ready made paint removers are available.

2 parts quick-lime, 1 part washing soda made to the consistency of cream is painted over the wood.

Thick layers of old paints are generally burnt with a blow lamp and scrapped. Flame of blow lamp cracks window glasses for which precautions must be taken.

French Polish is a spirit varnish and is made by dissolving 1/4 kg of shellac in 1/2 litre of methylated spirit or naphtha and straining the solution through a double thickness of coarse muslin. A number of other recipes are also in use. It should be applied to the prepared wood surface with a polishing pad of soft cloth containing absorbent cotton filling, and not with a brush, with quick and light strokes along the grain. Several coats will be necessary before the desired shine and finish is achieved. The pad may be dabbed with a drop of olive or mustard oil after each coat to allow a smooth working and finish. The wood to be polished should be first painted with a filler composed of 3 kg of whiting mixed in 3 litres of methylated spirit and then sandpapered when dried.

Fillers can also be made as follows : (*i*) Whiting mixed with water ; (*ii*) Linseed oil and bees wax (3 : 1) boiled ; (*iii*) Plaster of Pasis either in water or raw linseed oil.

French polish is worked upon the surface of hardwoods to heighten the effect of the grain.

Frequent applications of raw linseed oil rubbed in well with rags will give a very fine polish to the woodwork.

A good furniture polish can be made of equal parts of vinegar and linseed oil, or better still of vinegar and olive oil in the same proportions, as this mixture is less sticky than the former.

Wax Polishing. Wax polish is made by mixing 2 parts of bees wax with 2 parts of boiled linseed oil over a slow fire, when dissolved but still warm, add one part of turpentine. Smear the woodwork with the mixture and after 24 hours rub with a soft flannel to a fine polish. Wax polishing is mostly used for polishing cement concrete floors.

Whitening. Whiting mixed with size and water is used for whitening ceillings and walls. *Whiting* is made by reducing pure white chalk to a fine powder.

Varnishing. The woodwork when prepared should be sized with a coat of thin clear glue to which a little brown earth and ochre should be added if the wood is of oily nature and the varnish does not readily dry from this cause. This should be applied hot and rubbed down smooth. A second coat of thin clean glue with necessary quantity of staining colour consisting of equal parts of burnt umber and burnt sienia should then be applied, allowed to dry and rubbed down smooth with fine sand-paper. Two coats of boiled linseed oil can be given instead of glue size. Varnish should be laid on in thin coats over this when dried. English Copal varnish is considered best. For new woodwork a second coat of varnish should be applied after the first coat has thoroughly dried and rubbed down with fine sand-paper before the first coat and after each coat of varnish except the last. One kg of glue makes about 10 litres of size.

The varnish should become surface dry in not more than 6 to 8 hours and hard dry in not more than 18 hours. 1/2 litre of varnish will cover about 15 sq. m of surface, single coat Good varnish should be dry and free from stickyness within 2 days. Varnishing and painting should be avoided on stormy and rainy days. Varnishing is generally prescribed for interior works and painting for exposed prescrlbed for interior works and painting for exposed positions.

Oiling Woodwork. One kg of beas wax mixed witn 3 kg of double boiled linseed oil are heated over a slow fire till the wax is melted. After the mixture has coolcd, 3/4 litre of turpentine is added. This will cover about 150 sq. m of surface. The woodwork can also be oiled with country sweet oil to which equal parts of vinegar and turpentine have been added. This gives a darker effect. A mixture of oil and water should never be used.

Only well-seasoned wood should be painted or oiled. Painting damp or unseasoned wood will do more harm than good and will only induce dry rot and also result in the paint blistering.

Fire-Proof Paints for Woodworks (See under "Timber Structures")

Painting Iron Work

Preparing Iron-work for Painting. Corrosion in generally more rapid and severe in hidden places and pockets where water or rubbish collects. It is however, most severe in surfaces of steel or iron in contact with wood ; water is bound to collect between the wood and iron. Before painting, rust scales and dirt should be removed by means of iron brushes, scrapers or other effective methods. Bristle or wood fibre brushes can be used for removing the loose dust. Special attention should be glven to the cleaning of corners and re-entrant angles. Oil and grease can be removed by gasoline (petrol) or benzine excess of which shall be wiped off from the surface.

Flame cleaning is another method of preparing the surface. A flat oxy-acetylene flame is passed over the metal burning off the old paint and loosening the rust and scale and wire brushing. A solution of country soda and fresh slaked lime in equal parts will remove old paint from iron-work. 40 kg is enough for about 75 sq. m of surface area.

Paints containing red lead and litharge have been in use for a very long time and have given excellent results. There is nothing to compare with red lead for a priming or under-coat on structural steel where there is no abrasion, and is said to be very durable when pure. Red lead primer followed by a finishing coat of red oxide paint or paints with aluminium or graphite bases have been found very satisfactory. (Red oxide is grouped up with boiled linseed oil.) Red lead guards against rust while white lead and red oxide of iron do not stop rust. White lead applied directly to iron requires incessant renewal and probably exerts a corrosive effect. It may, however, be applied over the more durable paints when appearance requires it.

The first coat can be a mixture of pure linseed oil and dry red lead in the proportions of : 5 litres of oil to 15 kg of red lead. It should be applied immediately after cleaning the surface of the metal and when the metal is perfectly dry. If the coat is rained upon within 24 hours of application, it must be removed and another coat applied. The second coat will be applied when the first coat is thoroughly dry and set, which will be in about 4 days, and may consist of : 3 kg of red oxide paint ; 1/2 kg of lamp black ; 5 litres of boiled linseed oil.

The third coat can be of : 3 kg of red oxide paint ; 5 litres boiled linseed oil.

For unimportant iron works, or for roofs, red oxide paint can be made as follows :

Red oxide powder dry, 10 parts by weight ; linseed oil raw, 4 parts ; linseed oil boiled 1 part ; turpentine 1 part. 5 litres of this paint will cover about 40 sq. m of surface, two coats. For further details see Section on

"Estimating." 50 kg of red oxide powder and 36 litres of thinning will cover about 400 sq. m of surface.

Guarding against Rusting of Steel Works: For unprotected steel under conditions of complete immersion, rusting will result in an average reduction in thickness of 0.075 mm per year of face exposed to sea water, and 0.050 mm in per year in fresh water.

When the size of the exposed iron admits of it, its freedom from rust may be very much promoted by first heating it thoroughly and then dipping it into, or brushing it well with, hot linseed oil.

All structured steelwork should be primed and preferably given a coat of red oxide paint before erection except the surfaces to be riveted in contact and the surfaces which will be in contact with concrete.

Iron and steel work can be protected from rust as a temporary measure by means of a coat of white-wash or by covering it with slaked lime. Iron exposed to the weather can also be protected (temporary measure) by a coat of paint made with pulverized oxides of iron, linseed oil, and a drier. A coat of cement wash is also applied with advantage. Painting with simple coal tar does not prevent rusting of iron.

Where the sea atmosphere is likely to have a corrosive effect on the steel work, the steel work after being thoroughly scrapped and cleaned should be given a coat of raw linseed oil before the first coat of red lead paint is applied, and immediately after the steel work has been cleaned.

Painting Galvanized Iron. Galvanized iron should not be painted until it has been exposed to the weather for a year as paint adheres badly to new galvanized iron. If necessary to paint sooner, a coat composed of about 200 grams of copper acetate added to 5 litres of water, or 60 grams of muriatic acid added to a mixture of 60 grams each of copper chloride, copper nitrate and sal-ammoniac, dissolved in 5 litres of soft water, to which a small quantity of hydrochloric acid has been added, should be given. This is sometimes called Mordant solution. This mixture turns the galvanized iron black; the treated surface should be left for at least 12 hours before being painted. This will be sufficient for about 250 sq. m of the surface. Over this a priming coat of red lead mixed with linseed oil and turpentine in equal proportions is applied. Ready-made paints are also available for this purpose. It is considered that paint will adhere to galvanized iron if the surface is washed with vinegar or slaked lime and washing soda before painting. Zinc white does not adhere to galvanized iron.

Protection of Iron Work under Water. Iron or lead oxide paint is sometimes satisfactory, but probably refined coal-tar dissolved in a vehicle neutralized by the addition of slaked lime makes the most durable paint coating known for iron under water. Asphalt paints made by dissolving asphaltum in some suitable vehicle such as naphtha or benzine is also used for this purpose. Coal-tar has been especially effective as a protector for cast iron water pipes.

Quantity required—The average covering capacity is about 230 sq. m per 50 kg. Two coats are sufficient. Subsequent coats to be given only after the previous coats have dried.

Anti-corrosive black enamel paint is available for ironworks in water.

Paint for Steel Water Tanks: (inside)

The inside of all steel tanks can be painted with two coats of bitumastic solution. Not recommended for drinking water.

Painting Plaster: (See also under Distemper).

The free alkali in new lime and cement plaster rapidly destroys the oil in paint and prevents it from drying. For this reason it may not be possible to paint a plastered wall until 12 months after its completion and in such cases the wall should be white-washed in the first instance.

The walls should be primed with boiled linseed oil or glue size (glue mixed with water); glue size should not be used if the walls have been white-washed. First two coats should consist of white lead and boiled linseed oil. Third coat can be of white lead tinted to approach the desired colour and mixed with raw linseed oil and a small proportion of turpentine. The finishing coat should contain a large proportion of turpentine with a little varnish to serve as a binder and applied when the previous coat is still tacky. This will give a flat finish as a glossy finishing coat shows up the irregularities in the plaster.

In the case of new cement plaster walls, a solution of 2 kg of zinc sulphate in 4 litres of water should be applied to the surface and when dry given a coat of pure raw linseed oil; or the surface can be treated with dilute sulphuric or hydrochloric acid (I part acid to 50 parts water) and then washed down with water. (Acids should be added to the water and not water to the acids).

Two coats of paint thinned with turpentine and having a little varnish as a binder should serve as first and second coat. Third coat paint should be thinned with a mixture of three parts boiled oil to one of turpentine. The finishing coat can be the same as for lime plaster walls.

Paints are now available in the market which can be applied directly on newly plastered walls.

Painting Damp Walls:

Paraffin, 11 litres
Benzoline, 9 „
Pale resin, 6 kg.

Shake them in a vessel; when completely dissolved add 11 kg whiting and grind the whole mixture well. Keep the mixture airtight to prevent drying. Apply on damp walls as ordinary paint one or two coats according to dampness. It will dry hard. Paint can be applied on it.

Painting Cement Surfaces. The surface should be first treated with a wash of dilute white vitriol (zinc sulphate or washing soda are also effective and then primed as for plaster work; or alternatively, use the proprietory cement paints.

Cement Paints. Cement paints are available which are water paints and can be applied to all cement or concrete surfaces and brickwork. These pairts resist the penetration of moisture and have particular advantage for use over exterior walls of buildings, or on floors. They are applied with distemper brushes. Cement paints are of two types, for general use and for use on water retaining structures. Either of these types may be admixed with silica sand when used on open texture walls. Cement paints are supplied as powder to be stirred into water just before use.

The surface to be painted should be cleaned of all dust, dirt, oil, grease or efflorescence and wetted before the application of the paint. Soap should not be used for cleaning. An interval of about 3 to 4 weeks or more should be allowed between the curring of the concrete and painting. Generally two coats are sufficient for most purposes and an interval of not less than 24 hours should elapse between the two coats. About 5 litres of mixed paint is considered sufficient for 10 sq. m of smooth surface and 4 sq. m of very rough surface. Excessive thick coats are not recommended. As soon as after the paint has sufficiently hardened, the surface should be kept wet for about 3 days through a light spray of water applied several times a day.

Lamp Blacking : (For dark rooms and racquet courts) :

5 kg lamp black 3 kg dry white lead 6 litres boiled linseed oil 0.6 litre turpentine.	Will cover about 80 sq. m of surface.

Coal Tarring

Add one kg of unslaked lime to five litres of tar and heat it till it begins to boil. Take off the fire and add slowly 1 part kerosene oil to 4 parts tar, or 1/4 litre country spirit 4 litres of tar. Addition of the kerosene oil is often omitted ; lime is added to neutralize the free acid and to prevent the tar from running out in hot weather. Tar is also mixed with turpentine and linseed oil. The tar should be applied as hot as possible, the articles to be tarred should be dipped into the tar. Not less than 5 kg of coal-tar should be used per 10 sq. m of surface tarred. If possible the iron should be heated to a red heat and then tar brushed over. Where *Solignum* or *Creosote* is to be applied, these should also be applied very hot.

Creosote is a product obtained by distilling tar and is largely used as an effective preservative for wood.

Solignum is an excellent preservative for wood where timber is subjected to dry rot. It is made in several colours but brown is most generally used.

The wood to be painted must be clean and absolutely dry. Where two coats are specified each coat must be thoroughly dried before the next one is applied.

A once tarred surface cannot be painted well. After the tarred surface has been scrapped, two coats of good shellac knotting varnish should be applied before pointing.

Painting Brushes

The brushes should be of bristles and not horse hair. Bristles can be distinguished by the fact that each bristle is split at ends. A good brush should have springiness in the bristles.

The following sizes of brushes are generally used :

(*i*) For dusting large flat surfaces, sizes 12 or 14.

(*ii*) For girder work, size 8.

(*iii*) For woodwork, size 6.

(*iv*) For fine work, sizes 2 and 4.

A round brush is considered the best for painting.

New brushes should be placed in water for 2 to 3 hours, and then allowed to dry for one hour before use. When a brush is to used for another colour or is no longer required, it should be cleaned at once by dipping into kerosene (convering the bristles only) when not in use.

10. GLASS AND GLAZING

Types of glasses :

Crown glass. It is the cheapest quality of glass, used for window panes of small sizes, bottles, electric bulbs, etc.

Sheet glass. For all general engineering purposes the glass used is sheet glass.

Sheet glass is manufactured in thickness of :

2, 2.5, 3.0, 4.0, 5.0, 5.5 and 6.5 mm.

Glass window panes shall be not less than 2 mm thick for panes up to 30×30 cm size and 3 mm thick for panes bigger than 30×30 cm.

For sizes above 90 cm use plate glass 6 mm thick or above.

There should be a space of 1.5 mm all round the panes between the edges of the glass and the rebate ; a cushion of rubber, felt, or canvas may be given in between to absorb shocks.

Plate glass. It is stronger than sheet glass and also more transparent. Is manufactured in sheets varying from 3/16 in. to 1 in. (usual thicknesses are 3 mm to 32 mm). Plate glass is superior to sheet glass and is used for large-size glass panes for shop fronts, windscreens of motor cars, and looking glasses, etc.

Wired glass. Wire-netting is embedded in plate glass during rolling. It resists fire better than plate glass, and in case of fracture, the glass does not fall to pieces.

Safety glass or Shatter-proof glass. This is either (*i*) toughened variety of glass, or (*ii*) reinforced with wire-mesh, or (*iii*) a combination of two glass sheets between which a layer of transparent celluloid or any other transparent plastic is sandwitched.

Glass-crete are small square pieces of glass which are set in steel frames or concrete, for light in the basements. Semi-prisms are made on the underside of the glass pieces which collect light and project it into the basement.

Flint glass or lead glass is used for art glass, cut glassware, radio valves, lenses, etc. This glass is clearer than other types of glass and takes a finer polish.

Pyrex glass is a proprietory brand of heat resisting glass which is used for making cooking utensils, electric insulators and laboratory appratus, etc.

Anti-actinic or Heat Excluding glass. These glasses when used in windows resist heat passing through without affecting the normal passage of light. Are commonly used for railway coaches.

Rendering Panes of glass Opaque : White lead 1/2 kg, linseed oil 120 grams, varnish 30 grams, mix the whole till becomes plastic. The mixture is applied by tying it up in a piece of linen into balls about 25 mm dia. and pressing these balls against the glass with force (*i.e.*, tapping). They should not be rubbed over the glass as, if this is done, streaks make their appearance. Ordinary zinc white paint also produces good results when applied with a ball of silk cloth.

Glazier's putty. Glass panes are secured in place by means of putty or wooden moulds. Rebates should be painted or oiled one coat before glazing. Glazier's prigs or brads are fixed 75 to 150 mm apart. Rebates should be at least 12 mm.

Putty is made by mixing one part of white lead with three parts of finely powdered chalk and then adding boiled linseed oil to the mixture, to form a stiff paste, which is well kneaded and left for 12 hours covered with a wet cloth and finally worked up in small pieces. A little of varnish is sometimes added to the paste. Quantity of putty is 185 grams per metre of glass perimetre. For glazing in metal lashes, 5 per cent red lead should be added. Litharge is sometimes added as a drier. (Whiting is absolutely dead stone lime or finally powdered chalk.) In metal sashes, 4 glazing clips may be provided per glass pane for a size 30 cm × 60 cm, where glass panes exceed 80 cm × 200 cm, 6 glazing clips are used. Holes for glazing clips have to be dritted prior to fabrication.

Glass panes can be cleaned by : (*i*) Methylated spirit ; (*ii*) Painting the glass panes with lime wash and leaving it to dry and then washing with clean water ; (*iii*) Rubbing finally powered chalk ; (*iv*) Rubbing damp salt for cleaning paint spots.

SECTION 13

ARBORICULTURE

1. METHODS OF PLANTING TREES

There are various methods by which trees are propagated in addition to the direct sowing of seeds, the most common are described below:

Transplanting from Nurseries: Planting out of seedlings is a job that requires skill and a little carelessness results in failure. The root system of the plant must not be disturbed or injured while planting out and this can be done by carefully digging out the seedlings when the soil is wet and easy to work, along with a ball of earth around the roots and putting them in the new site just as they were in their original position. Care should be taken to see that the stem is vertical and that the roots point downwards in their natural direction without twisting or turning. Although trees may be raised by means of direct sowings or cuttings but in practice it is found much better to grow plants of various suitable species in a nursery and then to transplant nursery cultured seedling two to three years old when their root system is well developed so as to get established into the new soil.

The seedlings are planted out when they are 1 to 1.5 metres high and have 2 cm dia. of the stems, at the outbreak of the rainy season after a good shower has soaked in the soil. Some trees may be satisfactorily planted out in cold weather when the leaves have fallen. Shisham, Neem, Jaman, Siris, and Mulberry can be easily grown in a nursery and planted out. Shisham needs plenty of water; Mulberry, Neem and Siris need less water, while Kikar species can be tried in still drier places.

If plants are grown in pots, like eucalyptus, etc., the roots are sure to be pot-bound. In such cases the pots should be carried to the new site and the seedling roots bared of earth and straightened down into the holes dug for them. This greatly facilitates their subsequent growth.

Propagation by Cuttings: The cuttings should be 1.5 to 3 metres long with the lower end cut obliquely and set 60 to 90 cm in prepared holes in the ground. The upper end is plastered over with cowdung and clay. Cuttings should be made in the monsoons or cold weather.

Grafting: Consists in laying bare with a sharp knife the growing portion which lies between the back and wood of a branch. The proper season for grafting is spring, *i.e.*, immediately before fresh growth takes place.

Layering: To grow a tree by this method the branches are bent downward until they can be brought in contact with the soil and the tree is left alone for a season after which most of the branches so treated will be found to have taken root at the point of contact with the soil. Mangoes and some other trees can be grown by this method.

Collection, Maintenance and Sowing of Seeds

Seeds should not be collected unless they are ripe and before they begin to fall. A rough practical test to ensure soundness of seeds is to place some of them on an iron plate nearly red hot, when all the sound seeds will burst before burning. Another method is to place a number of seeds in a pot of earth, keep them moist and put them in a warm place in the dark. The sound seeds will germinate in a short time. The seeds should be collected in dry weather and should immediately be spread ous

to dry in the shade. When dried the seeds should be cleaned and uselest outer covering removed. As far as possible, the seeds used for sowing should be fresh, and it is best to sow seeds as soon as collected, especially, when the seeds are still oily; wet seeds on no account should be used. It is considered that seeds which have passed through the stomachs of birds or animals, germinate much better.

The seeds of different plants differ very much in the length of time for which they may be stored. Seeds unless properly stored are liable to be attacked by insects and effects of moisture. After the seeds have been dried they should be stored in air-tight tins or well corked bottles with naphthalene balls to safeguard against insect attacks.

Different seeds have different methods of sowing and all seeds require moisture, warmth and oxygen for germination, and should not therefore be denied the heat and light of the sun. A general rule is to sow a seed down to twice or thrice its diameter and in a hard soil up to 7.5 cm depth below the ground surface. Small seeds are mixed with charcoal powder or sand and broadcast in boxes or pots. When the seeds are large enough to handle individually they are put in singly to the required depth. Very small seeds are scattered as evenly as possible and then covered with sifted soil so that they are not disturbed by watering. Usually the best time for sowing is the beginning of the monsoons.

Level of Planting

(i) When the land is high and well drained the trees should be planted flush with the surrounding soil.

(ii) Where the trees are planted on the side of an embankment, no special precautions are necessary as water can escape down the sloping bank.

(iii) Where the land is low-laying, the trees should be planted on mounds about 30 cm high and a diameter at least three times the height, with sides sloping.

Trees should not be planted at a level below the surrounding surface.

Pits for Planting

The pits can be 1.2 m wide at top and 0.9 m wide at bottom and 0.9 m deep. The shape of the pit is immaterial. Unless the soil is naturally very rich it is advisable to have a good body of rich soil in the pit to help the growth of the tree. The upper soil to a depth of about 30 cm from the surface is the best and should alone be used for mixing with the manure for filling near the plant. Only one tree should be planted in each pit. The common practice of planting several trees and subsequently throwing away all but one, is not only wasteful but injurious.

The pits are generally dug a few days in advance of actual planting, the top soil mixed with manure is refilled in the pits and allowed to weather by rains or irrigation.

Watering

In early stages watering is required at shorter intervals say twice or thrice a week, but later on when the plants are 1.2 to 1.5 m high, they need not be spoon fed so often but watering should be done thrice or four times in a month. Watering is required at shorter intervals during the dry hot months and at longer intervals during winter. Sufficient quantity of water should be poured in to enable it to go deep down up to the roots. A little quantity of water too often is not helpful. Proper working of the soil at the time of planting and also at the time of watering is important.

At some places *mulching* is done which is as follows: As soon as the dry season commences, the soil round the stem 60 cm wider than the shadow at noon, is covered to a depth of about 7.5 cm with leaf mould. Watering is done over this mulched soil which retains the moisture much longer and the soil will neither cake nor crack. Earthen pitchers with holes are used at some places which are buried in the ground by the side of each plant and kept filled with water from which the water keeps on percolating and feeding the roots.

It is the roots which need water and not the stem. It is therefore well to heap up the earth round the stem and to hollow it out at a distance into a circular ditch so that when water is pured in the tree stands on a small mound surrounded by water. It is important to keep the surface soil for 30 to 60 cm distance all round the trees loose to effect proper watering.

One man can hand-water about 300 plants a day.

Fencing Young Plants—Tree-Guards

Immediately after the planting, fencing should be provided to protect the plants from grazing animals. Tree-guards are usually made from bamboos, old tar drums, or of mud walls (1.8 m internal diameter) or brick jallie. About 250 bricks are required for each brick-guard. R.C. fencings can also be made. Such fences should preferably be made in two or three pieces so that they can be easily removed and re-assembled at another place. Round shape is considered to be the best. All fencings should permit free circulation of air and sunlight.

C.P.W.D. Specifications:

Circular brick tree-guards: — Size 1.25 metres internal dia., 1.20 metres height above and 20 cm depth below ground level with half-brick wall throughout. The first two courses from bottom are built dry (without mortar) with bricks laid close to each other without honey-comb. Subsequent courses up to 30 cm from top of tree-guard are built dry in honey-combed pattern, using 15 bricks in each course, with equal openings in-between, Thereafter, the top three courses are built without honey-combing in lime-surkhi mortar (1:2) or in cement mortar 1:6.

In the bottom-most two courses and the top-most three courses bricks in adjacent courses are laid with 10 cm laps. In the honey-combed portion of the work, the vertical edges of bricks in alternate courses are in the same line. The laps of bricks on either side of opening are equal.

Flat iron tree-guards are formed of 8 nos. 25 × 3 mm mild steel flat verticals 2 metres long, 4 nos. 25 × 6 mm flat verticals 2.25 metres long, and with 3 nos. 25 × 6 mm flat rings at top, middle and bottom. Each ring is in two semi-circular halves with their ends turned out radially for a length of 4 cm at which they are bolted together with 8 mm dia. and 30 mm long bolts and nuts.

The verticals are riveted to the rings at equal spacings along the circumference with 6 mm dia. rivets, each 25 × 6 mm flat verticals being placed after every two 25 × 3 mm flat verticals. The top ends of all the verticals both 25 × 3 mm and 25 × 6 mm are cut to form pointed ends at the top and are at the same same level. The circular ring is at 10 cm, 1 metre and 1.9 metres above the ground.. The bottom 5 cm of the extra 25 cm length of the 25 × 6 mm flat verticals projecting below the 25 × 3 mm. Flat verticals are turned horizontally at right angles, the 4 nos. 20 cm long legs thus formed being buried below ground.

Tree-guards made of empty bitumen or local tar drums. These are of 53 cm average dia. and 1.3 metres of 2.0 metres height. The ends of the drums are cut. The 2.0 metres high tree-guard have four legs formed of flat section 30 mm x 3 mm of 40 cm length riveted to the tree guard with 6 mm dia rivets and projecting sufficiently below for being buried in the ground. The 1.3 metres of height just rests on the ground.

Turfing

Is artifically planting grass on side slopes of embankments and erodible soils. After the bank has been made to shape, the top earth is loosened to a depth of about 75 mm and if the soil is not suitable for grass growth, a better soil about 50 mm deep is obtained from outside and worked into the existing soil. This soil should preferably be taken from the same locality as the sod used for turfing. Any raking of the ground for loosening the soil and mixing new soil should be done parallel with the contours. Sod should be lifted in a thickness of 60 mm and cut to 30 cm by 45 cm pieces. The strips should be placed on the prepared bank slope parallel its its contours starting at the bottom. When the top is reached the edge of the sod should be turned into the surface and a thin layer of earth placed over the edge and compacted so as to divert waterover the edge and on to the top of the sod.

Turfing should be carried out immediately after the monsoons have commenced and should be kept well watered until the seeds or roots have sprouted. Watering by spray being preferable to watering by flow.

2. NURSERY

A nursery should be conveniently located for inspection and for transplantation of plants. Site should be level and of sandy loam soil where water is available according to requirement. Trenches of dimensions 30 cm top width and 20 cm bottom width and 23 to 30 cm deep should be dug 1.8 cm apart, or as necessary according to the plants. The excavated earth should be thrown 30 cm away on both sides of the trenches so that the berms are left clear for sowing seeds. The berms of the trenches should be made level before any sowings are carried out. The layout of a new nursery should be completed before the end of March.

Seedlings will grow with greater rapidly on ridges than in beds and can be more easily transplanted. The seedlings in the nursery should be raised from seeds but species which do not grow freely from seeds have to be raised from cuttings or layers of graftings. The plants should be allowed to remain in the nursery until they have become sufficiently hard to withstand planting out, but not allowed to remain so long in the nursery as to have their root system unduly cramped for space.

For transporting young plants from nursery for long distances, each ball (earth and roots) may be wrapped in leaves and grass or matting or both, or the balls may be packed in baskets or boxes and the empty space tightly filled with good earth or leaf mould and kept moist by sprinkling water.

Size of Nursery will depend upon the number and age of the seedlings to be supplied annually and the period the plants are desired to be accommodated in the nursery. Provision has to be made for paths, trenches and also for fallow, about one-third to one-fourth of the whole area, to be used in rotation every year. Failure of the plants, about one-fourths, should also be taken into account.

Approximate number of plants that can be planted in a nursery with one kg of seeds:—

Ber	270	Kikar	4000	Shisham	6000
Farash	3000	Pipal	1500	Mango (desi)	100

Number of seeds in one kg weight (approx.)

Sheesham ..	23000	Undi ..	150
Kikar ..	6000	Tun, Tundu ..	350—500
Siras ..	7400	Bhurwar, Goni ..	15000
Jamun ..	200	Nandruk, Pilala ..	14000
Banyan ..	200000	Champa, Sampighi	500
Pipal ..	15000	Khirni, Rayan ..	5400
Umber ..	10000	Asok, Devidari ..	400
Mango ..	40	Karauj ..	200
Safed or Dun Siris ..	1000	Padouk ..	1000
Candle-nut tree ..	250—300	Mohogany ..	1000
Jack, Phonas ..	150—200	Imli ..	400
Neem ..	300—400	Bahera ..	250
The best wood tree ..	20000	Eucalyptus ..	2000
Arjuna ..	300	Bhendi ..	800

3. MANURING AND IMPROVING THE SOIL

A plant requires good depth of soil to enable its root system to develop and this depends upon the kind of plant. If the roots of a tree cannot go deep down they spread sideways. Timber trees require a depth of 4.5 to 9 metres, fruit trees 3.5 to 4.5 metres, grain crops 1.2 to 1.5 metres and ordinary garden crops 0.45 to 0.9 metre. Sandy loams or clay loams are fertile and suitable for tree growth.

Sandy soil—In sandy soils large sheesham stumps having 30 to 60 cm long roots can be planted with success. Kikar and Ber can also be grown. In sandy tracts with an annual rainfall of about 200 mm the species known as "mosquito" should be tried.

Kalar soil—Pits 1 metre dia. and 1 metre deep should be dug out and filled with good soil and plantation tried.

Low lying and water logged areas—Mounds of good dry soil about 60 cm dia. and 60 cm high can be made and plantation tried on them. Willows, Eucalyptus and Arjun trees may be tried.

Stiff clay soil can be improved by the addition of sand or silt from a canal or with charcoal or brick-dust. Sandy soils can be improved by the addition of fine clay or silt from the bottom of a tank or a stagnant pool. Lime may be added with advantage to broken up grass land, both stiff clays and loose sands are improved by lime but it must be used in small quantities and as old as possible.

Manure

Leaf Moulds: The best general addition to a nursery soil is, however, vegetable mould obtained from leaves and weeds which have well decayed. Fallen leaves of avenue trees, particularly at the time when fruit-trees shed their leaves, are collected into pits 60 to 90 cm deep situated in a shady place. In about an year and a half after the pit is filled, the leaves thrown into the pit will have been converted into invaluable manure. If the pit is thoroughly soaked two or three times during the hot season the decay will be hastened. It will be found to contain worms and other vermins which should be carefully removed before it is used.

Chemical manure can be made of a mixture of equal parts of ammonium sulphate and bone-meal or super-phosphate of lime. The manure is is applied when seedlings have come up to 15 cm.

Compost: A compost is a mixture of dung and earth with other organic materials. A simple method of making compost is to sun-dry the organic matter in the air and place in a pit similar to that used for preparing leaf moulds, with a little earth sprinkled over the top and lightly watered, Further layers of organic matter and earth follow until the pit is full. The manure which can be most easily procured is cow or sheep manure. Horse-dung is somewhat less rich in manure values than cow, sheep or goat-dungs. This type of manure should never be used fresh but should be very thoroughly rotted as otherwise the grubs contained in it will attack the roots of the plants. The dung as it is collected in the pits should be coverd with a thin layer of earth to prevent the escape of valuable gases which are held together by the layer of earth. Urine also should be collected and added to the common stock of cow-dung manure. The urine should never be used as manure unless it is diluted about four times with water. If possible, the manure should be at least two years old, although the compost takes much less time to mature in hot places. About 3 to 4 tonnes of sush manure is used per 1000 sq. metres.

4. ROADSIDE PLANTING OF TREES

The trees should be best suited to the climate and the soil. Roadside trees must be fairly hardy and robust, those can stand winds and storms. The trees must be shady but not sending out large branches. Too rapid growing trees should be avoided as they are invariably short-lived and have very brittle branches. The species selected must be either truly evergreen such as the Mango, nearly evergreen such as the Margosa, or be in leaf during summer. Trees that develop straight and clean trunk up to a height of about 3 to 3.6 metres from the ground level and then spread out are suitable. The trees should be deep rooted as shallow spreading roots injure pavements by absorbing moisture from the sub-grades. All species of trees having large and thick leaves should be avoided as they require more moisture than the small leaved varieties. Trees with small sized thin leaves like the tamarind and the margosa stand draught very well as they need much less water than the large and thick leaved varieties.

Trees with valuable fruits or wood, and trees like babul which shed thorns, or those requiring much care and water for growing, are not suitable for roadside planting. Those trees which shed their leaves during April and May months are not also suitable. The nature of the road has also to be taken into consideration in selecting the type of the trees for planting. Where houses are proposed to be built close to the road or on narrow roads, thick growing large trees are not suitable.

Trees dry up the soil in summer and reduce the volume of the sub-grade which may be up to 6 per cent, and thus drop the road surface and crack it. In heavy clay soils keep fast growing trees at least 15 metres away from the road.

Do not plant trees of incongruous habits together, *e.g.*, banyan, and babul or cork tree. It is preferable to have mixed planting, *i.e.*, trees of different varieties, so that the plants flower and bear fruits in different seasons and shedding of leaves takes place in different parts of the season; in storms and gales, only some of the varieties are uprooted. Tall growing varieties with straight stems may be planted at some selected spots to serve as effective landmarks.

Trees should be planted 1.8 to 3 metres away from the outer edge of the side width (berm) or, min: horizontal clearance should be 6.7 metres for single lane roads, increasing clearance by 1 meter for each additional lane width. Rows on opposite side of road should be staggered, *i.e.*, each tree should come not opposite a tree in the row on the other side but mid-way between two trees on that side.

The trees must not interfere with the traffic on the road and should have stemps free from branches for a height of 2.4 to 3 metres on high-ways and 2.7 to 3.3 metres in streets of towns. If the branches grow horizontally it may be necessary to shorten some of these; but the height of 3.3 metres to the first branch must not be exceeded. Prune the trees in such a way that the natural shape of the foliage is retained as far as possible.

Telegraph and Telephone or Electric-lighting poles should be fixed normally at least 1.5 metres outside the existing road edge, and if possible at the road boundaries, so as not to interfere with tree growth by loppings.

No tree should be planted by the roadside before it is 1.5 metres high. Most of the failures in roadside planting are due to the use of too small seedlings or of trees which have not had their roots formed by being transplanted in the nursery.

Where trees have to be cut down for road widening purposes, those

on the north and east side, should be removed, and those on the south and west retained. The latter give most shades during the hottest part of the day. The chief influence exerted by the presence of trees upon the road surface is a reduction in daily or annual ranges of temperature of surface. This definitely prevents licking up of tar and the formation of pot holes and ruts and prolong the life of the surface. Rays of the sun have a deleterious effect both upon tar and bitumen, and the presence of trees along the road reduces that effect. The value of shade, therefore, apart from comfort, is not inconsiderable in keeping tarred surfaces in good condition. For earth and water-bound macadam roads, trees have a great effect in that by reducing the surface temperature they minimize the loss of moisture content in the soil in hot weather. Trees have the disadvantage on such surfaces of holding the dust created by motor traffic.

Suitable Spacings of Trees

Suitable spacing for roadside trees is from 9 to 15 metres according to the species of trees; the average being taken about 80 trees in a kilometre. The distance apart, however, should not be less than the diameter of the crown of a fully developed tree, and most of the trees will be about 9.15 metres apart. In dry districts the comparatively slow growing trees can be planted at larger intervals of say 15 metres, and fast-growing and less valuable trees interpolated so that some shade would be available as soon as possible. These intermediate trees can be cut down as soon as the more permanent trees attain a height of 3 to 3.2 metres. Special care should be taken for planting trees at road curves so that they do not obstruct the vision.

No tree should be planted within 6 metres of any masonry work and this distance should be increased to 24 metres minimum (prefer 60 where possible) with trees of spreading roots such as pipal, gular, pilkhan, and bargad, as the roots of these grow into the joints of masonry and damage them. It has been explained elsewhere in the book that roots of trees have damaged masonry structures at considerable distances.

Suitable Spacing:

Banyan	12 m	Mango	11-12 m
Bahera	15 m	Neem	10 m
Indian Cork	6 m	Pipal	17 m
Jamun	12-15 m	Siris	12 m
Karanj	10 m	Tamarind or Imli	12 m
Khirni	12 m	Teak	12-15m
Mahwa	12-15 m		

Numbering of Trees

Scraping the bark for painting should be done so as not to peel it off; only the dry scales should be scraped for an area 15 cm × 15 cm, at 1.2 metres above the ground level and facing the road. Nos. should be written in black paint and the background should preferably be painted in white.

5. MAINTENANCE

One Mali is considered sufficient for maintaining a length of about

3 to 6 kilometres of a road, depending upon the availability of water and whether the plants are young or grown up.

Protection Against Pests

A light spraying with a week mixture of lime water and blue vitriol (copper sulphate, 'nila thotha') solution or tobacco solution given over the plants will destroy most of the pests. Fifty grams of copper sulphate is dissolved in 1 litre of boiling water. Four litres of lime water is added afterwards at the time of spraying. Boil 100 grams of dried tobacoo leaves in 1 litre of water for about an hour. Dissolve in this 25 grams hard soap (Sunlight); add 6 litres of cold water for use.

Parasite plants should be completely removed, root and all, and destroyed. If a branch is severly infected or has otherwise become unhealthy or dead, it should be cut off to save the rest of the tree. For removing a branch, a cut should be made with a sharp saw and not an axe, close to the interfering branch and as nearly parallel to it as possible, leaving no stub of the amputated branch on the good limb or the tree trunk.

For removing a large heavy branch first cut through the bark all round with the pruning knife or chisel. Then saw it off roughly about 30 cm from the trunk, cutting underside first about half through and then completely through from the top, so that the bark may not be torn off. Small branches or twigs should be cut with a sharp pruning knife. The wounds or cuts caused by the removal of the branches should be painted over with tar to protect against moisture and decay. Or, the cuts may be treated thus: Striped bark should be traced to sound cambium and smeared with a solution of lac in methylated spirit. The central woody portion should be painted with a dilute solution of any good wood preservative such as creosote or coal-tar, putting a ring of white-wash round the branches to be cut is a good plan.

Filling Cavaties. Where a cavity has been formed in the trunk due to careless pruning, further damage should be prevented by cleaning and filling it up. This is best done as follows:

Carefully scrape out all dead wood. Wash the hollow thoroughly with a strong solution of premanganate of potash and fill it with 1: 2: 4 cement concrete rammed well and finish off with a neat cement a little below the level of the bark.

Established and grown up trees should also be looked after to prolong their lives. Stems should be coal tarred 60 cm high at least every alternate year, well pruned and wounds, if any, should be attended to. No excavation should be allowed near established trees which is likely to damage the roots.

Pruning is required for plants after they are placed in position so that they become straight growing and more dense.

The best season for pruning is the end of February or the cold weather, just before the leaves appear and the sap begins to rise when growth is least active as then the cut wounds are left exposed for a short time only. If pruning is done during warm weather, or when the trees are in full growth, the wounds bleed. and tar or bitumen if used, will

not adhere to them. The cut surface is coverd with bitumen or tar coating to prevent decay of the tree.

Pruning should begin while the tree is yonng and still in the nursery; (when they are about 2 years old) to obtain a straight healthy tree with a straight stem. Not more than two rows or tiers of branches should be removed in one year so that the tree does not become unhealthy or top heavy. Axes should not be used for hacking off the branches but pruning should be done with a pair of seissors, sharp knife or a saw.

Lopping is done to train the tree to develop a certain shape during its growth; extra branches are removed. The proper time for lopping is either September or February.

Weeding The excessive growth of grass around young plants should be removed by roots and not merely cut as that is very injurious to the growth of the new plants.

Felling Trees. When a tree is to be felled, a hollow should be dug around the base, and the trunk cut through as low down as possible, the hollow then being filled up to cover the roots.

6. DESCRIPTION OF SOME COMMON TREES

Arjun

Is a large white tree which grows very tall with smooth trunk and branches. A magnificant avenue tree; not with much shade, often found in company with teak. It furnishes a very good dark brown, heavy and strong timber suitable for masts, spars, beams and rafters.

Comes up in any rich soil, can be propagated by seed which should be collected when ripe during April/May and sown along berms of nursery trenches immediately after collection. On the day preceding sowing, the seeds are soaked in cowdung water. The seedling attains 0.6 to 1 metre height in about 3 months and grows up to 2.4 metres height in one season. Seedlings should be transplanted during the following rainy season with balls of earth attached to the roots. Does not require attention after about two years.

Asoka

A beautiful evergreen shady tree, excellent for town avenues. Easily raised from seed which ripen in July/August. The new flush of leaves is completed by April.

Bahera

Propagated by seed which are collected when ripe from Nov. to Feb. and are sown during March/April. Germination takes one to two months, and seedlings should be transplanted during the first rains before the tap root is too long. It is difficult to rear this as an avenue tree.

Banyan, Bangantrice, Bor or Bargad

Is a large spreading tree, not very suitable for roadside planting as it gives a rather low and dense shade and eventually becomes unwieldy. Its roots hang from the branches which on reaching the ground repidly take root and develop into independent stems. This tree is easily propagated either from seeds or by cuttings 2.5 to 3 meters high. The seed ripens in April or May; should be planted in May. Should be sown in pots in nursery beds; broken bricks or charcoal mixed with the soil will assist germination. Seedlings could be protected from sun and frost.

Ber, Jujuba or Regu

A very thorny moderate sized tree bearing the fruit. The tree is extraordinarily rapid in growth; when cut down after fruiting it will spring up again to a height of 4.5 metres and be covered by fruit the following season. It does not require much care for its cultivation. The wood is redish, hard, tough, durable and can take polish, and is used for well-curbs, agricultural implements and cheap building works.

Propagated by seed, sown during rains. Ripens in May/June. Grafting is done, which blossom in October. Seed should be sown deep. It may take some weeks for seeds to sprout. It is best to raise the plants in nurseries. Not grown on hills.

Cadas or Diar and Kail

Seed is sown in November, ripens in October. Watering is only required in May/June.

Coconut Palm, Nariyal

The best situation for their growth is proximity to the sea, can also be grown under artificial irrigation but not in great extremes of temperature. Coconuts are propagated by their nuts prepared in a nursery.

Date Palm or Khajur

Propagated from seeds which are sown direct in the pits, or from shoots. Ripens in August. It is best propagated by offsets as the seed always does not come out true. The young plants when 1 or 2 years old are planted out. Do not use wild palm for seed. It affords good protection against soil erosion.

Elengi, Mulsari

A moderate-sized evergreen tree with delightful fragrance diffused by numerous clusters of white starshaped flowers which appear during March/April. This tree is very suitable for small shade avenues. Propagated from seed obtained from the edible fruit which ripens in August.

Eucalyptus

Is a well-known white tall tree with long leaves which give peculiar smell.

Seeds are sown in about the middle of March in bottomless pots filled with compost made of leaf mould and soil in equal quantities and very lightly covered over with ash. The pots should be kept under a light shade of trees in small nursery beds with small cross bunds to hold water. If nursery beds are made in the open, the plants should be protected against sun. The soil in the pots must be kept moist till germination which generally requires about 2 weeks. The pots should be kept clear of weeds and only one healthy plant retained in each pot.

The plants should grow up 30 to 45 cm height in about 4 months time when they are fit for transplanting. Whole of the contents of the pots in which seedlings have been grown should be carried to the new site along with the plants.

Fig or Anjeer

Easily raised from cuttings planted in shady nursery bed. Fruit ripens from May to August. Requires careful protection.

Golden shower, Amaltas

A moderate-sized tree, somewhat unsymmetrical unless properly trained. Has long brown fruits which are used for some medicines. Timber has no special value. The tree does not need any special soil and can be grown in hot climates. Suitable for avenues. Propagated from seeds which take a few weeks to germinate. Seeds available in July/August. Seedlings stand transplanting well provided the root-system is not damaged; slow growing to begin with, they develop rapidly after a year. The tree is leafless during Feb./March.

Gulmohur

A medium-sized fast growing tree with an umbrella-like crown of finely cut bright green foliage and beautiful red flowers. Tree is leafless from March to May. Usually grown from seed which are somewhat obdurate to germinate, but it can also be raised from cuttings. The seeds are obtained by splitting open the fruits. The growth is rapid and trees begin to bear in four or five years. Suitable for town avenues in dry localities but the timber or fruit has no value. Generally grown for ornamental purposes.

Indian Beach

Is a moderate-sized, nearly evergreen, fast growing tree with a spreading shady crown of shining dark-green leaves. Prefers moist localities but would grow in the driest place. Easily raised from seed which are sown directly in May. Seeds are ready in Sept./Oct. Seed has a thick shell which should be broken without injuring the kernal and its thin covering, prior to sowing. The seeds can be preserved with the heavy shell on. The seeds fit for germination are light yellowish-green: if colour is changed, they are unfit for germination.

Indian Cork

Is a lofty tree with beautiful deep-green leaves and with fragrant flowers during Nov./Dec. Easily propagated from root-suckers but may be raised from seed. Not suitable on traffic routes or for plantation near electric and telegraph wires as it is extremely brittle and shallow rooted and is apt to be uprooted with heavy winds.

Jack

It is an important tree for the value of its fruit, where grown. The tree grows to a considerable size and has a thick foliage of dark leaves. There are two main varieties. The tree requires a deep, rich, light soil for its best growth but it can be grown in sandy soils as well. The heavy rainfall and moist air of coastal districts suits it best, but it can be grown in sheltered situations by irrigation. The size of the tree usually depends upon the soil. The tree is propagated by planting the seeds in groups of 4 or 5 in well manured pits 9 metres apart and then keeping the strongest seedlings in the nursery and transplanting 9 metres apart in the field The seeds should be sown absolutely fresh without injuring the membrane covering it. The tree bears fruits from the fifth to eighth year. Blossoms and sets its fruit in Nov. and continues to do so even until March. The tree has timber value.

Jaman, Indian Cherry

Is a large tree with fine shady verdant foliage of dark-green shiny leaves, very common in all parts of India. Is somewhat slow growing; well-known for its fruits, and very suitable for shade avenues. Seeds are collected when ripe during June/July and are sown in a nursery during the beginning of monsoons. Soil is kept moist by percolation and direct watering is avoided. Several seeds are sown together. Seedlings may be planted at the end of cold weather if the land is irrigated, if not, they should be planted during rains. Jaman roots cannot stand much exposure or injury during transplanting, therefore need careful handling.

Jhand

May be propagated by transplanting root suckers or by seed. Sown during rains or in spring. Ripens in June.

Kail, Biar, Chair, Chil

Seed is sown during Nov./Dec. Seedling should be planted out during the rains when tall enough to hold their own. Ripens in Oct. or end of autumn.

Kikar or Babul

Is a thorny tree; seldom attains a greater height than 12 to 15 metres or a greater thickness than 60 cm. It can be grown where the soil is poor or the sub-soil water level is low; can even grow in soils containing a small percentage of kallar although the growth is rather slow

Seed ripens in June/July. Before sowing the seeds are soaked for several days in cowdung and water. Best results are obtained when kikar seeds have passed through goats' stomach. Kikar seeds are sown on the berms of trenches 30 cm wide and 30 cm deep and lightly covered with earth. Sowing is done just before the break of monsoon or during the spring season. On unstable soils or on slopes kikar should be sown in circular pits, about 45 cm dia. 15 cm deep, 1.8 metres apart and in rows 3 metres distance. Plants should remain in seed beds for 9 months before transplanting. Does not require much of care or watering except when young during winters.

Kikar tree yields excellent firewood and charcoal. Heartwood is close-grained, hard, tough, heavy and durable. Is used for making wheels and spokes of country carts, handles of agricultural implements and for boat buiding. Bark of the tree is used by the tanning trade.

Kachnar

Is well-known for its fruits; easily grown from seed or from nursery. Seedlings require much light and will not spring up in shade. Ripens in June/July.

Mango

Is a common, large, evergreen. shady, long-lived tree grown in almost all parts of India and well-known for its fruit. It will grow on practically any soil but roots require good moisture, a well-drained soil, cannot stand water-logging. For full development the tree requires plenty of sunshine and protection against frost is essential of the young plants.

Mango trees are easily raised from seed but best variety is obtained by grafting. Seed is sown in August and seedlings planted in Sept. For planting, circular pits of 1.2 metres diameter and about 1.2 metres deep should be dug at least a month or two before the commencement of the planting season and filled with rich soil and watered liberally. The nut (seed) should not be kept for more than a month before sowing, and should be sown before the kernel dries. In sowing the seed should be placed flat. It will germinate in 15 to 20 days; requires watering every 2 or 3 days. The tree grows fast but should be carefully pruned in the early stages to obtain straight stems. Bears fruit during March to Sept. (only for about 3 months, varying with the climate). Seed ripens in June in the Punjab. Are generally planted about 9 to 12 metres apart.

Wood is of inferior quality, coarse and open grained, of deep grey or yellowish colour, used for inferior class of doors and windows or furniture and also as firewood. This wood should not be used for beams, battens or any load bearing structures. Planks are made for scaffoldings, etc. The mango wood decays if exposed to wet; the tree is readily attacked by white-ants and with age is also liable to develop hollows.

Mohua, Mowa

Is a very common large tree grown in the plains. It affords good shade and is considered very suitable for avenues. It is propagated by seed which should be sown absolutely fresh and directly in pots. The fruit ripens from June to August. Seeds are collected from ripe fruits and which should be removed without injuring their shells. In some places it is preferred to soak the seeds for 10 to 12 days before sowing. The plants may be but out when about an year old. In the South, a sort of liquid food is extracted out of the tree. Its wood is used for engineering works.

Malbury or Tut

It is a well-known tree for its fruit. Propagated from seed or from cuttings. Sown in June/July. Seeds ripen in April/May. It is a shady tree. Wood is useful in various ways; sports goods are made of it.

Neem, Margosa

This tree can stand a very dry climate and is useful for planting along roads in dry districts. Propagated by seed which should be collected from trees when thoroughly ripe in the month of June to August and should be sown as soon as possible after collection in well prepared nursery beds. As the seed has oil it will stand much storage. Seed should be covered lightly with earth, watered sparingly and soil kept loose. Seedlings may be transplanted when 0.9 to 1.2 metres high, and planted out with earth around the roots in Dec. or in the rains, and watered freqently. Roots and shoot cuttings can also be planted. Its timber is used for various purposes and oil extracted from seeds.

Olive or Kan

Sown at the end of cold weather in well prepared nursery beds or pots in a soil mixed with charcoal, and well watered. Seedlings should be planted in April if irrigated or during the rains. Seed ripens in August to Nov.

Pipal

A good shady tree for avenue or road-side plantation. May be planted at the end of cold weather or rains. The seed ripens in April May. Propagated by cuttings. Should be planted 15 cm apart in trenches 30 cm apart. Should not be planted near masonry structures.

Plantain or Kela

Well-known for its fruit. Grown in many provinces in India. Not suitable for road-side plantation. Propagated by root suckers which spring up from the roots and planted out in a trench dug 0.9 metre deep and filled in with a mixture of 2 parts manure and one part earth. Requires constant moisture and heavy manuring.

Shisham, Sisu, Sisoor or Tahli

It is the most important tree of Northern India well-known for its valuable wood which is dark in colour, hard and tough, used for important building works (doors, windows and heavy beams) and furniture. This wood is not usually attacked by white-ants. It is a shady tree and grows very successfully in irrigated areas with sandy and clayey loam soils and can be raised by hand watering.

Seeds should be collected from well-grown trees by shaking branches; only fresh seeds should be used. Seed ripens in Dec./Jan. and can be collected up to April, the best period for collection being the month of Feb. Sowing operations should be carried out in the beginning of the hot weather in irrigated land, the earlier the better. Sowing can also be done during winter rains. The soil should be kept moist till the seeds germinate, which commences after about 7 to 15 days.

A yellowish colour of the leaves shows either want of water or too much of wathring of the young plants. It requires much light and room to grow.

The easiest and best method of growing Seesham is from stumps of one to two years old nursery grown plants. Actual planting of the stumps should be taken up after one or two waterings of the plantation area so that the site becomes moist. More precautions are necessary to avoid water-logging of the stumps. Holes are made with an iron rod 0.7 metre long and 20 mm dia. The stumps are pushed in keeping only the shoot portion above the ground. The stumps should be irrigated soon after planting and the ground should be kept moist until they sprout. Young

trees 3 to 3.6 metres high can be planted successfully in the rains even in dry districts. The new shoots have to be protected for a year or two. Direct sowing involves great skill, labour and time, is expensive and needs more watering and can only succeed in good soil.

Plants which have suffered set-back in their growth due to any reason should be cut at ground level in the month of Dec./Jan. New shoots will come up if adequate irrigation is applied. The shoots must be properly protected if the new plants are to obtain normal conditions.

Siris (It is called Woman's Tongue in America)

A fairly fast growing common tree widely planted throughout India. A straight growing tree with large crown of handsome foliage and sweet scented flowers which appear during March/April. It grows to 12 to 15 metres in height and 1.5 to 1.8 metres in girth and is thus a good highways avenue tree. Easily raised from seeds which ripen from July to Sept. Pods are stripped open to obtain the seeds which can be preserved if necessary. Should be sown in a nursery. Seedlings should be planted out at the end of cold weather if irrigated, otherwise in rains. The timber is hard and is used in many ways; eminently suitable for making hubs of wheels.

Simbal or Simal

Is propagated from seed and also from cuttings. Seeds should be collected when ripe during the month of April/May, and should be sown as soon as possible after collection. Watering is done by percolation through the trenches. Seeds take 1 to 3 weeks to germinate. A loose sandy soil is more favourable and the plants will grow fast if well watered. The plants can be planted out when two months old or they can be planted as root and shoot cuttings at the beginning of the following rainy season, the length of the root being about 23 cm and that of the shoot about 7.5 cm. Wood is not very useful. Fruit is eaten in some parts after cooking.

Spanish Chestnut

Seed sown in March or April in prepared nursery beds. Seedlings may be planted out during winter when 3 to 4 years old. Seed ripens during August/September. The seed should be kept in dry earth until sown to protect from attacks of vermins.

Tamarind, Imli

Is a very handsome, slow growing, fairly big tree growing to a height of 21 to 24 metres and of a girth up to 7.6 metres. It is drought resistant and can thrive in any soil but is not very successful in high altitudes and water-logged areas. The tree is much valued for its fruit and is very suitable for avenues. The heart-wood is very hard, close-grained, dark-red and very hard to work and is used for various purposes and is also very good brick-burning fuel. Plants are raised from seeds which should be obtained in the ripening season from fully ripe fruits.

Teak, Sagwan, Saguna

This tree yields the best and most important timber of the tropics which is generally used for furniture making and superior class of doors and windows. It is grown in many provinces in India and has many varieties and qualities. The wood is very durable and is not attacked by white ants. The tree is best propagated by root-shoots. Grows tall and straight and has foliage of large leaves which does not give much shade.

Tun

It is a handsome shady tree grown extensively in the northern parts. Propagated from seed sown in the month of July or soon after collection in nursery. Ripens in June. Seeds should be collected from the trees when ripe in the month of May/June and should not be picked off from the ground. Seedling should be kept 1.5 to 1.8 metres apart and soil should be loosened in the vicinity of the seedlings. Young plants should be screened off from hot sun and frost. Entire plants may be transplanted with balls of earth attached to the roots or root and shoot cuttings can be planted in the usual manner in the beginning of the following rainy season. Root and shoot cuttings can also be planted during the following cold weather when the plants are leafless.

White Cedar, Bildevdari

This is a very large shady tree suitable for avenues. It yields a superior timber which is very durable, good in appearance and texture. The wood is not attacked by insects.

SECTION 14

HYDRAULICS

Page

1. HYDRAULIC DATA

1 cu. ft. of water	= 28.32 litres or kg=0.0283 cu. metres = 6.24 galls. (Imp) (gallons used in India are the Imperial gallons) = 7.48 US galls. = 62.4 lbs. fresh water = 64 lbs. sea water
1 gall. (Imp)	= 4.546 litres or kg = 4546 cu. cm = 10 lbs. = 0.16 cu. ft. = 277 cu. ins. = 8 pints = 160 fluid ounces = 1.20 US galls.
1 litre of water (also at page 1/12)	= 1 kg = 1000 cu. cm = 1 cu. dm = 2.2 lbs. = 0.22 galls. (Imp) = 0.264 galls. (US) = 0.0353 cu. ft. = 61 cu. ins.
Cu. ft./sec. (cusecs)	= 28.32 litres or 0.0283 cu. metres/sec. = 101.9 cu. metres/hr. = 1.69 cu. metres/mt. = 373.7 galls./mt. (Imp) = 448.8 galls./mt. (US)
Litres/mt. Litres/sec.	= 13.2 galls./hr. (Imp) = 19 galls./mt. (Imp) = 15.84 galls./mt. (US) = 2.119 cu.ft./min.
Galls./mt. (Imp)	= 272.758 litres/hr. = 0.0757 litres/sec. = 0.0027 cu. ft./sec.
1 cu. metre of water	= 1000 litres or 1000 kg = 35.3 cu. ft. = 220 galls. (Imp) = 264.8 galls. (US)
1 metre head of water	= 0.10 kg/sq. cm.
1 cu. cm	= 1 gram
1 kg/sq. cm	= 14.22 lbs./sq. in. = 10 metres or 32.8 ft. of water
1 gram/cu. cm	= 1000 kg/cu. metre = 1000 grams/litre = 62.4 lbs./cu. ft.

1 US gallon	= 3.785 litres or kg = 3785 cu. cm = 8.33 lbs. = 0.134 cu. ft. = 231 cu. ins. = 0.833 Imp galls.
Weight of fresh water	= 1000 kg/cu. metre
Weight of sea water	= 1026 to 1030 ,,
1 cu. metre of fresh snow	= 120 to 125 kg
1 cu. metre of compacted snow	= 250 to 280 kg
1 cu. metre of ice	= 900 kg

Acceleration due to gravity "g" (or gravity constant) is taken 9.81 metres per second per second (32.2 ft./sec./sec.)

$\sqrt{2g} = 4.43$ metres (8.025 ft.) $\qquad \sqrt{g} = 3.13$

Theoretical velocity with respect to head of water (or pressure) is taken as:

$$v = \sqrt{2\,gh} \text{ or } 4.43\ \sqrt{h} \text{ or } h = \frac{v^2}{2g}$$

h is the head of energy required to produce a velocity and is called "velocity head".

The ratio between the actual velocity and the theoretical velocity is known as *co-efficient of velocity*, which varies from 0.95 to 0.99, depending upon the shape of the orifice and the head of water under which the flow takes place.

1 British horse-power (h.p)	= 550 ft.—lbs./sec. = 1.014 metric h.p.
1 Metric horse-power (also at page 1/13)	= 75 kg—metres/sec. = 0.9863 British h.p.

Horse-power is a measure of power. (Strength of one horse is considered equivalent to that of 5 men.)

Pressure head = Pressure energy/weight of liquid.

Atmospheric Pressure (or Barometric Pressure)

The atmosphere is considered to extend to a height of at least 72 kilometres above sea level and is estimated that the maximum height of atmosphere above the earth's surface varies from 150 to 1,000 kilometres. The air (in the atmosphere) possesses weight and exerts certain pressure on the earth, and the velocity of this pressure varies according to the altitude as follows (theoretically):

Altitude in metres		0	500	1000	1500	2000	3000	4000	5000
Atmospheric Pressure	Mercury-cm	76	72	67.7	63.7	59.6	53.0	46.4	41.3
	Water-metres	10.3	9.8	9.2	8.6	8.1	7.2	6.3	5.6

Atmospheric pressure is generally expressed in centimetres of mercury column or metres of water column. Slightly different figures are taken for atmospheric pressure in the English and Metric systems. The following values are generally taken:

1 atmosphere (English) = 14.7 lbs./sq. in. (1.033 kg/sq. cm)
(at mean sea level) = 30 ins. of mercury (76 cm.) head
= 34 ft. head of water (10.32 metres)

Specific gravity of water is 1 and that of mercury 13.5.

1 atmosphere (Metric) = 1 kg/sq. cm (14.22 lbs./sq. in.)
= 73.5 cm of mercury (29 ins.) head
= 10 metres head of water (32.8 ft.)

Barometer is an instrument which measures the atmospheric pressure.

Pascal's Law—If pressure is applied on the surface of a liquid, the liquid transmits the pressure equally in all directions and with undiminished intensity.

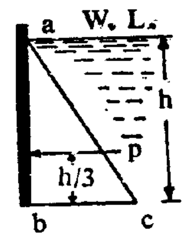
Absolute Pressure = atmospheric pressure plus gauge pressure.

Water Pressure

Fluid pressure acts normal to the surface and the intensity of pressure of any point is equal in all directions. Pressure varies directly with the depth and is independent of the size and shape of container. If a wall or dam is constructed to withhold the flowing water to form an artificial lake or reservoir, the pressure acting on the dam will depend only on the length and depth of the dam and not on the shape of the dam or the size of the lake.

Pressure Head

Intensity of pressure p at any point below water surface is wh, where w is the weight per unit of water, h is the height and or water head.



The total pressure of a section of a wall (or dam) is:

$$\frac{wh}{2} \times \text{area of surface}$$

A column of water 1 metre high produces a pressure of 0.10 kg/sq. cm.

Water standing against a wall exerts pressure P in the form of a triangle abc acting through the centre of gravity of the tringle or a distance of h/3 from the base, whence;

$$P = \frac{wh^2}{2}$$

When the upstream face is vertical the pressure is horizontal and $= wh/2$, but when the upstream face has a batter and forms angle Ø degrees with the vertical, the total pressure will be $\frac{wh^2}{2}\sec\varnothing$

General Properties of Water

(i) Water spreads evenly on a level surface and keeps its surface level while standing.

(ii) Frictional resistance to the motion of water increases with velocity.

(iii) Specific gravity of water is taken as 1, and that of mercury 13.5.

General Properties of Water and Ice

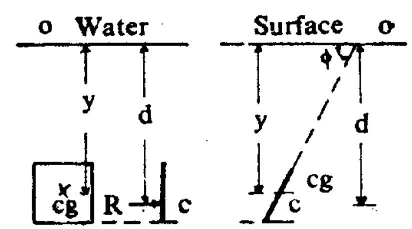
Pure water boils at 212 deg. F. or 100 deg. C. in open air at sea level, and at about 1 deg F. less for every about 158 metres above sea level. The weight of water decreases with rise in temperature and at 38 deg. C. it is 28 kg instead of 28.32. It evaporates at all temperatures.

Pure water is converted into ice at 32 deg. F. or 0 deg. C., when it weighs about 900 kg/cu. metre. Therefore, as ice water expands one-twelfth of its original bulk as water. This sudden expansive force exerted at the moment of freezing is sufficiently great to burst iron water pipes—being probably not less than 2,100 kg/sq. cm. A floating piece of ice has only 1/12th of its thickness above water.

Principle of Buoyancy

When a body is totally or partially immersed in a fluid, it is buoyed up (lifted up) by a force which equals the weight of the fluid displaced by the body. In other words — buoyant force is equal to the weight of the fluid displaced. The buoyant force acts through the centre of gravity of the displaced fluid. A floating object will sink more in a lighter fluid than in a heavier one. Sea water is heavier than river water.

Water pressure on submerged surfaces:



Total pressure on any submerged area (single face) = area × depth of the centre of gravity of the area below the free surface × unit weight of water.

Resultant Pressure R is acting at its centre of pressure at c. cg. is the centre of gravity of the area of surface A.

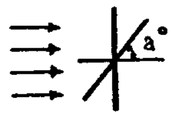
$$\text{Depth of centre of pressure} = d = \frac{Io}{Ay} = \frac{Ig + Ay^2}{Ay}$$

Io=Moment of Inertia of the figure about 00.
Ig=Ditto. about horizontal axis through centre of area. Total pressure on the plate will be W a y

For inclined surface, d will be $\frac{Io \sin^2 \varnothing}{Ay}$

$$Io = Ig + \frac{Ay^2}{\sin^2\varnothing}$$

Pressure of water in motion against a plane normal to the direction of flow



$$R = \frac{1.86 \times w \times v^2}{2g}$$

R=resistance of a plane normal to the current in kg/sq. cm.
w=Weight of water per cu. metre in kg (1000),
v=velocity of the current in metres/sec.,
g=acceleration due to gravity (9.81)

For a plane oblique to the current:

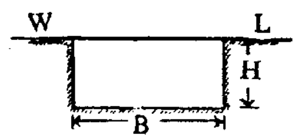
$$R^1 = R \times \frac{2 \sin^2 a°}{1+\sin^2 a°}$$

2. DISCHARGE THROUGH NOTCHES

A notch is an opening in the side of a tank or reservoir, which is a large orifice placed at the water surface so that the head of water at its upper edge is zero. (Water flows over a weir, while water flows through a notch.)

$$Q = \frac{2}{3} cB\sqrt{2g}\ H^{3/2}$$

$$= 1.84\ BH^{3/2}$$



For submerged notches
Q=c.A.V.

Q=discharge in cu. m/sec.,
c=co-efficient of discharge,
B×H=area in sq. metres

$\sqrt{2g}$ is 4.43 metres

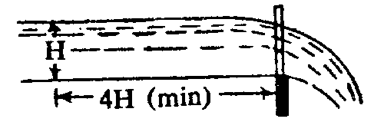
Co-efficient c is generally taken 0.60 to 0.62 for thin plates or sharp crested weirs, free overfall.

For a rectangular notch in a thin plate with two full end contractions if the length of the notch is not less than three times the head, a more accurate formula is:

$$Q = \frac{2}{3} c(B-0.2H)\ H\sqrt{(2gH)}$$

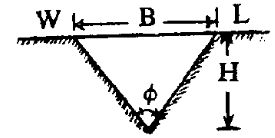
The value of the co-efficient remains more constant and is equal to 0.606.

Triangular Notches:



(*i*) $Q = \frac{4}{15} cB\sqrt{2g}\ H^{3/2}$

or $= \frac{8}{15} c\sqrt{2g}\ \tan\frac{\varnothing}{2}\ H^{5/2}$



(*ii*) *Notch with right angle:*

$$Q = \frac{8}{15} c\sqrt{2g}\ H^{5/2} \quad \text{(for sharp edged)}$$

$$= 1.417\ H^{5/2}\ \text{cu.m/sec.} \qquad \text{with } c = 0.60$$

Co-efficient of discharge c varies from 0.60 for a right angle notch to 0.62 for greater angles.

The V notch is recommended for discharges up to 90000 litres per hour. For smaller discharges a half 90 deg. V notch may be used; the discharge of water over a half 90 deg. V notch is taken half that over a 90 deg. V notch with the same head. Such notches are suitable only where the gradient is sufficiently steep and a good drop is available. For larger flows in the open, the rectangular notch is suitable and weirs for still larger flows, e.g., waterfalls in rivers, overflows in reservoirs.

The head should be measured in the corners of the flume formed by the notch bulkhead if the flume is sufficiently wide, or at a distance upstream from the weir approximately 4 times the head (90 cm min.) The depth of the bottom of the channel below the apex of the notch should not be less than 15 cm on the downstream side, while on the upstream side it should not be less than 30 cm for heads up to 23 cm, nor less than 45 cm for larger heads. The falling sheet of water should have access of air behind it. If the notch is made of wooden planks, an iron plate about 15 gauge thick should be fixed over them to keep accuracy of form and permanent correct sharpness of edges.

Discharge through 90 deg. Vee Notch, Sharp Crested, in Litres per minute.

H in ins.	0	1/8	1/4	3/8	1/2	5/8	3/4	7/8
3	134	147	162	170	195	213	232	252
4	275	289	315	340	368	392	421	446
5	507	538	571	598	640	677	715	754
6	795	836	879	923	968	1015	1063	1112
7	1163	1215	1267	1321	1376	1432	1490	1548
8	1608	1673	1738	1804	1870	1939	2009	2082
9	2156	2231	2308	2387	2465	2546	2629	2712
10	2798	2887	2975	3066	3157	3192	3276	3382
11	3420	3572	3663	3770	3876	3990	4100	4202
12	4309	4416	4522	4636	4750	4879	4986	5130
13	5244	5396	5495	5632	5761	5928	6042	6141
14	6346	6460	6551	6726	6878	7030	7220	7334
15	7575	—	—	—	—	—	—	—

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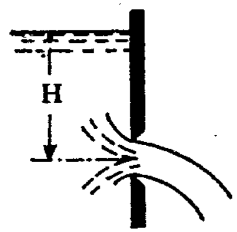
Doubling the depth increases the discharge 5.65 times; trebling the depth increases the discharge 15.57 times.

3. ORIFICES

Sharp Edged Orifice: (Small)-Free fall

$$Q = cA\sqrt{(2gh)} = 2.75A\sqrt{H}$$

Q is in cu. m/sec.



c is generally taken 0.62 for sharp edged orifices, free fall.

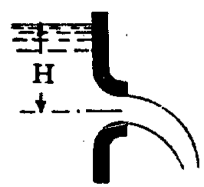
This formula is applicable so long as the orifice is small compared with the head acting on it, *i.e.*, the head on the orifice is at least twice the vertical dimension of the orifice. (For shallow waters c will be about 0.57).

Square and rectangular orifices have slightly higher discharges than circular orifices, by about 1 to 1½ per cent.

If the orifice is in a thick wall (thickness about 2.5 to 3 times the diameter of the orifice) or a tube of similar length, with large head (H), the value of co-efficient is increased up to 0.815. Thus, with the same head, the discharge from an orifice through a thick wall may be about one-third greater than that from a sharp edged orifice of the same diameter. (The tube to project outside the wall and both entries to be square edged.)

***Bell-mouthed Orifice:** Orifice with well rounded entry:

If the inner edge of an orifice is carefully shaped and well rounded the co-efficient c can be 0.97.



$$Q = 0.97\,A\sqrt{(2gH)}$$

*A bell-mouthed orifice is made of the contour of the free jet issuing.

There can be various values for the co-efficient c in between 0.62 and 0.99 according to the shape of the orifice. If the orifice is near the bottom or the side of a reservoir, c will have a value in between the two limits.

The entrance to pipes from reservoirs are often bell-mouthed to avoid the contraction and consequent loss of head which would otherwise occur.

A *sharp-edged* orifice has a sharp upstream corner so that the water in passing touches only a line. The stream of water issuing from the orifice is termed a *jet*. If the orifice discharges into the air, it is called *free fall* and if it discharges under water, it is called *submerged*.

Mouthpieces are small pipe extensions of certain lengths fitted to orifices to increase discharge based on the shape of "vena contracta." For definition of vena contracta see under "Glossary of Terms."

Values of Co-efficient of discharge c for various conditions of Orifices and Mouthpieces :

Description	Value of c
Small regular openings with shallow water	0.57
Orifices in thin plates (sharp edged)	0.62
Cylindrical short tubes	0.63
Sluices without side walls or ordinary-lock sluices ...	0.62
Conical divergent 5° mouthpieces	1.50
Short tube (external cylindrical) or orifice in thick wall with square edged entry, when the length of the tube or the thickness of the wall is 2 1/2 times to 3 times the diameter of the orifice	0.82
Ditto. when the thickness of the wall or the length of the tube is short	0.61
Re-entrant tube, length about 2 1/2 diameters (tube projecting inside the wall)	0.73
Ditto. when the length is about 1 diameter — it is called "Borda's Mouthiece"	0.52
Bell-mouth orifice (vena contracta) well rounded ...	0.97
Divergent Bell-mouth	2.00
Sharp edged orifice with converging mouthpiece (angle of convergence 13°-21')	0.98
Opening whose bottom is on a level with that of the reservoir; for sluices with walls in a line with the orifices, for bridges with pointed piers and abrupt projections ...	0.86

Flow through more than one Orifice :

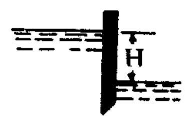
In order that two orifices in the same plane may have no effect on the discharge of each other, there should be no overlapping of the minimum clear margin or the minimum area of approach sections requisite for full contraction. Minimum margin is considered 2.75 times the least dimensions of the aperture. If the orifices are closer, the discharges are increased; the co-efficient of discharge in the case of sluice gates increases from 0.63 for a single gate to 0.65 for five gates, all open at the same time.

Short Pipes : As the cylindrical adjutage is gradually increased in length so as to become a short pipe, the frictional resistance increases, and the co-efficient diminishes as follow :

L	1	2	3	5	10	15	25	50
C	.62	.82	.815	.79	.77	.74	.71	.64
C_1	.62	.79	.780	.76	.72	.69	.63	.53
L	75	100	150	200	250	300		1000
C	.59	.55	.49	.44	.41	.38		.216
C_1	.47	.43	.36	.32	.29	.27		.155

L is length in diameters; C is co-efficient of discharge for new pipes and C_1 for old pipes.

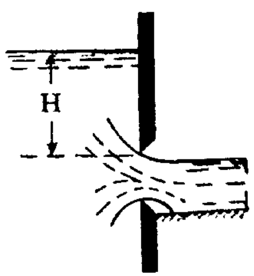
Submerged Orifices :



$$Q = cA\sqrt{(2gH)}$$

H is the difference of water surfaces each side of the orifice.

Value of c for a submerged orifice may be taken about 1 per cent less than its value for the same orifice under the same effective head when discharingg freely into the air. For small size sharp edged orifices and head up to 1.22 metres the approximate value of c is 0.60. Average value of c=0.62 for rectangular orifices.



For *Partially Submerged* small orifices the same formula applies and c may also be taken the same without much error. Some engineers take H to the centre of the orifice. There are also more accurate formulae but results do not vary much.

Some approximate values of c for *Regulator Openings*, etc. :

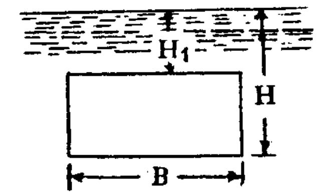
For sluices of moderate size in lock gates, etc., c=0.62;

For regulator openings between 2 to 4 metres wide, c=0.72;
For regulator openings above 4 metres wide, c=0.82;
For very large sluices and bridge opening, c=0.92.

The co-efficient for head sluices is ordinarily taken =0.80. In bridge and sluice openings provided with cut waters and wing walls, c=0.90 to 0.95.

Large Vertical Orifices with Small Heads :

(*i*) Rectangular Opening :



$$Q = \frac{2}{3} cB\sqrt{2g}\left[H^{3/2} - H_1^{3/2} \right]$$

Co-efficient c varies with the proportions of the sides of the orifice, and the head is from 0.63 to 0.60.

Unless the head over the upper sill is less than the depth of the orifice, it will be sufficiently correct in practice to use the expression for the discharge from a small orifice, *viz.*, $Q = cA\sqrt{(2gH)}$, the head H being measured to the centre of the orifice.

For heads above 60 cm the value of c for square orifices varies from 0.60 to 0.62 and for circular orifices from 0.59 to 0.61.

This formula applies for discharge of water through a *sluice gate* or large orifice where water flows into water from a higher level to a lower level.

(*ii*) Circular Openings :

$$Q = c\,\pi R\sqrt{(2gH)}\left(1 - \frac{1}{32} \cdot \frac{r^2}{H^2}\right)$$

r=radius of vena contracta, R=radius of opening, c=0.6 nearly.

(*iii*) Traingular opening :

(*a*) Base up :

$$Q = \frac{2}{3} cB\sqrt{2g}\left(\frac{2}{3}\,\frac{H^{5/2} - H_1^{5/2}}{H - H_1} - H_1^{3/2} \right)$$

(*b*) Base down :

$$Q = \frac{2}{3} cB\sqrt{2g}\left(H^{3/2} - \frac{2}{5}\,\frac{H^{5/2} - H_1^{5/2}}{H - H_1} \right)$$

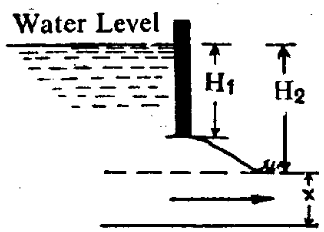
Partially Submerged :

This may be divided into two portions :
Q_1—discharge through a rectangular orifice of depth $H_2—H_1$;
and Q_2 through a submerged orifice of head H_2.

(i) Discharge through free portion (like a notch) :

$$Q_1 = \frac{2}{3} cB\sqrt{2g}\left[H_2^{3/2} - H_1^{3/2}\right]$$

(B is breadth of the weir)



(ii) Discharge of water into water:

$$Q_2 = c_1 B \times \sqrt{(2gh_2)}$$

Total discharge will be $Q_1 + Q_2$.

The value of co-efficient c may be taken equal to 0.62.

The value of co-efficient c_1 may be taken as given in the following table:

Description of orifice	Value of c_1
Sluices without side walls	0.66
Canal lock and dock gates	0.70
Sluices in lock gates	0.83
Narrow bridge openings	0.90
Wide bridge openings or very large well-built sluices with side walls	0.94
Wide opening the bed of which is level with the bottom of the reservir	0.96

Sometimes for a sluice gate or a large orifice whose bottom is on a level with the reservoir and the water flows into water the co-efficient is taken equal to 0.86.

Where the difference between H_2 and H_1 is small compared with (H_2+x), the formula becomes:

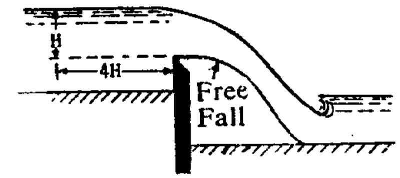
$$Q = c_1 \times B\left((H_2+x) - H_1\right) \times \sqrt{(2gH_2)}$$

4. WEIRS

Water in open channels or streams is measured through weirs. A weir is a wall or dam over which the water flows. Weirs are of two classes. (i) sharp-edged or sharp-crested which have sharpened upper edges, and (ii) broad-crested which have thick crests. Where the edge of the weir extends over the full width of the channel, it is called weir without end contractions. If the weir width is less than the width of the channel, it is known as weir with end contractions. The upper surface or edge of the weir over which the water flows is known as *crest* or *sill* of the weir. The overflowing sheet of water over the crest of the weir is called *nappe*. If the nappe discharges into air the weir is said to have a free fall or free discharge. The depth of water producing the discharge is head H shown in the sketch. (See also under "Irrigation")

Sharp-Crested Weirs :

Bazin's formula for discharge over rectangular weirs: clear overfall without end contraction:



$$Q = \frac{2}{3} cB\sqrt{2g}\ H^{3/2} = 1.84\,B\,H^{3/2}$$

taking c=0.62. Q in cu. m/sec.

Francis formula:

$$Q = 1.84\,(B - 0.1\,n\,H)\,H^{3/2} \qquad Q \text{ is in cu. m/sec.}$$

n is the number of end contractions (2, 1 or 0).

The Francis formulae apply accurately to weirs from 2.5 to 3 metres wide with heads of from 18 to 48 cm and velocity of approach between 6 to 30 cm per sec. Weir lip should be between 60 cm and 150 cm above bottom of tank.

If end contractions are perfect, it causes at each end a shortening of the effective "breadth" by approximately 0.1×H and if allowance is to be made for it, the Bazin's formula (for two end contractions) becomes:

$$Q = \frac{2}{3} c(B - 0.2H)\sqrt{2g}\ H^{3/2} = 1.84\,(B - 0.2H)\,H^{3/2}$$

When the length of a weir B is big relative to H, end contraction does not matter.

The above equations assume no *velocity of approach*, water reaching a weir has some velocity which is known as velocity of approach. This velocity tends to increase the discharge. If the head due to the velocity of approach is small compared to the measured head, it can be neglected. (This is not generally considered the, differenee being negligible, the discharge is calculated with H) Where, however, the velocity of approach has to be taken into account the formula will become:

$$Q = \frac{2}{3} cB\sqrt{2g}\left\{(H+h)^{3/2} - h^{3/2}\right\}$$

Where h is the head due to velocity of approach.

The head H+h is known as *still water head.*

Value of c varies considerably with the head, length and thickness of the weir crest and the depth of water in front of the weir. For a thin edge it is from 0.59 to 0.66. The following values may be taken in the absence of any definite figures :

Description of weir	Value of c
Broad-crested or flat-topped	0.577
With narrow crests (less than 90 cm)	0.623
Weir overfalls where breadth (B) = full width of the channel (end contractions suppressed)	0.666

c decreases with increase in width and increases with increase in depth up to 120 cm beyond which it is constant and varies from 0.62 to 0.68.

The measuring channel upstream of the weir should run straight for a distance of at least twenty times the maximum head above the crest. It should have parallel vertical smooth walls and a smooth horizontal bottom. The weir should be made perpendicular to the channel walls and bottom, and should have fairly level crest. The point where head is measured should be at least 4 H from the weir crest. Silt and/or debris accumulate on the upstream of the weir, this changes head discharge relationship, and measurements become inaccurate. When measuring discharge the sediments should be removed.

Water-fall from a weir:

$$x=\frac{4}{3}c\sqrt{Hy} \qquad \text{where :} \qquad y=D+\frac{4}{g}H$$

x=horizontal distance of the centre of the falling water from the downstream lip of the crest at any depth D below the level of the crest,

H=height of still water above weir crest.

Cipolletti Weir or Trapezoidal Weir:

This type of weir is used for larger discharges than can be measured with a triangular notch and is fixed where end contraction conditions exist (*i.e.*, the width of the discharging channel is greater than the width of the weir.)

$$Q=1.86\ BH^{3/2}\ \text{(sharp crested)} \qquad Q \text{ is discharge in cu. m/sec.}$$

B is width of the weir at bottom; H is height of water measured at a distance 4×H from the edge of the weir.

Where the depth of water on the upstream behind the weir is more than four times the head over the weir crest and where the channel edges recede a distance more than three times the head over the crest of weir on either side beyond the weir ends, the velocity of approach need not be considered.

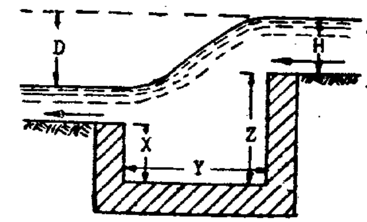
The sides of a cipolletti weir are sloped at 1 hor. to 4 vert., so made to compensate for end contractions. The depth of water should not be more than one-third the length of the sill. The velocity of approch must be practically nil and it should have free overfall. The notch is usually made in wooden planks and protected by a metal plate 1.5 to 3 mm thick.

Water Cushion :

Is a pond of water constructed below a high water fall to protect the foundation from scour and to destroy the energy and the velocity of falling water. The width of the water cushion should be equal to the width of the weir (or canal).

$$X=H+\sqrt[3]{H}\times\sqrt{D}$$

$$Y=\frac{4}{3}\sqrt{Z}\times H$$



where :

X=depth of water cushion;

H=depth of channel or height of water over top of weir;

D=difference of water level above or below the fall;

Y=width of water cushion trough;

Z=depth of water cushion trough below the weir, or the difference between the crest of the weir and the bed of cushion.

Discharge over a **Rectangular Notch** or **Weir, Sharp Crested,** per foot length, without end contractions, in Litres per minute

H in ins.	0	1/8	1/4	3/8	1/2	5/8	3/4	7/8
0	0	5.70	17.10	31.16	48.64	68.40	91.20	110.2
1	130.7	163.4	190.0	220.4	250.8	285.0	319.2	353.8
2	363.3	429.4	459.8	501.6	539.6	581.4	623.2	668.8
3	710.6	782.8	801.8	847.4	896.8	942.4	988.0	1049
4	1075	1148	1201	1250	1307	1360	1414	1474
5	1547	1585	1642	1706	1763	1824	1885	1949
6	2010	2067	2136	2200	2276	2333	2386	2462
7	2508	2576	2668	2736	2812	2880	2953	3010
8	3116	3169	3234	3310	3390	3466	3534	3625
9	3686	3762	3846	3922	4013	4096	4150	4248
10	4294	4408	4484	4560	4651	4742	4822	4902
11	4940	5077	5160	5229	5335	5419	5502	5677
12	5685	5776	5882	5951	6042	6110	6202	6308

If the depth of water is measured at the notch instead of 90 cm upstream, add 1/2 in. to depth (H) if velocity over notch is 0.61 metre (2 ft.) per sec., or 3/4 in. to depth if velocity is 0.91 metre (3 ft.) per sec. This is for rough calculations.

Approximate Discharge of water over a Rectangular Weir one foot wide, without end contractions: cubic metres per minute

$$Q=3.33\ BH^{3/2}$$

H		Disc.	H		Disc.
ft.	metres	cu. m mt.	ft.	metres	cu. m/mt.
1.00	0.305	5.59	2.10	0.610	15.81
1.10	0.335	6.45	2.25	0.686	18.73
1.20	0.366	7.34	2.50	0.762	22.11
1.30	0.400	8.27	2.75	0.838	25.51
1.40	0.427	9.24	3.00	0.914	29.06
1.50	0.457	10.25	3.50	1.067	36.62
1.60	0.490	11.27	4.00	1.219	44.76
1.70	0.518	12.38	4.50	1.371	53.41
1.80	0.549	13.49	5.00	1.524	62.55
1.90	0.579	14.62	5.50	1.676	72.16

Discharge over a Cipolletti Weir per foot of Bottom Width: cubic metres per minute

H		Disc.	H		Disc.	H		Disc.
ft.	cm.	cu. m/mt	ft.	cm.	cu. m/mt	ft.	cm.	cu. m/mt
0.1	3.05	0.180	1.1	33.53	6.525	2.1	64.00	17.21
0.2	6.10	0.506	1.2	36.57	7.436	2.2	67.05	18.46
0.3	9.14	0.929	1.3	39.62	8.383	2.3	70.10	19.73
0.4	12.19	1.431	1.4	42.67	9.369	2.4	73.15	21.03
0.5	15.24	1.999	1.5	45.72	10.39	2.5	76.20	22.36
0.6	18.29	2.629	1.6	48.77	11.45	2.6	79.25	23.71
0.7	21.34	3.313	1.7	51.81	12.54	2.7	82.29	25.09
0.8	24.38	4.047	1.8	54.86	13.66	2.8	85.34	26.50
0.9	27.43	4.830	1.9	57.91	14.81	2.9	88.39	27.93
1.0	30.5	5.655	2.0	60.96	16.00	3.0	91.44	29.39

If the cipolletti weir is *partially submerged*, fairly accurate results can be obtained by multiplying the figure for free overfall by a co-efficient which will vary with the percentage of submersion as under :—

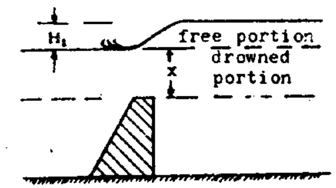
Per cent submersion	Multiplier	Per cent submersion	Multiplier	Per cent submersion	Multiplier
4	0.995	28	0.944	52	0.870
8	0.989	32	0.930	56	0.855
12	0.981	36	0.922	60	0.840
16	0.973	40	0.910	64	0.824
20	0.964	44	0.896	68	0.807
24	0.956	48	0.864	70	0.799

Discharge over a Submerged Weir :

If the level of water surface down stream of the weir is higher than that of weir crest the weir is called a submerged weir.

This may be divided into two portions—Q_1 and Q_2.

(*i*) *Discharge through free portion :*



$$Q_1=\frac{2}{3}\ cB\sqrt{2g}\ H_1^{3/2}$$

$$=2.95\ cBH_1^{3/2}$$

$$c=0.577$$

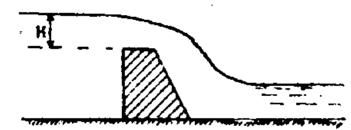
(*ii*) *Discharge through drowned portion :*

$$Q_2=c_1Bx\sqrt{(2gH_1)} \qquad c=0.80$$

(This is without velocity of approach.) The drowned portion x on the crest is called "tail water".

This is the most general and useful formula to determine the value of H_1 for a known maximum flood discharge. The value of H_1 deduced from the formula is added to the given maximum depth at which the river flows and from this heights of the wing walls and flood banks, etc., can be fixed to avoid over-topping during the floods. Increase of head due to the velocity of approach should be added by approximation.

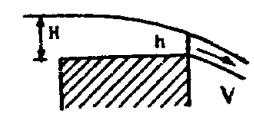
Rise of water caused by weirs :



$$H=\sqrt[3]{\frac{Q^2}{11B^2}}\ \text{(approx)}$$

B = breadth of weir

Discharge over Broad-crested Weir :



$$H=h+\frac{V^2}{2g}$$

Simple approximate expression :

$$Q=0.35B\sqrt{2g}\ H^{3/2}$$

Max. discharge occur when $h={}^2/_3\ H$. In that case

$$Q=1.65\ BH^{3/2} \qquad \text{Metric Units}$$

Flat crests decrease the discharge until the head becomes so high (between 1.5 to 2 times the width of the erest) that the water jet jumps clear of the crest and the weir becomes sharp-crested for all practical purposes. A broad flat crest may reduce the discharge 25 per cent below that of the sharp edge.

Rounded Crested Weir:

$$Q=c_1B\ H^{3/2}$$

$c_1=3$ to 4.5 depending upon shape of crest.

The effect of side concentration has been evaluated as a reduction of 0.1 H in breadth for each contraction. A deposit of silt against the upstream face of the weir will alter the discharge co-efficient.

Rounding the upstream corner of the crest of a weir increases the discharge. With flat-crested weirs Bazin found this effect to amount to as much as 13 per cent when the radius of the rounding was 10 cm and the breadth of crest 2 metres.

Inclining the upstream face away from the current decreases the contraction and increases the discharge as much as 10 per cent when the slope is one of 45-deg. If inclination in the opposite direction, the contraction is increased and the discharge decreased. With a 45-deg. slope, the decrease may be as much as 7 per cent. Inclining the downstream face does not materially alter the dicharge.

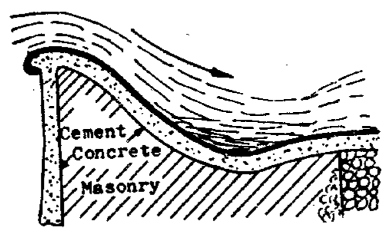
Rounding the entire crest reduces the discharge for low heads. By a combination of a rounded crest and an inclined upstream slope, the discharge may be increased 20 per cent above that of the sharp edged weir.

Ogee shaped Weir :

Ogee weir is a special type of weir employed as a spillway of a dam.

$$Q=2.21\ BH^{3/2}$$

(Due to this shape the discharging capacity is more by nearly 50 per cent than that of a rectangular broad crested weir)



Open Spillway & Siphon Spillway

An open spillway is a part of a dam over which excess water is allowed to pass as a free fall. Discharge through an open spillway is as through a rectangular weir. Co-efficient of discharge ranges from 0.6 to 0.75.

Where the upstream water level is connected to the downstream side by means of a conduit or conduits, it is known as *Siphon Spillway*. When the upstream water level rises above the crest of the spillway, water begins to flow through the conduit (siphon) over the crest creating a partial vacuum which increases the rate of flow over the crest. Under this arrangement crest of the spillway can be raised and greater amount of water can be stored in the reservoir. (See also under "Irrigation".)

$$Q=cA\sqrt{(2gH)}$$

c=0.75 for a simple type,
=0.65 per hood type,
=0.80 for volute type.

A=the total area of all the siphon spill-way opening.,

H=the difference in levels of water upstream and downstream of the spill-way, or operating head.

5. FLOW FORMULAE FOR OPEN CHANNELS, DRAINS & PIPES

(i) **Chezy's, Bazin's, Manning's and Kutter's formulae**
General Equation:

$$V=C\sqrt{RS}$$

V=velocity in metres per sec.,

C=co-efficient of roughness,

R=hydraulic radius or hydraulic mean depth,
=cross-sectional area of the liquid divided by wetted perimeter=d/4 for circular pipes.

Wetted perimeter p is the surface which is in contact with water, A is area of cross-section,

$$A=\frac{\pi d^2}{4} \text{ and } p=\pi d;\quad R=\frac{\pi d^2}{4\pi d}=\frac{d}{4}$$ for pipes running full.

S=sine of slope: gradient (vertical fall divided by the length measured along the length of the pipe, and not horizontally) or fall/length.

This formula is the basis of all formulae for flow in conduits, but the value of C is variable.

Value of C:

C may be taken 70 for glazed surfaces; 55 for cemented or smooth finished concrete; 47 for brick drains.

Kutter's and Bazin's formulae being very cumbersome have been omitted in this edition.

For long pipes the head lost due to frictional resistance is only taken into consideration for all practical purposes. A pipe is considered long when its length exceeds 100 diameters:

Hydraulic Gradient or Hydraulic Grade Line is the line joining the pressure head in the beginning (top of water level at the inlet) with the pressure head at the end (end of the pipe). In an open channel the hydraulic grade line is the water surface.

The entire pipe-line should be laid below the hydraulic grade line, if possible, to ensure full flow. If a pipe rises above the hydraulic grade line, the pressure head will become negative and flow will cease. The maximum height to which a pipe-line can be raised above the hydraulic grade line at any point is 10 metres (theoretical atmospheric pressure) at which point the weight of the water and the atmospheric pressure would be the same and balance. In practice, however, this height is only about 7.6 metres at which the water starts vaporising, forming bubbles of gas and bstructing regular flow in the pipe line. (See also under "Irrigation)

A simpler expression for C is:

$$C = \frac{1R^{1/6}}{n} \quad \text{Metric units}$$

The Manning formula, by reason of its relative simplicity, is suitable for design purposes and approaches closely to the results of the other two formulae.

Values of C in the formula $V = C\sqrt{RS}$

For Clean Asphalted Cast Iron Pipes: Metric units

Dia. of pipe-ins. (mm)	3 (80)	6 (150)	9	12 (300)	15	18 (450)
Value of C	39	47	53	58	62	64
Dia. of pipe-ins. (mm)	21	24 (600)	30 (750)	36 (900)	42	48 (1200)
Value of C	66	68	70	72	74	76

This is a compromise value of all the empirical formulae, given in the Proc. Inst. C.E. Vol. CLIII page 297.

The discharge is calculated by multiplying the velocity by the cross-sectional area of the pipe.

(*ii*) **Manning's formula :**

$$V = \frac{1}{n} R^{\frac{2}{3}} S^{\frac{1}{2}} \quad \text{(Metric units)}$$

(*iii*) **Crimp and Bruges' formula for flow in Brick Sewers:**

$$V = 83.5\, R^{\frac{2}{3}}\, S^{\frac{1}{2}} \quad \text{(Metric units)}$$

(*vii*) **Hazen and Williams' Formula** for flow in Pipes and Channels:

$$V = 0.85CR^{0.63}\, S^{0.54} \quad \text{(Metric units)}$$

Use roughest condition likely to exist for estimating capacity.

Values of co-efficient "C" in Hazen and Williams' formula for various kinds of Pipes, Sewers and Channels:

Kind of Pipe	C	Kind of Pipe	C
Cast Iron		Ordinary iron	100
New well laid	130	Old Iron	80
4 to 6 years old	120	Very rough	60
10 to 12 years old	110	**Masonry**	
13 to 20 years old	100	Brick sewers	100
26 to 35 years old	80	ditto-rough	90
Very rough	60	ditto-very rough	80
Riveted Steel Pipes		**Open Channels**	
New	110	Good masonry	80-120
10 years old	100	Rough ditto	65-75
Asbestos Cement	145	Gravel	50-80
Wrought Iron		Earth very rough	65-75
Smooth and new	120-135	ditto. with grass & weeds	35-60

Co-efficient 130 in the above formula corresponds to co-efficient (n) of 0.011 in Manning's formula.

Increase of head necessary to maintain the same pressure and velocity in old cast iron pipes:

Hazen and Williams' formula with basic C of 130

Value of C	120	110	100	90	80
Percentage increase of head	16	36	63	100	146

6. CROSS-SECTIONS TO GIVE MAXIMUM FLOWS

(See also under "Irrigation")

Maximum velocity and discharge in a rectangular channel occurs when the depth of water is half the breadth. For a given area of cross—section, the most economical section will have the least wetted perimeter. Discharge is maximum when velocity is maximum. Velocity of flow is maximum if hydraulic mean depth (R) is maximum. The best form of channel which complies with this condition is the one which has semi-circular cross-section.

Cross section of a Trapezoidal channel to have max. flow:

R to be	=0.5 d	A=area of flow,
A will be	=1.828 d²	d=depth of water,
P will be	=3.656 d	P=wetted perimeter,
Bottom width	=0.828 d	R=hydraulic mean depth
Side slope	=1 : 1	=A/P

For Circular Sections:

Depth for max. velocity=.81×diameter of pipe.

Depth for max. discharge=.95×diameter of pipe.

When depth is .25d, velocity will be 25 per cent less than the maximum:

Egg—shaped Section closed:

Proportions of cross-section which will give an approximately constant hydraulic mean depth:

Radius of crown	=R
Radius of invert	=R/2
Radius of sides and total depth ...	=3R

The following values of "n" (roughness coefficient) are general used for Manning's formula:—

0.010 for	glazed pipes, very smooth iron pipes, neat cement surface.
0.011 for	cement plaster, iron and other smooth pipes in good order.
0.012 for	ordinary iron pipes, new concrete sewers overs 1500 mm dia.
0.013 for	cast iron pipes asphalted or coated with usual bends and valves, etc., new brick sewers of all sizes, new concrete sewers of 600 to 1500 mm dia.
0.015 for	cast iron, rough brickwork, good stone-work, new sewers under 600 mm, ordinary concrete, existing sewers in average condition.
0.017 for	existing sewers in poor condition.
Open Channels	
0.015 for	channels with brick sides and concrete bottom.

0.017 for	brick lined channels.
0.020 for	rubble masonry; coarse brickwork; earth in good order; very fine gravel; rough concrete; smooth rubble. surface).
0.025 for	canals and rivers in earth in tolerably good order, free from stones and weeds.
0.030 for	canals and rivers in bad order, occasional stones and weeds.
0.035 for	canals and rivers obstructed by detritus and weeds, (very rough surface).
0.040 for	ditto, rough rubble with rough bottoms and much vegetation.
0.050 for	torrential rivers with beds covered with detritus and boulders.
0.060 for	very rough heavy grass. (See also under "Irrigation")

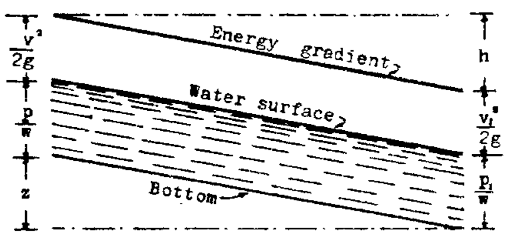
Value of C for drains and channels is given in Section 16.

7. FLOW THROUGH PIPES

Bernoulli's Theorem:

The energy head at any section in a flowing stream is equal to the energy head at any other downstream section plus the intervening losses.

$$\frac{V^2}{2g} + \frac{P}{w} + z = \frac{V_1^2}{2g} + \frac{p_1}{w} + h = \text{Total energy of water and that remains constant}$$



z is the elevation

p=pressure intensity

w=weight of water

h=friction head lost

The energy or head due to velocity is taken $\frac{V^2}{2g}$ — V being the velocity of flow and g acceleration due to gravity.

Losses in Pipe Flow

Flow in pipes is retarded due to the following causes which is taken as "head lost":

General Formulae:

(*a*) Friction in pipes: Amount of internal friction consumed by water passing through it:

$$h = \frac{4fLV^2}{d2g} \quad \text{Darcy and Weisback formula}$$

where:

h=head lost in friction; f=friction co-efficient; L=length of pipe; V=velocity of flow; g=acceleration due to gravity (9.81); d=diameter of pipe.

Darcy's frictional co-efficient varies from 0.0005 for new pipes to 0.01 for old pipes.

Computing this equation with Chezy's formula:

$$C = \sqrt{\frac{2g}{f}}$$

The value of the frictional co-efficient f depends upon the surface of the pipe and also its diameter. Loss of head in old pipes may be as much as three times the loss in new pipes. If the diameter of a pipe is increased, head loss in friction is reduced but it increases the cost. The cost of a pipe is proportional to the square of the diameter.

Minor Losses in Pipe Flow

(*b*) Entrance losses:

When water flows into a pipe from an open reservoir or a large container there is a loss of head proportional to the velocity in the pipe. In long pipes this loss of head is very small and may be neglected.

$$h = cV^2/2g$$

Approx. values of c:

End of pipe flush with reservoir wall........0.50
Pipe projecting into reservoir........0.25
Bell-mouth entrance, average........0.10

(*c*) Sudden enlargement or expansion:

$$h = \left(1 - \frac{A_1}{A_2}\right)^2 \frac{V_1^2}{2g} \quad \text{or} \quad \frac{(V_1 - V_2)^2}{2g}$$

If the enlarged section is very large as, when a pipe is joined to a tank or reservoir,

$$\text{loss} = V_1^2/2g$$

(*d*) Sudden contraction:

$$h = \frac{cV_2^2}{2g} \quad \text{c varies with the ratio } \frac{A^2}{A_1}$$

For practical purposes this loss is taken $= \frac{1}{2} \times \frac{V_2^2}{2g}$

With a reasonably rounded entrance, the loss may be reduced to about $\frac{1}{20} \times \frac{V_2^2}{2g}$

(*e*) Due to obstructions: (This is applied in certain instruments in measuring flow of water).

$$h = \left\{\frac{A_1}{c(A_1-a)}\right\}^2 \times \frac{V_1^2}{2g}$$ c is a co-efficient of contraction, may be taken=0.66.

(*f*) Due to bends, elbows, valves and fittings:

$$h = \frac{kV_1^2}{2g}$$ k is a co-efficient

(See under "Water Supply.")

where:

A_1=cross-sectional area of initial pipe or reservoir,
A_2=ditto, final pipe or reservoir,
V_1=initial velocity or velocity in the smaller section;
V_2=final velocity or velocity in the bigger section;
a=cross-sectional area of the obstruction.

Functions of Flow in a Circular Pipe Running Partly Full

Depth of flow	Area (× d²)	Proportional V	Proportional Q	Depth of flow	Area (× d²)	Proportional V	Proportional Q
0.05	0.0147	0.257	0.005	0.55	0.4426	1.034	0.586
0.10	0.0409	0.401	0.021	0.60	0.4920	1.072	0.672
0.15	0.0739	0.517	0.049	0.65	0.5404	1.099	0.756
0.20	0.1118	0.615	0.088	0.70	0.5872	1.120	0.837
0.25	0.1535	0.701	0.137	0.75	0.6318	1.134	0.912
0.30	0.1982	0.776	0.196	0.80	0·6736	1.140	0.977
0.35	0.2450	8.443	0.263	0.85	0.7115	1.137	1.030
0.40	0.3934	0.902	0.337	0.90	0.7445	1.124	1.066
0.45	0.3428	0.954	0.417	0.95	0.7707	1.095	1.075
0.50	0.3927	1.000	0.500	1.00	0.7854	1.000	1.000

To obtain area of pipe of diameter d, multiply factor in area column by d^2. Maximum discharge in a circular pipe occurs when the wetted perimeter subtends an angle of 306-deg. in the centre or depth is 0.95 d.

A Siphon (or syphon) is an inverted U-pipe used to convey water to a lower level by passing over a high level obstruction above the hydraulic grade line over which the pipe has to be laid. If a segment of a pipe line lies above the hydraulic grade line, there will be negative pressure or partial vacuum in that segment. Such a segment is said to constitute a siphon. In a siphon force of atmospheric pressure is utilized as in the case of suction pumps.

An air chamber has to be fixed at the summit to catch the accumulated air. The air chamber should have a stop cock at its connection to the siphon, and valved or plugged top end. At intervals the stop cock at the bottom can be closed, the top opened and the chamber filled with water to drive out the air, then close the top end and open the stop cock.

A siphon will operate to lift water over an obstruction up to about 7.6 metres height. The further the discharge end of the siphon is below the level of the liquid being removed, the faster will be the flow of the water.

Time for Emptying Tanks: (Also see under "Water Supply.")

$$T = \frac{2A}{c.a\sqrt{2g}}\left(\sqrt{H_1} - \sqrt{H_2}\right)$$

If the tank is completely emptied H_2 will be=0.

T=time in seconds;
H_1=head at beginning in m;
H_2=head at end in m;
A=cross sectional area of tank in sq. m;
a=area of orifice, in sq. m;
c=co-efficient of friction;
=0.62 for steel tanks, thin plates; 0.92 for bell-mouth.

Time of flow from one vessel into another:

$$T = \frac{2A_1 A_2}{A_1+A_2} \times \frac{\sqrt{H_1}-\sqrt{H_2}}{c.a.\sqrt{2g}}$$

If both the vessels have the same area:

$$T = \frac{A_1\left(\sqrt{H_1}-\sqrt{H_2}\right)}{c\ a\ \sqrt{2g}}$$

A_1=area of the vessel from which water flows to the other vessel of area A_2 in sq. m;
a=area of orifice in sq. m;
H_1=difference of head between the two vessels at the beginning in m;
H_2=difference at the end in m;

Time of Emptying and Filling a Canal Lock:

$$T = \frac{2A\sqrt{H}}{c.a\sqrt{2g}}$$

H=difference in level between the head bay and the tail bay, a=area of orifice, A=area of horizontal cross-section of lock, c=co-efficient of discharge.

Time of Emptying Reservoir with Rectangular Weir:

Bazin formula

$$T = \frac{2A}{\frac{2}{3}c B\sqrt{2g}}\left(\frac{1}{\sqrt{H_2}} - \frac{1}{\sqrt{H_1}}\right)$$

in seconds

c may be taken=0.6

A=area of reservoir in plan;
B=width of water,
H_1 & H_2=heights above the level of the sill.
(water falling from height H_1 to height H_2)

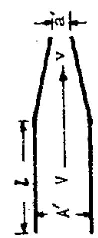
Nozzles: (Also see "Water Supply")

A nozzle is a converging mouth-piece, i.e., a small piece of pipe tapered into a smaller diameter which is fitted at the end of a hose-pipe for extinguishing fire or for "fountains", as it gives a high velocity.

Water issueing from a small orifice under pressure forms a jet. In order that a jet, supplied by a pipe, may ascend the greatest height, the pipe adjutage should be of a form which gives the highest velocity (pipe adjutage 13-deg. 24-min. has a co-efficient of discharge of 0.94 and gives highest velocity).

HEIGHT OF WATER PROJECTED FROM NOZZLES & THE QUANTITY OF WATER DELIVERED

Pressure Head				Diameter of Nozzle									
				¼ in. (6 mm)		½ in. (12 mm)		⅝ in. (15 mm)		¾ in. (20 mm)		1 in (25 mm)	
ft.	metres	lbs/sq. in.	kg/sq. cm	Height of Jet in metres	Discharge in Litres/minute	Height of Jet in metres	Discharge in Litres/minute	Height of Jet in metres	Discharge in Litres/minute	Height of Jet in metres	Discharge in Litres/minute	Height of Jet in metres	Discharge in Litres/minute
10	3.05	4.3	0.30	2.7	14	3.0	55	3.0	86	3.0	128	3.0	220
20	6.10	8.6	0.60	5.2	19	5.8	78	5.8	122	5.8	175	5.8	311
30	9.14	13.0	0.91	7.3	24	8.2	95	8.5	149	8.5	215	8.8	381
40	12.2	17.3	1.22	9.1	28	10.7	111	11.0	172	11.3	248	11.5	440
50	15.2	21.6	1.52	10.4	31	12.8	123	13.4	192	13.7	277	14.0	494
60	18.3	26.0	1.83	11.3	34	14.9	135	15.5	210	15.8	303	16.5	540
70	21.3	30.3	2.13	11.9	36	16.8	145	17.7	227	18.3	328	18.9	585
80	24.4	34.6	2.44	12.2	39	18.3	156	19.5	234	20.4	350	21.3	621
90	27.4	39.0	2.74	11.9	41	19.8	165	21.3	258	22.2	372	23.5	658
100	30.5	43.3	3.05	11.3	43	21.0	174	22.9	272	24.1	392	25.6	694
120	36.6	52.0	3.66	—	—	22.9	190	25.6	298	27.4	429	29.6	762
140	42.7	60.6	4.27			24.1	206	27.7	322	30.2	463	33.2	821
160	48.8	69.3	4.88			24.4	220	29.3	343	32.3	496	36.6	880
180	54.9	78.0	5.49			24.1	233	30.2	365	34.1	526	39.3	934
200	61.0	86.6	6.10			22.9	246	30.5	385	35.3	553	41.7	984
220	67.1	95.1	6.71			—	—	30.2	403	36.3	581	44.2	1033
240	73.1	104.0	7.31					29.3	421	36.6	607	45.7	1079
260	79.2	112.6	7.92					28.0	438	36.3	632	47.2	1123
280	85.3	121.2	8.53					25.6	455	35.7	655	48.2	1165
300	91.4	130.0	9.14					22.9	472	34.1	676	48.8	1207



The pressure of head of water is taken at the nozzle, no allowance being made for friction in the pipe. In practical calculations to determine the height to which water can be thrown, the head consumed by the friction of the water in flowing from the pump to the nozzle must be considered.

Adopted from "Richey's Reference Handbook"

Rate of Discharge from Fire Hydrants:

(Also see "Water Supply")

$Q=67\ d^2\sqrt{p}$

Q=discharge in litres/minute;
d=internal dia. of hydrant nozzle in cm;
p=pressure in kg/sq. cm

8. GLOSSARY OF TERMS

Aqueduct: Is a general term for a channel conveying water; may be a canal, pipe or tunnel. An artificial channel in which water flows with "free" surface, constructed across a valley, canal, river, drain, road or railway, may be below or above the ground level. (Also see under "Irrigation".)

Back water curve is said to be formed if the depth of water increases (due to obstruction) and a *drawdown curve* if the depth of water decreases, in the direction of the flow.

Conduit: A general term including canals, ditches, flumes, pipes, or any other means or devices for the conveyance of water or liquids, gases or even wires.

End Contraction: Contraction caused in the ends of flowing water by the ends of weir notch.

Energy: Is the capacity to perform work. In a stream or pipe the total energy at any section is the sum of its potential and kinetic energies. *Kenetic* energy is due to motion and *Potential* energy is due to position of the mass of the water.

Energy Gradient: The slope of the energy line with reference to any plane.

Energy Head: The elevation of the hydraulic grade line at any section plus the head due to the velocity of the water in that section.

Energy Line: A line joining the elevations of the energy heads of a stream. The energy line is above the hydraulic grade line by a distance equivalent to the velocity heads at all sections along the stream.

Entrance Loss: The head or energy lost due to eddies and friction at the inlet to a conduit or pipe.

Flow Line: (i) The hydraulic grade line. (ii) A conduit or pipe laid on the hydraulic gradient.

Fluids include liquids and gases.

Force is mass x acceleration.

Free Flow: A condition of flow through or over a structure not affected by submergence.

Free Overfall, or Free Fall Weir: A weir that is not submerged, that is, in which the tail water is below the crest or the flow is in no way affected by the elevation of the tail water.

Free Surface: The surface of a fluid unrestrained by a rigid boundary, particularly the exposed surface of water in an open channel or tank.

Friction Head (or Loss): The head or the energy lost as a result of friction between the moving stream of water and its containing conduit or pipe. Friction head depends upon the length, perimeter, gradient and material of the inside surface of the pipe.

Head: The height of water above any point or plane of reference; the actual or potential difference between any two points.

Hydraulic Co-efficients: The terms co-efficient of velocity, co-efficient of discharge, co-efficient of contraction, co-efficient of resistance are known as hydraulic co-efficients

Hydraulics treats of liquids in motion, particularly of the flow of water through orifices, pipes, channels, etc.

Hydrostatics treats essentially of the pressures exerted on surfaces by liquids at rest, and especially of the pressure of water.

Hydrodynamics deal with the forces or energy acting on or exerted by liquids (pressure, kinetic and potential).

Hydrology: The science treating of the waters of the earth in their various forms, their occurrence, distribution, movements, etc., often restricted to underground waters in distinction to *hydrography* as relating to surface waters.

Hydrometry: The measurement and analysis of the flow of water.

Hydrograph: Graph (or curve) showing the stage, flow, velocity, etc., of water, with respect of time.

Hydrography: Water surveys. The science of measuring, recording and analysing rainfall, flow of water, precipitation, evaporation and analogous phenomena.

Kinetic Energy or Velocity Energy: Is the ability of a mass to do work due to its velocity.

Manometer: A tube containing a liquid the surface of which moves proportional to the changes of pressure; a U-tube type of differential pressure indicator; a pressure gauge. There are two types of manometers: (i) Simple manometers which measure the pressure at a point by a glass tube having one of the ends inserted in the pipe and the other end open to atmosphere. (ii) Differential menometers measure the differenc of pressure between any two points on a pipe line. If the pressure to be measured is beyond the scope of a simple Piezometer, a double column U-tube manometer is used.

Nappe: A sheet or curtain of water over-flowing a weir, dam, etc. The nappe has an upper and a lower surface; (*ii*) stream discharging over a crest.

Orifice Meter: The installation of a Venturi Meter requires much space because of its length. When space is limited, an Orifice Meter is used for discharge measurements in pipes.

Piezometer: An instrument for measuring presssure head, usually consisting of a small pipe tapped into the side of a conduit and flush with the inside, connected with the pressure gauge, mercury, water column or other device for indicating pressure head.

Pitot Tube: Is a device used for measuring the velocity of flowing water in pipes (especially those of large diameters) or open channels. It is an open ended L-shaped glass tube. The lower end of the tube is held facing the current and at a depth at which the velocity is required. Due to the force or flow the water rises in the tube from which the velocity can be worked out and which is represented by the equation $V = c\sqrt{2gh}$. c is a co-efficient which in well formed instruments can be taken as unity. h is the height of the water in the tube above the stream. The Pitot tube may be simple or a compound one. As the velocity of flow is not the same at all points of cross-section, an average of a few readings at different points is determined. Velocity of flow is maximum in the centre of a pipe and mean velocity of flow is approximately 0.84 of the central maximum velocity. (Mean velocity of channels is discussed in the Section "Irrigation")

The best form of Pitot tube is the Pitometer and has two orifices which are bent to point upstream and downstream These two orifices are symmetrical and can be reversed to check accuracy. An improved type of Pitot tube is Amsler Hydrometrical Tube.

Pressure Head: Pressure expressed in units of vertical head of fluid.

Sharp-crested Weir: A measuring weir consisting of a thin metal plate fixed vertically, over which the water flows.

Static Head: The total head without deduction for velocity head or losses. It is the vertical height of the water above certain point or datum line.

Streamline Flow: Path of a particle of water which is flowing without turbulence.

Tail Water: The water immediately downstream of a conduit, weir or any such structure.

Turbulent Flow: A flow of a liquid in a state of turbulence (state of agitation, unsteady motion) as distinguished from *laminated* flow which occurs along steady stream lines.

Up-lift: The upward water pressure on the base of a structure.

Velocity of Approach: The mean velocity immediately upstream of a weir, dam, conduit or an orifice.

Velocity of Retreat: The mean velocity immediately downstream of a structure.

Vena Contracta: The most contracted sectional area of a stream, jet or nappe, passing over a weir or through an orifice and beyond the plane of such weir or orifice through which it issues. The boundaries of the stream or jet are parallel at this point. The vena contracta for a circular orifice is at a distance of about one-half the dia. of the opening from the plane of the orifice beyond which it begins to expand. The actual velocity at vena contracta is different from the velocity given by the equation $\sqrt{2gh}$.

Rotameter is a device for measuring discharge in a vertical segment of a pipe line.

Velocity Gradient: Rate of increase of velocity with respect to distance normal to the direction of flow.

Velocity Head: The height a body of water must fall freely under the force of gravity to acquire the velocity it has to possess.

Venturi Tube: A closed pipe which is gradually contracted to a throat causing a reduction of pressure head by which velocity through the throat may be determined. The contraction is generally followed, but not necessarily so, by gradual enlargement to the original size. Piezometer connected to the pipe above the contracting section and at the throat indicates the drop in the pressure head which is an index of flow.

Venturimeter is a measuring device employed to determine the rate of flow in pipes, fixed in any position and of practically of all diameters.

Venturi Flume is a device for measuring flow of water in an open channel. It has a constricted portion called "throat".

Working Head: The difference between supply and delivery water-levels (between the upper and lower channels when water is delivered through an outlet).

(More terms will be found under "Irrigation".)

SECTION 15

WATER SUPPLY
(Purification, Distribution & Pumping)

Page

Water gets contaminated not only at its origin. The contamination occurs owing to intermittent supply, leakages in underground pipes and improper and insanitary storage in houses and hotels. Faulty joints and valves and corroded pipes are sources of contamination with microbes. Some of the house service pipes are passing through sewage drains, sullage and manholes. They often become sanctuaries of pathogens, Mosquitoes and parasites. That is why tap water sometimes contains faecal coliforms, faecal streptococci and protozans.

1. WATER FOR DRINKING

All water supplies originate as rainfall. Rain-water is derived from water vapours rising from the seas, rivers, lakes, etc., forming into clouds and then condensing as rain, hail or snow. On reaching the earth, part of it is held by the top soil to feed vegetation, but the remainder either falls upon impervious surfaces or percolates through the earth, and it is from these two sources that we obtain our water supplies. Rain-water being evaporated or distilled water is supposed to be pure and soft. But rain absorbs gases (generally carbon dioxide), bacteria, and solid impurities such as dust particles and soot, while falling through the atmosphere and its purity on reaching the earth depends upon the cleanness of the air through which it falls. Rain-water at the beginning of a storm is dirtier than at the end of a storm. It is almost dead soft, colourless, quite corrosive and usually "flat" to the taste.

The main two basic sources of water supply are surface water and ground water. *Surface water* is a mixture of surface run-off and ground water and includes rivers, springs and lakes, etc. In most instances surface water is subject to pollution and contamination and must be treated before use for drinking purposes.

Ground water or Sub-soil water is the most important source of supply such as from wells, and has the following advantages : (a) It is often most practical and economical to obtain and distribute ; (b) It is likely to be free of pathogenic bacteria and other common impurities, and is also free of colour and turbidity (so often found in surface waters) and can generally be used without further treatment. But, water that has travelled a long distance underground is likely to be more highly mineralized than water drawn a short distance from where it entered the earth.

A potable water is one that is safe to drink, pleasant to taste and usable for domestic purposes. A contaminated water is one that contains the bacteria which cause diseases. A polluted water is one that contains substances which are undersirable or unfit for drinking or domestic use.

Essential Requirements of a Drinking Water Supply

For selection of source of supply two most important factors to be considered are the quantity and the quality.

Impurities in Water. It is almost impossible to find an absolutely pure water in nature free from danger for human consumption. Water being a universal solvent it readily dissolves and collects all kinds of impurities : gases, liquids, solids, and carries with it even insoluble matter and disease producing bacteria, while falling as rain or flowing through the ground. Impurities are broadly classified into organic and inorganic impurities, and which may be dissolved, suspended or colloidal. Organic impurities are derived from decomposition of plants and animals, contamination by sewage and manufacturing wastes, which make the water unfit for human consumption without purification. Inorganic impurities are soils, mineral salts and dissolved metals, etc. All these impurities give colour, odour, taste and turbidity to the water and produce various kinds of diseases in the human body.

Drinking Water Qualities : Drinking water must be free from (a) disease producing organisms or harmful bacteria or micro-organisms as colloidal matter ; (b) disolved poisonous chemical substances of all kinds ; (c) objectionable gases—but may contain good amount of dissolved oxygen ; (d) dissolved minerals which impart excessive hardness to water, and must not deposit sediment on standing. Drinking water should not contain any appreciable amount of suspended matter. It should, as far as possible, be colourless, odourless, cool and pleasant to the taste. Deep well waters may have a slight earthy odour. Pure water, when in large quantities, has a faint bluish tint, peaty water has a brown tint. Any other colour may indicate pollution. Warm waters are generally deficient in oxygen and are therefore somewhat unpalatable.

A clear and sparkling water is not necessarily fit for human consumtion.

The suitability of a water for any particular purpose can be determined only after a complete analysis and investigation of the source of origin of supply. A complete water analysis consists of four sections: Physical, Chemical, Bacteriological and Microscopic. For drinking water supplies the bacteriological and physical analysis are of prime importance. The microscopic analysis is necessary to determine the existence and extent of organisms that may create taste and odour problems. Raw water contains suspended, colloidal and dissolved impurities.

Frequently, water is treated not only to improve its quality from aesthetic and sanitary point of view, but also to reduce its corrosive or scaling properties which would affect the life and carrying capacity of water pipes. Except in a few special cases all surface supplies need to be filtered, and supplies from large rivers require rather more elaborate treatment. Filtration is often unnecessary in the case of underground waters which are usually hard and are frequently softened ; soft upland waters are hardened. As a final process water has to be sterilised.

Simple Tests for Detecting Dangerous Matters in Water

Presence of Organic Impurities, or Contamination from Sewage : Add four drops of a solution of potassium permanganate (Condy's Fluid) to a small glass of water and shake. If the sample is pure it will appear purple or red. If any organic impurities are present it will turn yellow or pale. Potassium permanganate can also be added in crystal form.

Presence of Lead : (a) Six drops of sulphuric acid in a small glass of water—white precipitate will be formed if lead is present. (b) By adding one drop of ammonium sulphide and stirring—a black discolouration of the precipitate will indicate the presence of lead.

Presence of Zinc or Iron : A few drops of ferro-cynide of potassium to a small glass of water will turn it green if zinc is present, and turn blue if iron is present.

Presence of Copper : Eight drops of ammonia in a small glass of water will turn it blue if copper is present.

Presence of Carbonic Acid : May be detected by shaking the sample with lime water when it will turn milky. This turbidity will disappear on the addition of hydrochloric acid.

Description of Common Impurities in Water, their Effects and Remedial Measures

Plants and Small Living Organisms

Algae are small weeds like plants which grow in water exposed to sunshine. Extent of the growths depends upon conditions of turbidity which reduces the sunlight, temperature and wind action. Presence of carbon dioxide gas in water also encourages growth of algae. Therefore, storage of water in an open reservoir provides an opportunity for algae to grow and develop. Fungi are plants that grow in absence of sunlight in water mains. Plants or organism by their death and decay influence the quality of water in a reservoir causing turbidity, colour (brown or green) and acidic taste. They often grow so densely that they clog filters. Algae give rise to a variety of troubles in water supplies.

Algae growth in small reservoirs can be controlled by roofing the reservoir in order to exclude sunlight. Taste, colour and odour can be removed by aeration and activated carbon treatment. For large areas, copper sulphate is used most frequently for the control of algae and other organisms. Copper sulphate can be added to water as it flows into a reservoir, or can be distributed over the surface of the water ; crystals of copper sulphate in burlap sacks may be dragged behind a boat. In deep reservoirs it is not necessary to treat the full depth and only the upper portions should be included. A dose of 0.5 kg. per million litres, of copper sulphate should first be given, if not successful, after two or three days the dose may be increased with a maximum of up to 2 kg per million litres which is enough to kill all plant life. A dose of about 0.7 kg kills some kinds of fish. Prompt dispersion of copper sulphate is, therefore, necessary to avoid killing fish.

Chlorine is also effective in the control of some organisms but it is more difficult to disperse uniformly, and is more useful for flowing water. Dose for chlorine is : from 0.2 to 1.5 ppm which will kill fish as well. Treatment is given prior to filtration. The chemicals must be applied rapidly in doses of adequate strength ; small doses will not be effective and nothing will be achieved by partial treatment. (Also see Section 17 under "Weed Growth".)

Any algae or other such growths in cisterns or submerged structures should be scrubbed off with wire brushes and treated with bleaching powder solution.

Iron and Manganese. Iron is often found in ground waters (well waters) in very minute traces which is dissolved from the earth's crust as a result of the presence of carbon dioxide in water coming in contact with iron ore deposits. Iron in water may also result from corrosion of the cast iron water mains. Iron is often associated with manganese found in natural waters which has almost the same effects as iron. Iron is more commonly found in natural water than is manganese. Iron compounds may be soluble, insoluble or colloidal. Iron in very small amounts (about 0.3 ppm and above) will cause unpleasant taste ; water with over even 1 ppm of iron is metallic in taste. Iron and manganese make the water bitter.

Iron salts in small quantities can produce acidic conditions in the stomach with costipation but it may not be very injurious to health on the whole. But certain natural waters containing iron are detrimental to those with high blood pressure. Iron causes reddening of the water, rust stains or discolouration of clothes washed, and incrustation in water mains and pumping fixture, thus decreasing their capacity. Iron also stains white glazed sanitary wares. The stains caused by manganese are brownish or black as against reddish stains caused by iron. Iron can be removed by "aeration" followed by sedimentation and/or filtration of the water and also to some extent with treatment through pressure filters and lime softening process. When iron and manganese are present in combination with organic matter, lime is added first and then aeration and sedimentation done. For large institutions Zeolite equipment especially built for iron removal may be considered.

Lead. As little as 1 milligram of lead dissolved in 3 litres water may be dangerous and as lead is a cumulative poison, water containing even smaller quantities than this may cause lead poisoning if taken regularly over a long period. The waters likely to take up lead are soft or acidic—rain-water and swamp or moorland waters. Also waters containing carbon dioxide in excessive quantities dissolve lead. Lead is also dissolved from lead service pipes and storage tanks painted with lead paints. See under "Lead Pipes".

Gases in Water

Carbon dioxide is the most important and trouble-some of all the gases present in waters. It is dissolved by water from the atmosphere from de-composing organic matter. Its presence encourages the formation of bicarbonates of calcium and magnesium carbonate. Natural water containing carbon dioxide will dissolve various mineral salts while flowing through the ground which give hardness, alkalinity and saltish taste to the water. Such waters cause heavy and rapid corrosion of unprotected pipes, or deposit a film of carbonate. Carbon dioxide may also stimulate heavy growth of algae if water is exposed to sunlight. Excess carbon dioxide can be removed by aeration or by addition of lime.

To prevent *corrosion* and scale due to water containing carbon dioxine and bicarbonate alkalinity it may be passed through a bed of marble stones or limestone chips. This process is simple and requires no special equipment. Carbon dioxide is used to reduce pH values and to liberate hydrogen sulphide from sulphur waters.

Hydrogen sulphide gas gives objectionable smell—has rotten egg odour. It attacks cement and concrete and destroys storage tanks built

of these materials ; it is corrosive to metals and poisonous in concentrations. This gas is a product of de-composition of organic matter and is sometimes found in deep well waters. It is removed by aeration.

Nitrogen indicates presence of organic matter due to sewage or urine pollution or vegetable matter as result of leachings from cesspools or livestock yards. Presence of nitrogen can also be the result of application of fertilizers to cultivated land. Nitrogen may be present as ammonia or nitrates, etc.

Oxygen is present in ground waters in variable quantities and is an important factor in self purification of polluted waters. Dissolved oxygen combines with iron compounds and gives water a rusty colour. Oxygen in excess helps corrosion of pipes and tanks but removes flat taste of water.

Goiter is caused by deficiency of iodine in water, particularly from certain underground supplies derived from limestone regions but some amount of iodine in water is essential. Sodium iodide is added artificially where goiter is endemic. Iodized table salt also provides the same benefits.

Acid and plumbo-solvent conditions may be corrected by the addition of chalk, hydrated lime and/or sodium silicate at the filtration works.

Mottled Tooth Enamel. Waters containing fluorines in amounts greater than 1.5 ppm are likely to cause mottling or brown-staining or pitting of the teeth of growing children and in lesser degree in adults. (In some areas in Andhra Pradesh and Punjab ground waters have fluorine as much as 2 or 3 ppm.) An amount lesser than 0.5 to 1.0 ppm is detrimental to good teeth development and fluorine has to be added to the water

Turbidity in water is due to the presence of finelly divided particles of clay, silt, and organic matter (living or dead plants or other organisms) all in suspension. Silt and sand produce less turbidity than finely divided clay. Turbidity and colour are different ; turbidity is muddiness in water and is a measure of the suspended colloidal matter which obstruct (in water) the passage of light. A perfectly clear water has a turbidity of zero. A turbidity of 2 to 5 ppm is noticeable ; with 20 ppm the water becomes cloudy in appearance and with 50 muddy. For drinking water a turbidity of less than 5 ppm is acceptable and should not be more than 10 ppm. Instruments are available for measuring turbidity. Turbidity is removed by sedimentation, with or without coagulation and filtration, according to the quantity of turbidity present.

Colour in water is due to colloidal organic matter and dissolved material resulting from decomposed vegetation, iron, manganese or other mineral matter. Presence of some colour is by no means always deleterious to health. A yellowish looking water is not necessarily impure, neither a clear bright sparking water always pure.

Odour and Taste are chiefly due to decomposing organic matter (decaying plants) in solution, mineral salts, iron oxide and manganese or industrial wastes and sewage. Odour and taste may also be due to the presence of dissolved gases like hydrogen sulphide and carbon dioxide (gases of decomposition).

Colour and Taste Removal. Colour is removed to a great extent by filtration preceded by coagulation with alum to help sedimentation. Filtration through rapid sand filters is more effective as regards colour removal than through slow sand filters. Excess chlorine treatment also removes colour. Colours that are difficult to remove may be due to metallic salts such as those of iron and manganese.

If animal bone charcoal in powdered form is put into a coloured, dirty and tasting water, much of the colour, taste and odour will be absorbed and its quality greatly improved. A concentrated solution of charcoal is made in water and it is fed into the water to be treated. If this treatment is given before sedimentation, it improves settling down of impurities in the tanks. Usual dose is 5 to 20 ppm Water is also passed through the charcoal used as a filter as described hereafter under "Activated Charcoal."

Hardness in Water. Salts of calcium and magnesium which are dissolved in water during its passage through the ground cause hardness in water. Hardness is of two distinct types, "temporary hardness" and "permanent hardness."

Temporary or Carbonate Hardness is generally due to the presence of carbon dioxide in water which dissolves insoluble carbonates of calcium and magnesium to form them into soluble bicarbonates which are held in solution. The insoluble carbonates cause deposits of scale and sludge in steam boilers, hot water heaters and vessels, when boiled or converted to steam. Temporary hardness is removed by boiling ; boiling drives off the carbon dioxide and the bicarbonates are converted into insoluble carbonates which settle as precipitates at the bottom. Temporary hardness is called "alkalinity".

Salts of sodium do not cause hardness but these salts in large doses can make water unpalatable and unfit for consumption.

Permanent or Non-Carbonate Hardness is usually caused by sulphates and sometimes by nitrates and/or chlorides of calcium and magnesium which are more stable and are not removed by boiling. These salts are corrosive to steam pipes and boilers.

Very often, a water contains a mixture of both temporary and permanent hardness and which is *total hardness*.

Hard waters promote corrosion and incrustation of pipe lines and plumbing fixtures. Form curds with soap and leave deposits on clothes and make laundering difficult. Each degree of hardness destroys about 16 grams soap in each 100 litres of water used for washing, and thus resulting in waste of soap. Water with hardness of over 100 to 150 ppm when used for laundries and boilers usually require softening. Hard waters produce intestinal troubles in human bodies and need more fuel and time to cook ; there is loss of tenderness and palatability of vegetables and meat cooked in it.

River waters are softer than well waters, and shallow well water is usually harder than deep well water. Well waters usually have dissolved salts.

Soft waters have corrosive action on a number of metals ; dissolve lead (from lead pipes and cisterns) and become dangerous to health. Pure and very soft waters corrode iron and produce tubercolation in cast iron pipes. Soft water feels smooth to the touch and readily forms a lather with soap, while hard water is rough to the skin and curdles a certain amount of soap before a lather is formed. Soft water is "flat" and tasteless whereas a moderately hard water is more palatable and often a good drinking water. Soft water can be improved in taste by the addition of a little common salt or by oxidation. If water is allowed to fall over a series of weirs, or poured from one vessel to another several times allowing it to fall some distance through the air, this will improve its quality for drinking purposes and also help to purify it.

Measurement of Hardness. The term "hard water" is indefinite and hardness has no set limit over drinking water. The common practice is to measure hardness in terms of parts per million by weight—in terms of calcium carbonate. A water having hardness not exceeding 70 ppm is termed soft and above that hard. In public water supplies, it is customary to reduce carbonate hardness to 35 to 40 ppm and total hardness to between 50 to 100 ppm.

"Heart Attack" due to Soft Water. (Extracts from The New York Times. News Service) "...Researches done in England, Canada, Finland, Netherlands and Ireland have shown a clear trend toward greater occurrence of heart attacks in populations using soft water than in those relying on hard water from wells, reservoirs and rivers. Conversely, areas supplied with hard water generally show fewer deaths from heart attacks than do areas supplied with soft water.

Soft water, which is low in calcium and magnesium salts, tends to be more acidic than hard water. Distilled water, for example, is 100 per cent soft. But just why such differences should be important to man's health is what baffles scientists. They offer, however, two possible explanations as follows :

1. Hard water could contain a factor that helps prevent attacks. This factor might be missing in soft water.

2. Soft and acidic water could leach cadmium and lead from water pipes and become harmful."

There are two systems of expressing "degree of hardness." 1 grain of hardening salt dissolved in 1 gallon (70,000 grains) of water is taken as 1 degree of hardness on Clark's (English) scale. The Metric (French) standard refers to 1 part by weight of salts in 1,00,000 parts of water and is 7/10 times the Chark's system. In other words, 1 grain of salt in 1 gallon of water is 1 degree in hardness on Clark's scale and 1-3/7 degree in hardness on Metric scale.

Hardness Table (two different systems)

I		II	
ppm	*Degree of Hardness*	*ppm*	*Degree of Hardness*
15	Extremely soft	under 50	Very soft
30	Very soft	50 to 100	Soft
45	Soft		
90	Moderately soft		
110	Moderately hard	100 to 150	Neutral
150	Hard		
170	Very hard	150 to 200	Rather hard
230	Excessively hard	200 to 300	Hard
250	Too hard for use	Over 300	Very hard

World Health Organization recommends that drinking water should not contain more than 500 ppm of dissolved solids. Water is brackish if it contains over 1000 ppm. Sea water contains 35,000 ppm.

Chloride content of Jumna Water at Delhi was found to be 31 ppm, while normal desirable is 12 ppm.

A much softer water is needed for boilers and some industries than for drinking purposes.

10 lbs/million galls. (Imp)=1 ppm=0.07 grain/gall.
8.34 lbs/million galls. US=1 ppm=0.058 grain/gall.
ppm × 0.07016=grains/Imp. gall.
ppm × 0.0584 =grain/US gall.
1 grain/Imp. gall.=14.3 ppm=119 lbs/million galls. Imp.
1 grain/US gall.=17.12 ppm=143 lbs/million galls. US
1 milligram/kilogram or litre=1 ppm.

ppm is parts per million by weight and is equal to 0.0001 per cent. To change the result into grains per Imp. gallon, divide by 14.3, and to change into US gallons, divide by 17.12.

Sea water contains from 3 to 4 per cent by weight of salts, i.e., about 30 grams per litre. Saturation point is reached when 220 grams per litre are present. Further concentrations cause salts to be deposited.

Simple Tests for Detecting Hard Waters

(*i*) Boiling in a kettle will leave white deposits at the bottom. Some hard waters will go milky with white matter floating. (*ii*) Addition of a little of powered sodium carbonate (washing soda) will form precipitates and make the water milky. (*iii*) Put some grated soap and warm the water, in hard water it will form scum and not lather. Wash a bit of cloth in water, rinse it and then leave it to dry, the cloth will dry hard.

Comparative figures of hardness for different waters are obtained by finding the amount of standard soap solution required to produce a permanent lather. The only really accurate method of determining the degree of hardness of water is by chemical analysis and that is the work of an analytical chemist.

Alkalinity or acidity. Alkalinity is due to the presence of mineral salts in water which cause temporary or carbonate hardness. Acidity is caused by the decomposition of vegetable matter, free carbon dioxide, mineral acids and sulphates of iron and aluminium. Acidity or alkalinity of water is measured in "pH" value. The pH scale runs from 0 to 14 with 7 as the point of absolute neutrality; distilled water has a pH value of 7. Water with pH value from 0 to 7 is acidic and that with pH value of 7 to 14 alkaline. (pH value of strong caustic soda is about 13 and that of concentrated sulphuric acid between 0 and 1). A liquid with pH value of 9 is ten times more alkaline than a liquid with a pH value of 8 as the scale is logarithmic, and a liquid with pH value of 5 has 1/10th the acidity of one with a pH value of 4. Test is made by comparing the colour of the water after phenolphthalein or other regent has been added, with a standard chart.

pH value of a drinking water should be as close to 7.0 as economically feasible. Waters with pH between 7.4 and 8.4 are practically inactive. Waters below 7 0 (acidic waters) are corrosive and cause tuberculation but are bacteriologically purer than alkaline waters. Waters above 7.0 cause incrustation of pipes. pH value of boiler feed water should be about 8.4.

The determination of pH value provides information concerning the corrosive character of water. Waters with pH value of below 7 (acidic waters) can be improved by the addition of a small amount of soda ash or lime before admittance to the mains. Alkalinity is also determined for the operation of water treatment of water treatment plants as small alkalinity helps in the coagulation and sedimentation, some alkalinity in water is rather essential for proper reaction while excessive alkalinity may interfere with the reaction. Alkalinity also makes water tasteful, but excessively acidic or alkaline waters are unfit for drinking and should be avoided.

Water Softening. Waters which are excessively hard have to be softened before distribution. The following processes are usually adopted :

Lime process. Hydrated or slaked lime (calcium hydroxide) is added to the water which converts the soluble bicarbonates in solution to insoluble carbonates and hydroxide which are removed by sedimentation and/or filtration. The function of lime is to remove carbon dioxide and the lime process is suitable only for the removal of calcium and magnesium carbonates. Generally an over-dose of lime up to 10 to 20 ppm over the theoretical requirement is given which is later on neutralized with carbon dioxide before the water enters the filter beds.

The amount of lime must be carefully regulated and altered to suit the variable hardness as an excess of free lime would render the water alkaline. It takes twice as much lime to remove magnesium hardness as to remove calcium hardness. The lime is made into a solution and introduced into the flow of untreated water as it enters a tank and thoroughly mixed. This is allowed to stand for 3 to 4 hours for the sludge to settle (see under "lime—soda process").

For every 100 parts of calcium bicarbonates in water, 56 parts of fresh unsaked lime are required and 200 parts of calcium carbonate (chalk) is precipitated. Reaction and settling tank is required as part of the plant. Sludge can be burnt and a stock of quicklime or calcium oxide is obtained which can be suitably hydrated and used again. Disposal of sludge sometimes becomes a problem. The use of lime for removing temporary or carbonate hardness is called Clark's process and has been in use for a long time.

Lime-softened water is not stable and is likely to cause deposits of calcium carbonate in pipes and on the sand grains of filters. To correct this the water is recarbonated, generally by the application of carbon dioxide gas, the process consists in diffusing the gas through the water. In certain cases a dose of alum is added in the settling tank.

Soda or alkali process. Soda-ash or caustic soda (sodium carbonate) is added to remove salts of sulphates, nitrates and chlorides of calcium which form the permanent hardness. The procedure is similar to the lime process, and the sludge formed is calcium carbonate. The required dosage of soda-ash is 9.25 × ppm-hardness to be removed.

Lime-soda process. This process is useful where salts of sulphates, nitrates, and chlorides of magnesium are present. Almost complete softening can, therefore, be achieved where water contains both carbonate and non-carbonate hardness, by the "lime-soda process".

After treatment in the softening plant, feed-water for boilers must be definitely, though only slightly, alkaline.

Lime-soda softening is normally carried out in works similar to those described for coagulation, sedimentation and filtration. Mechanical equipment for handling chemicals and removing sludge is usual. The lime and soda may be added, either in solution or dry, to the raw water, either separately or together. A coagulant (alum) is not necessary with clear water but if used will improve coagulation and settlement of lime sludge. A continuous feed is preferred to an intermittent and the chemicals must be thoroughly mixed with the water followed by slow agitation for about 30 to 60 minutes to allow for sufficient contact and completion of the chemical reaction. After thorough mixing the water is passed on to a sedimentation tank where sufficient retention period is allowed for the flocculent precipitate to settle down completely, after which the clarified water is allowed to pass through the filter units. Rapid gravity filtration plants with equipments for dosing lime, soda ash, etc., are available on the market.

Zeolite or Base Exchange Process

A zeolite softener is a mechanical plant similar to rapid sand filters (mechanical) in construction either open or closed, in which the filtering medium is zeolite instead of sand. The layer of zeolite is 80 to 200 cm thick. The flow of water may be either upward or downward. The zeolite most commonly used is an artificially prepared hydrated alumino-silicate in granular form which are marketed under various trade names. The filters are either of the rapid gravity type or the pressure type, more

usually the latter. For small installations, softeners in closed steel cylinders are made but in large plants gravity softening beds are generally used. The plants can be arranged for fully automatic operation including regeneration, after passage of a pre-determined quantity of water. When ordering a zeolite water softening plant, it is necessary to specify the hardness of water in ppm and the capacity desired.

The base-exchange process is suitable for waters of any temporary or permanent hardness, and is a very convenient method, easily operated and automatic. When hard water is passed through it the calcium and magnesium salts are replaced by corresponding salts of sodium with the result that water is completely softened. But the salts forming the temporary hardness become sodium carbonates and remain in the treated water. If the concentration of total salts is high it makes water unfit for consumption. Zeolite softners remove hundred per cent of the hardness in water, and for ground waters they often form the only treatment process.

The zeolite process is not suitable for acidic waters and waters containing iron or manganese, or turbid waters and waters containing sodium chloride or bicarbonates in appreciable quantities, for which pre-treatment is necessary. Presence of turbidity causes clogging of the zeolite material. A better method is to do the softening first with the lime and soda process and then pass the water through a zeolite softener to remove the residual hardness.

As zero hardness waters are unsuitable for distribution since they are likely to be corrosive, it is customary to soften by the base exchange methods only a part of the total flow and thereafter to obtain the degree of hardness desired by admixture with non-softened water. Initial softening to which the whole flow is subjected, is provided by the addition of lime prior to the sedimentation basins, after which the whole of the flow is passed through rapid sand filters and then part of the flow through zeolite filters.

A zeolite softner has to be regenerated at intervals before it ceases to be effective, and the periods have to be adjusted to suit the variable hardness. The process consists of : back-washing, brining or salting and rinsing. Selection of softening process depends upon the type of hardness present. Lime-soda process and the zeolite process have their own advantages and disadvantages. In the base-exchange process no sludge is formed as in the lime-soda process. Zeolite equipment is simple to operate, completely automatic, requires comparatively much smaller space and is especially useful in small water works not requiring filteration ; and the capital cost is generally less than that of a lime and soda plant.

For removal of temporary hardness, the lime process is usually the cheapest method. but it requires more attendance than a base-exchange plant, with water containg much permanent hardness the addition of soda increases the cost. The lime process, with or without soda, is generally effective with water containing iron or manganese without pre-treatment and the softened water is always alkaline and non-corrosive. For waters containing appreciable amounts of non-carbonate hardness, zeolite method seems to be the cheapest.

2. WATER PURIFICATION & TREATMENT

It has been stated earlier that water found in nature is seldom, if ever, all pure and free from danger for human consumption and most waters require treatment for the removal of germs of diseases, solid impurities, taste, odour, colour, iron and mineral salts, etc. In general, the treatment given to water is adjusted suitably to the characteristics of the raw water and the nature of impurities to be dealt with. The chief treatment required for surface waters is for : (*i*) contamination ; (*ii*) corrosion ; (*iii*) turbidity, taste and colour ; and for underground waters is for removal of hardness and scale forming salts, corrosive salts and excess iron and other minerals.

Particular attention should be paid to planning and lay-out of water treatment works in such a way that its capacity may be enlarged without difficulty in future for any of population. The treatment works should be located as near the source of supply as possible. The various processes involved in the purification of water are :—

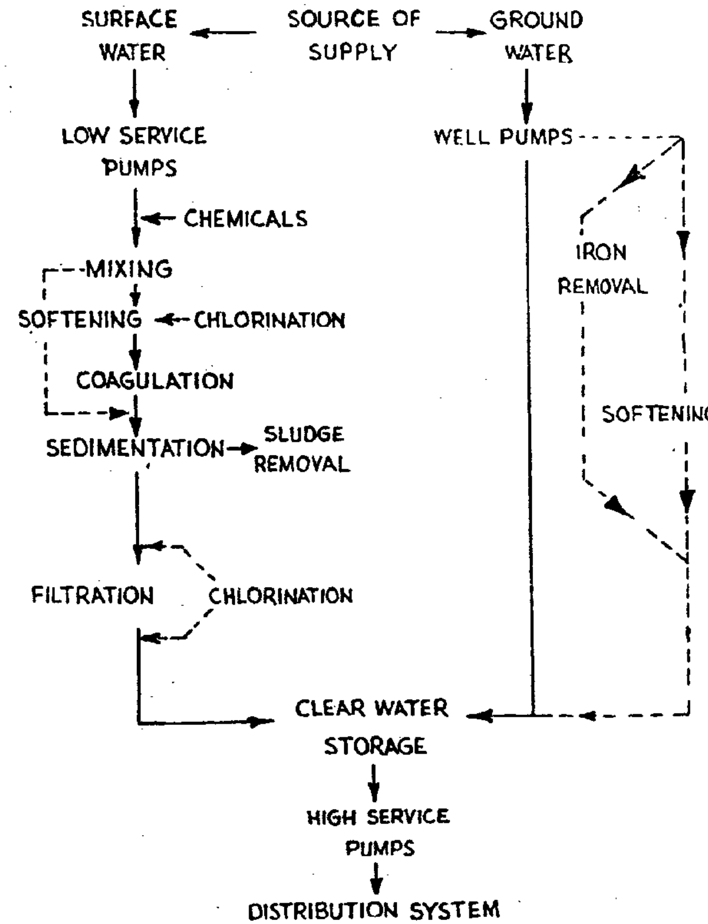
Storage tends to improve the quality of water through sedimentation of silt and other suspended matter and by the oxidation of dissolved impurities. Colour and turbidity are reduced to a considerable extent and bacteria also disappears to as much as 90 to 95 per cent which in many cases offers an effective substitute for pre-sedimentation and pre-chlorination. Polluted water derived from rivers is stored undisturbed in large impounding reservoirs which may be for a period of 2 to 4 weeks. But stored water deteriorates biologically.

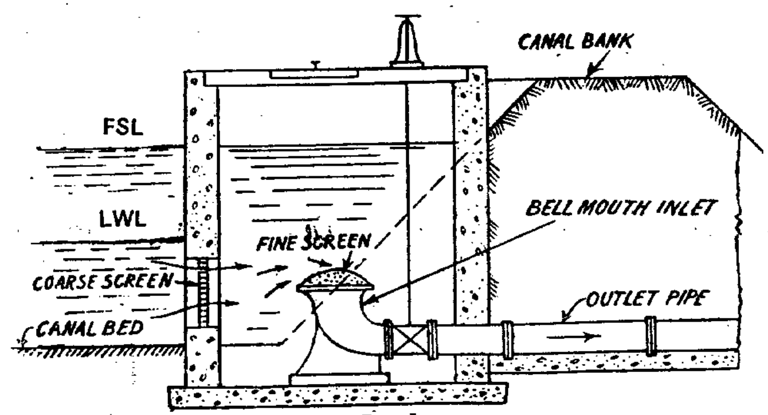
Penetration of Sunlight into Water. The disinfecting effect of sunlight and its bleaching action on the colouring matter is limited to a depth of 1.5 metres in clear water, and up to only a few centimetres in turbid waters. For this reason the growths of organisms are much less likely in turbid silt-bearing waters than in clear waters.

Screening. Screens are used at river intakes to screen out the coarser solids and floating matter. The screens used are of two types, the bar screens with openings 25 to 75 mm and the fine screens with 3 to 10 mm openings ; the two types are operated in series. Coarse screens are located at the openings in the intake structures and fine screens at the entrance to the treatment plant. Velocity of water through the screens should not be greater than 60 cm/sec. and the total area of the openings in a screen should be kept at about twice of the channel or pipe area which can be provided by widening at the screens. (Also see Section 16).

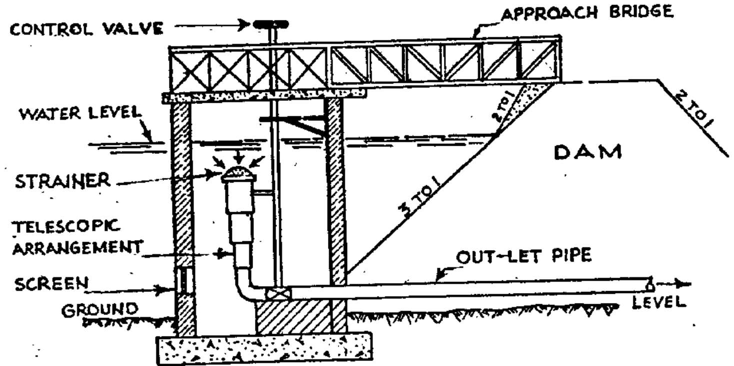
Sedimentation consists in passing water through basins or tanks in which the velocity of flow is so reduced, or the water is given a period of rest, that permit the heavy suspended matter to settle at the bottom by gravity. Sedimentation before filtration is only necessary when the water contains much suspended matter which might choke the filters. Most surface waters require a period of sedimentation so as to reduce much of

Water Purification Flow Diagram

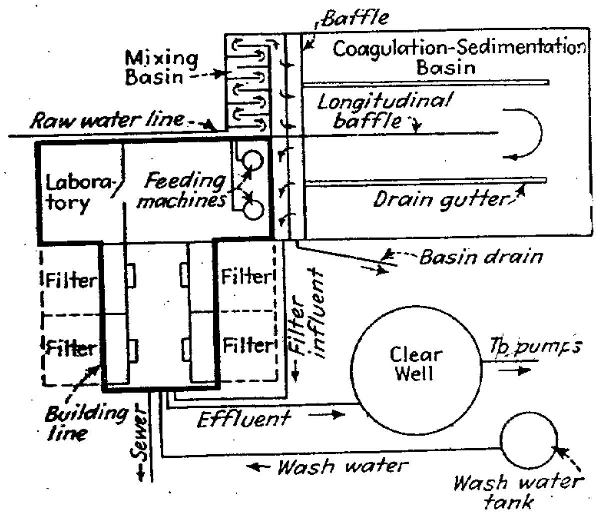




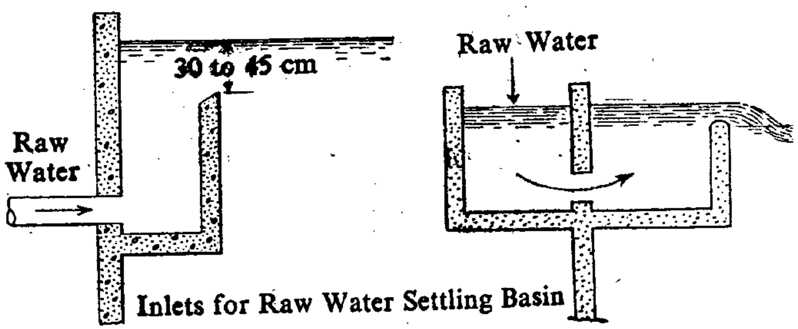
Canal Intake



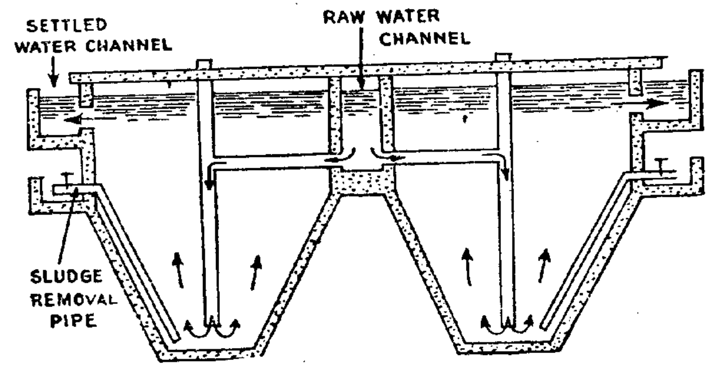
Intake Well for a Reservoir



Plan of small water treatment plant

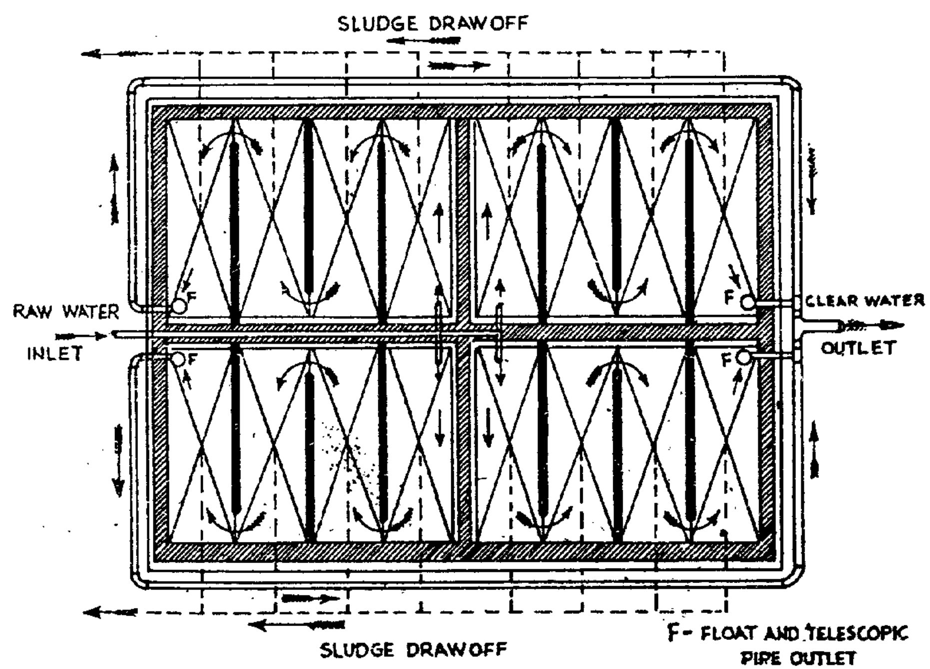


Inlets for Raw Water Settling Basin

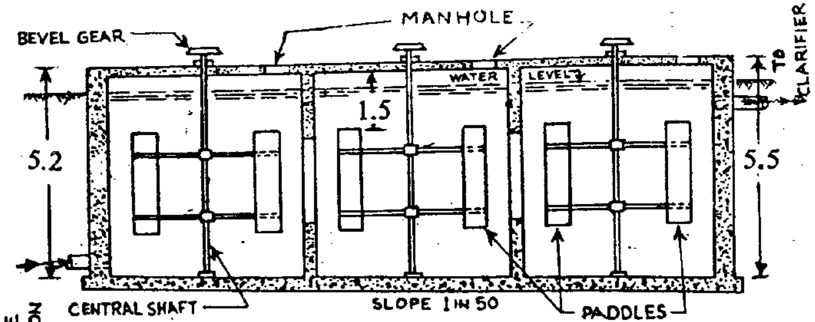


Vertical Flow Sedimentation Tank

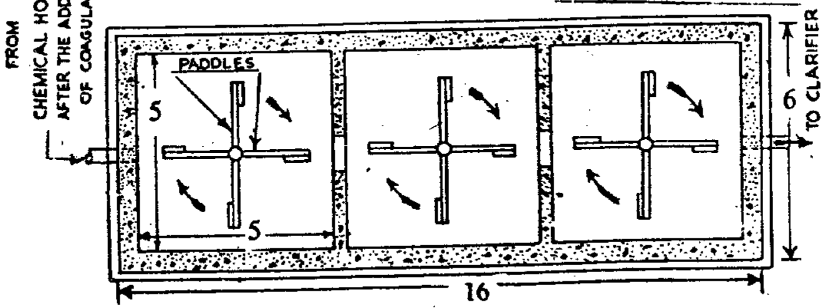
(Can be made where space is not available for horizontal flow tanks)

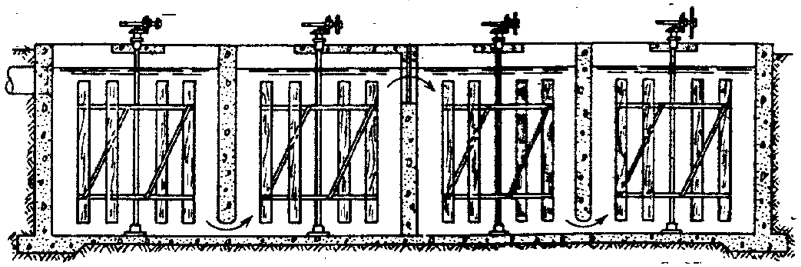


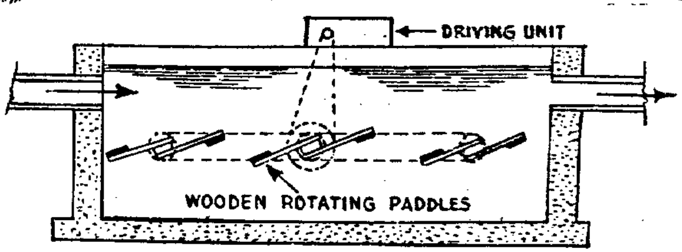
Sedimentation Basin with Round-the-End Baffles



Sectional Elevation All dimensions in metres







Mechanical flocculators may be either circular tanks with paddles revolving on a vertical shaft, or rectangular tanks with paddles revolving on an horizontal shaft. Water mixed with chemical (coagulant) enters into one end and leaves at the other end to clarifiers. This induces a spiral flow. The slowly revolving paddles are placed in the path of water and they help in the formation of flock in considerably short time. Velocity of flow is 15 to 20 cm per second.

the suspended matter and also reduce load on filters. Coagulants are en used to promote sedimentation of colloids. Waters somewhat heavily charged with sediment usually benefit from a preliminary sedimentation with a detention period of 1.5 days to 3 days, unassisted by chemical or flocculating devices, followed by the main sedimentation with a detention period of 1.5 to 4 hours. This reduces quantity of chemicals required and loading on the subsequent filtration processes. In favourable conditions about 60 to 70 per cent of the suspended matter, and about 80 to 90 per cent of bacteria present in a raw water are reduced by plain sedimentation for a period of only 24 hours.

Clarification is filtration and/or sedimentation.

Approximate time required to settle through 30 cm depth in water for different types of materials :—

Material	Size in mm	Time taken
Coarse sand	1.00	3 seconds
	0.50	6 "
Fine sand	0.25	10 "
	0.20	15 "
	0.10	40 "
Silt or colloidal matter	0.06	80 "
	0.05	100 "
	0.04	2.5 minutes
	0.02	8.5 "
	0.01	35 "
	0.005	2 hours
Clay	0.001	57 "
Bacteria	0.001	35 "

It is obvious from the above table that reduction of turbidity caused by fine clay or colloidal matter may take considerable time unless a chemical coagulant is used to hasten the settling process. Particles settle more slowly in cold water. In actual practice there is a tendency for particles to stick together and to form larger groups that might reduce the settlement time.

In the storage and plain sedimentation process suspended impurities are removed which settle down by the action of gravity and other precipitating matter but it does not ordinarily process adequate treatment of water. The production of pure and clear palatable water requires the use of filters and sterilisation.

Sedimentation Tanks or Settling Basins. These are built for preliminary settlement of solids where the water obtained is from a canal or a river and is muddy. These tanks are made either for continuous flow or for intermittent flow. In the continous type water is allowed to flow slowly through the settling tanks and in the intermittent type water is allowed to remain quiescent in the tank for a certain pre-determined period for settlement and then drawn off. Continuous flow tanks are built for a detention period of 3 to 8 hours or more flow, for plain sedimentation, and for 3 to 4 hours flow where a coagulent has been used, based on the turbidity of the water, depth and length of the tank and the time the solids will take to settle. Intermittent flow tanks are designed for storage of 2 to 3 days and are not now much in favour as at least three tanks would be required and of much bigger size too.

Sedimentation tanks are of various shapes but the most common forms are rectangular for large plants and circular or square for small plants. Circular tanks are built in duplicate and rectangular tanks may be built in two or more compartments for facility of cleaning, which form longitudinal units with long baffle walls (optional) extending up to a few centimetres below the water surface. The function of a baffle wall is to increase the effective length of travel of a particle and reduce its velocity. Most of these units provide in a single tank arrangements of coagulation and mechanical mixing and are frequently equipped with revolving sludge-removing equipment.

For rectangular tanks the proportion of breadth to length in each compartment is generally 1 : 2 with 2 : 3 min: and 1 : 4 max:. Long, relatively narrow tanks are less effected by inlet and outlet disturbances and cross currents caused by breezes. The velocity of flow in the tanks is kept 15 to 30 cm per minute. Depth is generally 3 to 4.5 metres with 1.8 metres min: and 6 metres max: Width should not exceed 12 metres. The over-flow rate is 500 to 750 litres per hour per sq. metre for plain sedimentation and 1000 to 1400 litres per hour per sq. metre for flocculated water. Depth should be fixed after allowing about 30 cm space for the sludge (not where the tank has been provided with arrangements for continuous sludge removal) and the free board at the top for waves which may be about 45 cm. A tank with minimum depth is preferable as narrow tanks are more efficient since the time taken by suspended matter to settle at the bottom is less, and shallower the basin the shorter the time for detention. Too great a surface area in relation to the tank volume is conducive to the creation of currents by the wind.

Settling tanks are generally provided near the source of supply. The inlet should be so designed that the water may enter the tank with as little disturbance as possible. A large size catchpit or trough can be made before the main sedimentation tank in which water is discharged from the inlet pipe. From this catchpit water flows gently over a weir into the settling tank. Velocity is checked and heavier matter sinks to the bottom. This deposit from the catchpit is removed daily through the scour pipe. From the catchpit water flows gently over a weir into the settling tanks. At the bottom of tank, small water cushions are formed under the weir to break the fall of the water when the tanks are being charged after cleaning. The water level in the sedimentation tank should always be controlled by equalizing the inflow and the outflow, otherwise eddies or currents may be set up which will interfere with proper sedimentation. A most common and efficient method is to collect water by decanting it over a weir across the entire width of the tank at the outlet end, with a uniform depth. The outlet should be so arranged that the draw-off will take place at from 38 to 46 cm below the surface. For outlets, some engineers prefer skimming weirs extending across the outlet end of the tank. A skimming baffle extending a few centimetres below the water surface near the outlet will prevent floating matter from going to the filters.

Vertical flow sedimentation tanks are also used although they are more expensive, but are more efficient and occupy much less space than horizontal flow tanks. These are usually made with hopper bottoms and depth varies from 4.5 to 6 metres. Raw water is admitted at the lower end of the hopper shape. Water when rising to the top leaves behind the sediments at the bottom.

To facilitate mud or sludge removal, the bottom of the tank should be sloped to one side at a grade of not less than 1 in 25, or a drain made in the centre which runs to the outlet. Sludge or mud should be removed quite often ; if the impuritites are organic they are liable to decompose if allowed to accumulate for any length of time which may cause tastes and odour. In the simplest types sludge is forced out through drains or outlets by means of fire hoses. Since mud settles in large amounts at the inlet end, it is advisable to have the basin deepest at that point. Each compartment should be provided with inlet, outlet, washout and overflow, the last two being sometimes arranged through a common set of piping. All control valves should be outside the water and fitted with proper head stocks.

Coagulation/Flocculation

A coagulant is a chemical compound which when added to water forms a heavier flocculent precipitate, known as *floc*. The process is called *flocculation*. Detention period is called *flocculent period*. The process of addition of coagulants is also known as *dosing*. *Flocculators* are basins, usually equipped with mechanical devices, in which flocculation is formed. *Coagulation basins* are sedimentation tanks in which water coming out of the flocculators is retained for some period to allow the settlement of the floc on the bottom. Coagulation basins equipped with continuously scraper arrangement to remove the settled floc continously are known as *clarifiers*. Generally, flocculators and clarifiers are provided in one single unit called *Clariflocculators*.

Very fine and light suspended matter in water which cause colour and turbidity will not settle unless allowed long periods of detention. The addition of a coagulant before sedimentation (followed by filtration) results in the removal of a greater amount of suspended matter (or turbidity) in a shorter time than by plain sedimentation ; turbidity may be reduced up to 50 per cent. By the addition of a coagulant a flocculent precipitate is formed which is a gelatinous spongy mass (called floc) having a large volume comparatively heavier than water which will settle to the bottom more rapidly and in this process of settling it will entangle and carry down with it many impurities in the water including a large proportion (may be about 50 to 70 per cent) of bacteria. Coagulation is also used as a preliminary step to filtration through rapid sand filters.

A coagulant may be added in the raw water either as dry powder or in aqueous solution ; dosing in the form of solution is better. It is held in storage tank where it is stirred continuously during use. Thorough mixing and a short reaction period between the water and the coagulant are desirable before the coagulated water enters the coagulating basin. If the reaction of the coagulant is not complete, it may result in the formation of some free sulphuric acid (or alum) in water making it unpalatable. Various types of mechanical devices (Flocculators) are available on the market, which consist of tanks having paddles on horizontal or vertical shafts. The solution storage tanks are usually rubber-lined steel for small plant and acid-resisting asphalt-lined concrete tanks for large plants. Desirable periods of retention in coagulating basins are determined by test, or experience in operation. In practice, they seldom exceed 4 to 6 hours, and 2 hours is a frequently used period. A certain period is necessary for a satifactory floc to form. This floc formation is facilitated by rapid stirring for several minutes followed by a slower agitation for 15 to 20 minutes. The velocity water from the flocculating to the settling basins should not exceed about 30 cm/sec. and which should be such that the floc neither settles nor breaks up. For water, low in suspended solids but high in colour, artificial turbidity in the form of clay is sometimes introduced to aid coagulation. Alum (aluminium sulphate, or sulphate of alumina) is the most commonly used coagulant. Alum is available in crystal form. Adequate stocks must be maintained at all times.Alumino ferric (15 to 16%) as A1203 is often used.

Gravel bed can be used effectively as a flocculator. Just like other flocculators it can be used before the setting tank. No skilled labour is required for operation and maintenance of a gravel bed flocculator as it does not have any mechanical parts. Small gravel give good results—8 mm mean size is better than 15 mm mean size.

The usual economical method is to pass the water through long channels with baffles after mixing with alum solution and then allowing the water to settle in long settling tanks. This will reduce the suspended inorganic impurities to the extent of about 50 to 75 per cent. The remaining inorganic and micro-organic impurities are got rid of by passing the water through slow or rapid sand filters and by sterilising it.

The quantity of coagulant for correct dosage (although determined by trial) will vary with other factors such as time of mixing and water temperature. The use of the minimum quantity of coagulant determined to be effective in producing good flocculation in any given water, will usually require a fairly long mixing period, varying from 15 to 30 minutes in summer and from 30 to 60 minutes in colder months, as water temperature approach the freezing point.

For proper coagulation the dose of alum must be correctly determined by trial. For clear waters it may be as low as 7 grams, and in the case of muddy water from a river, it may be up to 50 grams per 1000 litres. The normal amount used in practice varies between 15 to 20 grams per 1000 litres of water. The quantity of alum must be such as to produce visible floc. A floc heavy enough to settle in the tank must be maintained at all times or the filters will be clogged. Very finely divided suspended matter is more difficult to coagulate than coarse particles, necessitating a larger quantity of coagulant for a given turbidity.

The control of pH is an important factor for proper coagulation. The most efficient coagulation occurs at some definite pH value which depends upon the water and the coagulant used. With alum water must be alkaline ; optimum coagulation for most waters occurs with pH value between 6 and 8. Alkaline salts are present in most hard waters in sufficient quantity to react with alum, but in soft waters it is necessary to add a small quantity of lime or soda ash to increase the pH value of the water. (Sulphuric acid is added to reduce the pH value.) The pH value is changeable with the same water taken at different times. Alum increases the acidity of the water treated, and if an excessive dose were administered it would attack the iron pipes in the distribution system. Alum also increases the permanent hardness of water; if 10 ppm of alum is added the increase in hardness is about 6 ppm.

Another coagulant is ferrous sulphate, which is sometimes used with lime for highly turbid waters, is cheaper and the floc is heavier and sinks more rapidly, but alum is preferred, since the use of ferrous sulphate requires more skill and control. When ferrous sulphate is used as a coagulant, the pH should be maintained above 9.5 to ensure complete precipitation of the iron. Sand, alum and dry bone powder are also used.

STERILISATION

Sterilisation of water is necessary to kill pethogenic bacteria of water-borne diseases to make it safe for human consumption. Although by filtration all suspended impurities together with most of the organic impurities (bacteria) (may be, up to 90 percent or more) are removed but still some of these bacteria are very dangerous and may not be removed by simple filtration, therefore, water has to be sterilised where contamination is suspected.

There is every chance of water being contaminated during distribution, especially in an intermittent system of supply where the pipes remain empty for long periods. Sterilisation of water can be done in a number of ways but chlorination has been found to be the most practical, effective, cheapest and convenient method for public water supplies. Chlorine is available in solid, liquid, and gaseous form. In solid form it is used as bleaching powder containing about 30 to 35 percent of available chlorine.

Chlorination. Chlorine is very active agent and reacts quickly with

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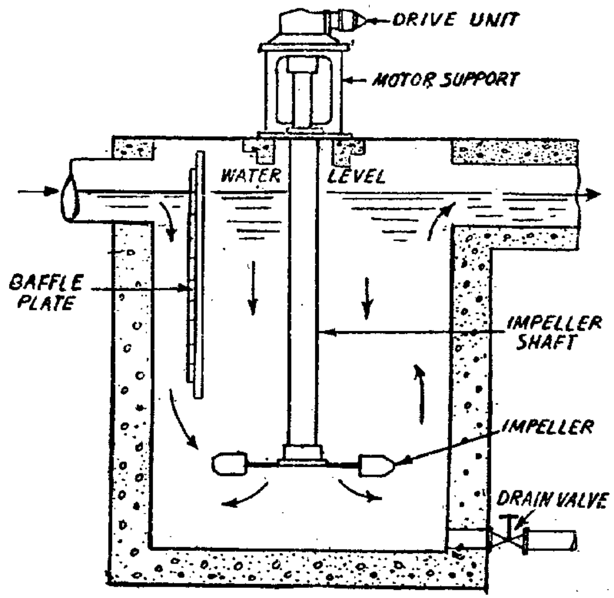
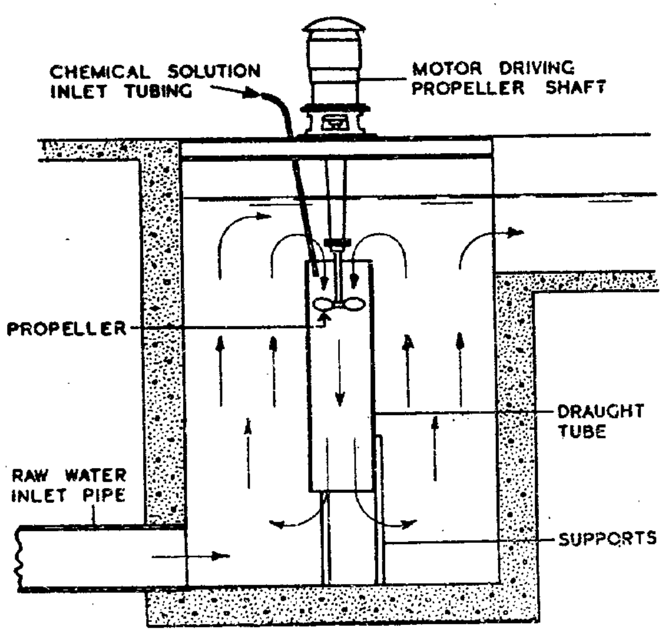
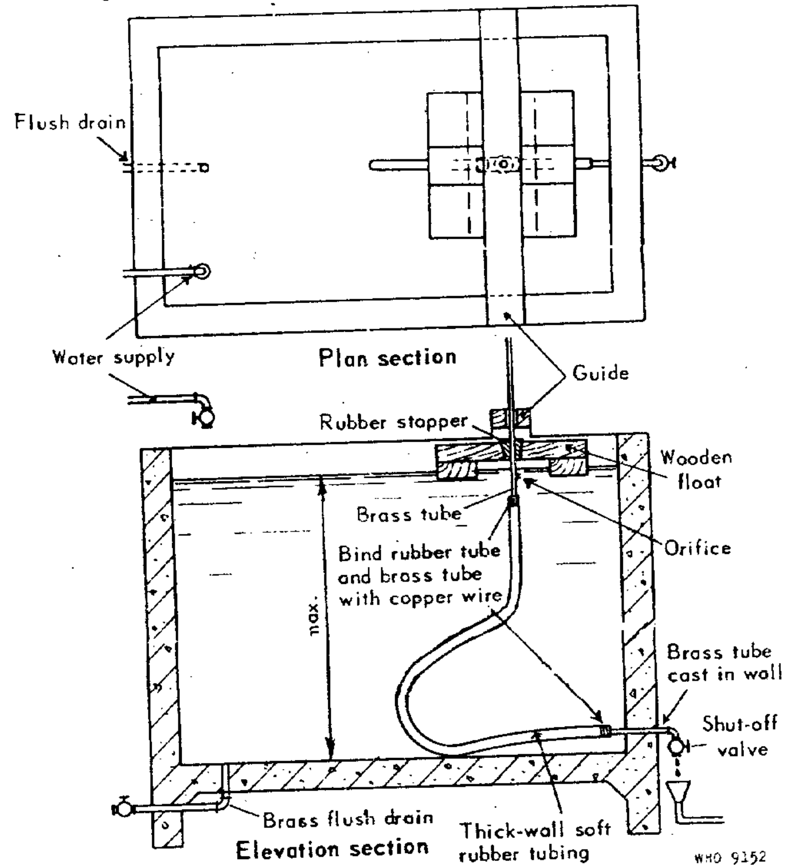
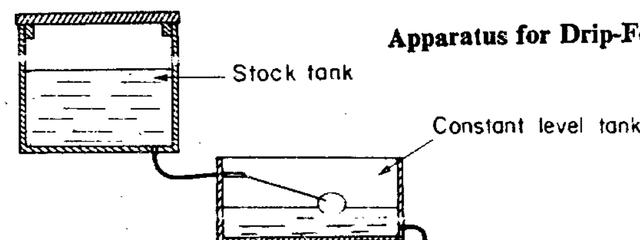
Flash Mixer : Quick mixing of raw water with a coagulant is done in a flash mixer and then to transfer the water to a slow mixer. A drain valve is provided to remove sludge from the bottom of flash mixer.

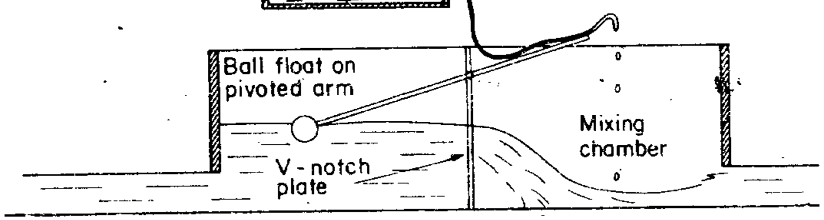






Apparatus for Drip-Feed





A simple arrangement to meter the solution feed so that it is approximately proportional to the flow of water in a channel organic matter present in the water. It is very effective against the bacteria commonly associated with water-borne diseases, and control of algae and other plant life. Chlorine eliminates tastes and odours, improves coagulation, oxidises iron and manganese and removes colour.

Chlorine content in water 23 per cent is supposed to be sufficient to kill all typhoid, cholera and gastroenteritis germs. But flue, colds and hepatitis germs are killed only by boiling water.

Chlorine may be given in three forms ; (*a*) For very small supplies, as bleaching powder or chloride of lime. (*b*) For small town or village installations, as liquid or sodium hypochlorite solution containing 10 to 15 per cent by weight of chlorine. The liquid form can be readily mixed with water. (*c*) For medium and large size water-works, as chlorine gas by gaseous chlorinator. Chlorine is easiest to apply in the form of a solution. The chlorine may be added to the water in the pipe leading from the filtered water impounding reservoir to the distribution mains or in the clear well (made for the purpose) so that an adequate contact time will be ensured. Where the raw water is highly polluted it would be advantageous to add some chlorine into the suction pipes of raw water pumps or to the water as it enters the mixing chamber, before any other treatment is given.

This *pre-chlorination* reduces the bacterial load, improves coagulation, reduces tastes, odours, algae and other organisms and keeps the filter sand cleaner, and also prevents decomposition of previously settled sludge. The dosage should be such that a residual of 0.1 to 0.5 ppm goes to the filters ; a dose of from 5 to 10 ppm is common. Sometimes heavy chlorine doses are given with de-chlorination of the filtered water. Arrangement for pre-chlorination should be made in all water treatment plants, but used only for algae control or when B-coli count is high.

The requirement of chlorine depends upon the type of bacteria and the amount present in the water, and can be determined by the *orthotolidin* test. One millilitre of orthotolidin is dissolved in 100 millilitres of water to be tested and allowed to stand 10 minutes in a dark room. The solution will turn yellow if residual chlorine is present, the intensity depending upon the amount of chlorine present in the water. The provision of a reliable testing apparatus is very essential. The bacterial efficiency of chlorine is effected by the acidity or alkalinity of the water and its temperature, which also effects the necessary contact period. The required dose is determined by noting the *chlorine residue*, or the excess chlorine present after the period of treatment. The required dose of chlorine may vary considerably from time to time, therefore frequent testing has to be done, at least 6 times a day.

Chlorine demand of water is the difference between the amount of chlorine added and the residual chlorine present in water after the reaction is over.

Dosage of chlorine. For disinfection alone a dosage of 0.1 to 0.2 ppm may be required for good underground waters and 0.5 to 1.5 ppm for surface waters. A dosage of 1.0 ppm will destroy most of the germs.

In the case of highly polluted waters the requirement of chlorine may be as high as 3.0 ppm giving a residual of 1 ppm in 20 minutes. For town water supplies, after sterilisation there must be residual chlorine in water 0.1 to 0.2 ppm at the last tap in the distribution system after 30 minutes to ensure that all the supply has been fully disinfected, but in the case of epidemics, under rural conditions, and for small supplies, the residual chlorine is kept at 0.5 ppm. High concentration of chlorine for short periods of time is more effective than smal concentrations for longer period.

After chlorination the water must be allowed to stand for at leas 20 minutes before use (between the point of treatment and the first draw off) during which time the chlorine can act upon the water withou dilution. If such a contact period cannot be provided in the distributio pipes and/or storage reservoirs, detention basins will have to be arranged If the treated water is atonce discharged into a large reservoir, the effec of disinfection is largely lost. Where water is supplied to the consume direct from treatment works without the interposition of a servic reservoir, the contact period with chlorine may be too short. The efficac of any chlorinating process depends upon a period of contact long enoug to ensure complete sterlisation.

Chlorine gas which is most commonly used is supplied in ste cylinders compressed into liquid form (liquefied gas). Chlorine is yellowish green gas prepared commercially. Many ingenious devices ar available for applying chlorine to the water which operate either unde manual control to given fixed dose or under automatic or semi-automati control for varying the dose to the volume of water flow. The chlorin feed apparatus is known as *chlorinator* or *chloronome*. Chlorine fe through the apparatus is dissolved in a relatively small quantity of wat and the chlorine-water solution is then mixed with the main flow. Chlorin may also be applied directly as a gas to the water, but uniform control an mixture are difficult to obtain. Chlorine solutions should be kept i brown or green bottles and stored in dark places. Chlorinators shou be installed in duplicate so that service is not interrupted in case of break-down of the apparatus, and a two weeks supply of chlorine shou be at hand.

For small rural water supplies disinfection may be arranged by simple *drip-feed* device with decanted bleaching powder solution. gravity feed arrangement should be followed.

There are different types of *pressure feed chlorinators* available f injecting chlorine solution into force mains. These may be "possiti pressure-feed" type or the "vaccum" type.

Chlorine produces unpleasant acid taste and pungent odour whi vanish in course of time. If, however, the taste persists, the free chlori can be removed by adding a de-chlorinating agent. Very occasiona a small dose of 0.2 to 0.5 ppm of potassium permanganate is also add to the contact tank prior to filtration in order to control tastes.

Chlorine is a dangerous poison, particulary the gas which is hig corrosive in the presence of moisture, and dangerous to handle and breathe. Gas cylinders should be stored in a cool dry place and handled carefully. Valves should be opened slowly with the special wrench provided. Leaks can be detected if a stick dipped in liquor ammonia is held near the suspected point, a white smoke will show the point of leak. Chlorine gas is much heavier than air and runs towards the lowest levels ; therefore, in case of leaks the head should be kept high.

Plant in which gaseous chlorine is used should not be placed in such a position that an accidental escape of gas might percolate into adjacent premises. The plant should preferably be housed in a separate chamber with ample ventilation. The main door should open outwards and there should be a second door or window arranged to promote a thorough current of air and which should always be kept open. Chlorine storage at the plant should be segregated from main plant and should be properly ventilated. It is very dangerous to place any localized heat, such as an electric radiator, near the chlorine cylinder. Suitable gas masks or respirators should always be available in an easily accessible position adjacent to the plant room for use in case of an accident, or the necessasity of shutting off the supply in the event of an escape of chlorine gas.

For the medium sized and smaller plants a number of ingenious tanks have been commercially produced to combine the processes of coagulations, flocculation, and sedimentation in one unit.

Super-chlorination and Break-point chlorination

This system is adopted for the treatment of waters having a high degree of organic pollution such as swimming pool waters and waters intermixed with sewage. Heavy doses of chlorine ranging from 1 to as much as 20 ppm (3 to 7 ppm is usual) are given depending on the degree of pollution, followed after a suitable interval of time by a dose of some de-chlorinating agent, about 0.3 to 0.6 ppm, *e.g.*, sulphorous acid gas (sulphur dioxide), sodium thiosulphate or potassium permanganate to neutralize the residual chlorine where necessary. Considerable difficulty sometimes occurs in adjusting the doses of chlorine to maintain a constant residual. In some cases, after the break point residual has been obtained, ammonia gas (or ammonium sulphate) to the extent of about 1/10th of the chlorine dose is added to produce *chloramine* in order to preserve the residual chlorine effect over a wide distribution area. Where super-chlorination or break-point chlorination is practised, it is desirable to provide a baffled contact tank of approximately 30 minutes' retention, in order to ensure complete sterility.

The term "break-point" indicates the stage at which the chlorine, demand (for the destruction of organic matter) is satisfied and any further dose of chlorine reappears as free chlorine. The break-point is determined by measuring the chlorine residue obtained with increasing doses of chlorine.

Chloramine. The germicidal activity of chlorine is rapid but short lived. Chlorine being an evaporative chemical, and it is also continuously

lost by reaction with polluting substances, residual chlorine will not be available after sometime, therefore, there will be no safeguard against further pollution in the distribution mains or storage. Ammonia is added to filtered water before chlorine dose is given and this ammonia retains chlorine for much longer period and also removes unpleasant taste and the chlorine. Thus, it permits a large residual chlorine content which is more stable and without a chlorine taste or odour. This process is called chloramine process.

Ammonia is usually added in the ratio of 1 part ammonia to 3 or 4 parts of chlorine. Dose of ammonia on the whole may be 150 grams of ammonia gas or 600 grams of ammonium sulphate per million litres of water. It is used as gas or ammonium sulphate or ammonium chloride ; ammonium sulphate is more frequently used at large installations. Ammonia is thoroughly mixed with water about half a minute before chlorine is added. Chlorine is sometimes applied first in order that it may have full effect on bacteria, and the ammonia is added 15 to 20 minutes later. Ammonia and chlorine gas should never be mixed together as they may cause explosion. In the presence of ammonia the sterilising qualities of the chlorine may remain effective even up to 24 hours, and the germicidal power is considerably increased.

High degree of sterilisation should not be done for public water supplies as it proves harmful in the long run.

With chloramines the residual should be twice that with chlorine and a reaction period of up to 2 hours should be given to provide for the slower bactericidal velocity of chloramines.

Chloramine process is also used for disinfecting *swimming pool* waters to avoid irritating effects of chlorine.

Other Chemicals for Water Treatment for Small Quantities

Bleaching Powder (Hypo-chlorite of lime or chlorinated lime) is a whitish grey powder and is employed for temporary or emergency chlorination. It contains about 25 to 30 per cent of active chlorine when fresh but loses its chlorine content with age and exposure to air. The powder is commercially supplied in jars or drums. It should be stored in a dry cool place ; once the container has been opened, it deteriorates rapidly in the presence of heat or moisture. Dose varies from 1.5 to 4.0 kg per million litres of water. 5 per cent of the powder is dissolved in 95 per cent of water, thoroughly stirred, and the solution left to settle for 24 hours. The inert lime will settle at the bottom leaving the active chlorine in the clear solution. The clear solution is run into a second tank and fed into the filtered water in the required proportion. It is not used much. Bleaching powder can be painful if it gets into the eyes or an open wound. Immediate flushing with plenty of water is the most usual and probably the most effective treatment in such cases.

Iodine is a very good disinfecting agent. Tincture of iodine can be used to disinfect water ; normally 3 drops of 2 per cent tincture of iodine are sufficient for one litre of water. If the water is heavily polluted the dose may be doubled. Cloudy, muddy or water having noticeable colour is not suited for disinfection with iodine.

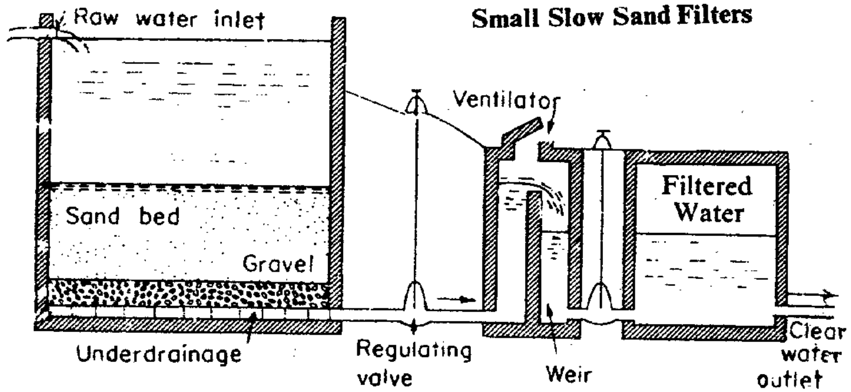
Caustic lime or quick lime. A does of about 150 to 200 mg per litre of water has a bactericidal effect. Is suitable for small quantities. Water is filtered after reaction.

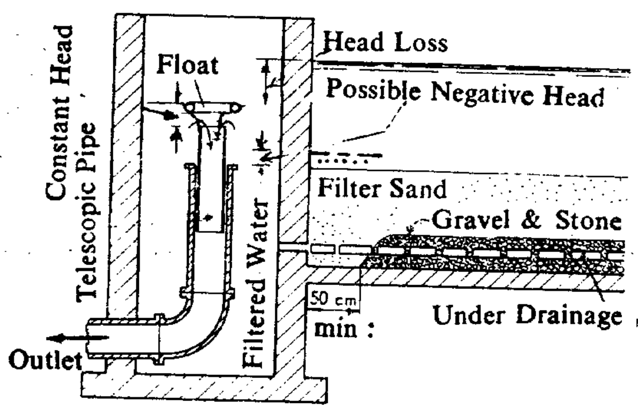
Potassium permanganate is not now considered satisfactory for water disinfection. It may possibly be effective against cholera germs but is of little use against other disease organisms.

FILTRATION OF WATER

Filtration consists of passing the water through a thick layer of sand which acts as a strainer. Suspended and colloidal matter in the water and also a large number of becteria are caught in the interstices of the sand during its passage. There are principally three types of filters ; (*a*) Slow sand filters ; (*b*) Rapid gravity filters ; and (*c*) Pressure filters

Small Slow Sand Filters





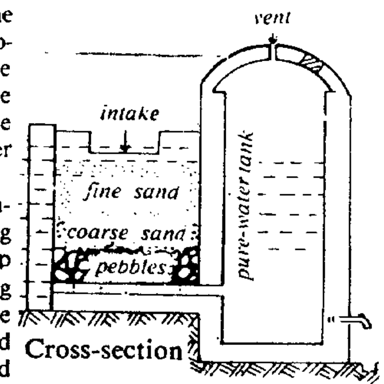
The thickness of the filter also has an important affect on filtration. It is usually between 0.5 and 1.0 m thick but should be thicker if the sand in the bed is coarse, thinner if the sand is fine. The filtering bed area is calculated from the consumption and filtration rates. . . . . If the particle radius is 0.3–0.5 mm, the bed should be approximately 0.8–1.0 m deep, the filtration rate being 0.1–0.3 m/h (i.e., 0.1–0.3 m^3 of water through 1 m^2 of filter area per hour).

When the height of the filtering well is calculated, besides the depth of the bed, a supporting layer and a distance between the sand and the top should be taken into account. The supporting layer could be 0.15–0.3 m thick. The water above the sand surface will exert pressure on the bed and help to increase the filtration rate. After a period of operation, the openings in the sand filter become blocked and the filtration rate decreases; 5 cm of sand is taken off the surface layer, washed, and replaced. If the filtration rate does not then recover, all the sand must be removed and thoroughly washed; this should be necessary only after the filter bed has been in use for a relatively long time. The bottom of the filter consists of channeled, porous blocks. There are three or four layers of sand, including the foundation layer: the top layer is fine sand (0.3–1 mm diameter), 800–1000 mm deep; the next layer down is coarse sand (1–2 mm), grains 100–150 mm deep; the next is pebbles (2–8 mm), 100–200 mm; and the bottom is stones (8–32 mm), 100–200 mm. The filter tank is 2–3 m high and the total depth of sand layers is 1.2–1.5 m. Thus, above the sand, a space 0.5–1.5 m is available for inflowing-water storage.

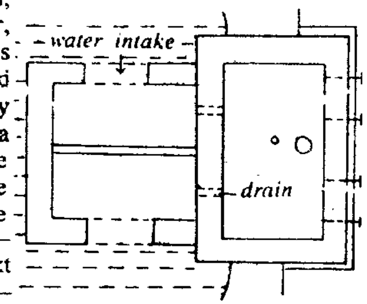
Depending on the quality of the raw water, periodic cleaning of the 100–200 mm fine sand on the filter surface is needed. The sand is withdrawn and washed about every 1–2 months. The frequency of this washing is determined by the turbidity of the raw water. When the difference between inflow and outflow (head loss) is greater than 1 m or when the treated water is unsatisfactory, washing is necessary. All the sand should be washed every 1–2 years.

Washing the sand is labour-intensive and time-consuming. Usually, 5–7 work days are required for a single washing, although this can be reduced by use of a backwash tank

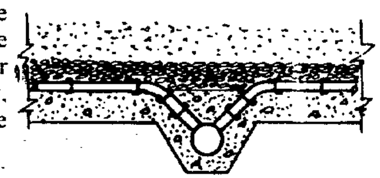
Sand-filtering Well



Cross-section



Plan



Under Drainage

Sand-filter (World Health Organization)

The household sand filter, unless skilfully operated, is relatively ineffective against bacteria. It will, however, remove cysts, ova, cercariae, and similar relatively large organisms, and will strain out most of the coarse and visible matter in suspension, although it may pass some fine turbidity or cloudiness. Sand filtration may be made more effective by first carefully treating the water with alum, as a result of which a clear water can be obtained. Some household filters also contain charcoal. Charcoal has no purifying effect, its only function being to absorb certain taste-producing compounds and to make the water " sweeter "; but even this effect is lost unless the charcoal is frequently renewed. Sand filters often get partially clogged with organic matter, and under some conditions this results in bacterial growths in the filter. Reports are common of instances where the filtered water has a higher bacterial content than the unfiltered water. Household sand filters are not recommended unless the water is to be boiled or disinfected after filtration.

With this reservation in mind, the household sand filter fills a definite place in water treatment. It can easily be made for household use wherever fine sand is to be found. The essential points in making a filter are, first, that the depth of sand through which water passes should be at least 60 cm (2 ft)—and an additional 15 cm (6 in.) is desirable—and, secondly, that the maximum rate of flow through the filter should not be greater than 3.6 litres per square metre per minute (4 gallons per square foot per hour).

A simple filter can be constructed from a steel drum 60 cm (24 in.) in diameter and 75 cm (30 in.) high, with the head cut out. Place the drum on a stand, with a container underneath, and drill a hole 2 mm ($^{3}/_{32}$ in.) in diameter in the bottom of the drum to serve as the filter outlet. Place a few centimetres of small stones, about pea-size, in the bottom of the drum and fill to within 10 cm (4 in.) of the top with rather fine sand. Make a hole in the side of the drum just below the top rim for an overflow, and to it connect a piece of pipe for an overflow line. To operate the filter, keep a continuous flow of water running into the top, just sufficient to keep the filter filled, with a slight overflow. It may be necessary to place a small disc on the surface of the sand under the inlet to avoid a hollow forming in the sand. A filter of these dimensions should deliver one litre per minute (12 gallons per hour) of clear water, suitable for chlorination.

In operating such a filter, it is desirable to keep a continuous flow through the filter at all times. The rate of filtration may fall off in time, but the filter should only be cleaned at long intervals, possibly several weeks or even months, since its efficiency depends on the biological growth on the surface of the sand. Trouble with green growths can be eliminated by covering the filter to keep it perfectly dark, since the green algae depend on light for growth. When it becomes necessary to clean the filter, a very thin layer, about 1/2 cm (1/4 in.) can be scraped off and discarded, following which the surface should be lightly raked or scratched to leave it loose. After several such cleanings, the sand should be restored to its initial level with clean sand after scraping the surface down to a clean level.

Slow Sand Filters

A slow sand filter is an underdrained water-tight basin in which the filtering materials are placed. The size of a slow sand filter unit may be about 30 to 60 m × 15 to 30 m or more and about 2.5 m to 3.5 m deep according to the desired flow. Water after passing through the filter is collected in an outlet chamber, which is equipped with a flow regulating arrangement. The filtering material about 90 cm to 150 cm of which about 60 cm to 90 cm is fine sand, is laid on top of the under-drainage system in five or six layers in progressively smaller sizes towards the top. The sand is supported on two or three layers of graded gravel, with the finest layer immediately below the sand and the coarsest material at the the bottom of the filter, packed around the drains. The gravel layers must be graded sufficient to prevent the material from mixing and the sand being drawn down. Most of these filters are open gravity filters.

The following thicknesses may be taken for the filtering materials from the bottom towards the top :

10 cm to 15 cm of broken stone, brick-bats or gravel 40 mm to 65 mm size

8 cm to 15 cm of gravel 20 mm to 40 mm size } graded
5 cm to 10 cm of gravel 3 mm to 6 mm size } graded
15 cm of coarse sand } graded

60 cm to 90 cm of fairly uniform fine sand

Alternatively :

20 cm to 25 cm thick layer of broken stone or gravel 3 mm to 6 mm size, laid over the under-drainage system followed by a 15 cm thick layer of coarse sand over which comes a layer of fine sand about 90 mm thick.

Filter sand should be fairly uniform. Much variation in the size of individual grains will affect the efficiency of the filter ; more uniform the sand grains better the result. Effective size of sand grains should be as near to 0.3 mm size as available, varing from 0.25 to 0.40, with uniformity co-efficient to 2.0 to 3.0 (Terms "Effective Size" and "Uniformity Co-efficient" have been explained in Section-8, Glossary of Terms.) The filter sand should have sharp grains and be as nearly as possible of pure silica or quartz, and must be free from limestone. All the filtering materials should be thoroughly washed and cleaned of all impurities before stacking in the filter basin.

To collect water from the filter several simple methods are adopted for the under-drainage system. A central main drain is laid lengthwise into which series of cross drains pour. Under-drains are laid with open joints to permit the entrance of water into them, Branch drains may be laid not more than 3 to 3.5 metres apart and laterals 150 to 180 cm, surrounded by a layer of coarse gravel or brick-bats upon which the layer of graded gravel rests. The central drain can also be covered with open jointed slabs or bricks, with about 25 mm gaps in-between, and again covered with bricks laid flat with about 12 mm gaps in-between. Alternatively, bricks are laid on the floor under the gravel in transverse rows in two or three layers at right angles to the one below so as to form a series of channels. Water percolates through the gravel into the channels. The central drains proposed to be covered with slabs may be made semi-oval in shape about 30 cm in size. Slabs made of "no-fines" concrete can also be used which will hold the gravel and allow the water to pass down to the collecting channels.

Drains may also be of loose jointed pipes of stoneware or half-rounded tiles or hollow blocks and of size 20 cm or 15 cm for the branch drains and 8 cm or 5 cm for the laterals. The pipes may be laid to a gradient of 1 in 100. Pipes may have perforations 6 to 12 mm.

The spacing of under-drains depends upon the size and the rate of filtration. Roughly, for a filtration rate of 120 litres per hour per sq. metre of filtering media, a 5 cm pipe drain will drain about 7 sq. metres and a 8 cm pipe drain about 16 sq. metres of filter area. The floor of the filtering chamber may be of cement concrete and given a slope from the end walls to the central main collecting drains which is fitted at a lower level of the cross-drains, and given slope towards the filtered water chamber. The walls to be cement plastered on the inner faces. The filtering chambers may not have roofs in hot climates. A covered chamber complicates the periodical washing of the filter sand. The filter chamber should be made in four units, plus one extra, two working at a time. No plant should have less than two, preferably three, filter units ; and should have sides in the proportion of 2 to 3.

For operating a filter the depth of water on the surface of sand has to be adjusted which will provide for the head necessary to overcome the frictional resistance of the sand and the thickness of the biological film or scum formed on its top, desired rate of filtration and the turbidity of the raw water. Too great or too small heads do not work well. A depth of water equal to the thickness of the filtering sand or about 2/3 to 4/5 of its thickness has been found to give good results in common cases. Slow sand filters are not generally worked under a filtration head of more than 125 cm but it should be greater than the maximum loss of head anticipated through the sand and the biological film. The rate of flow through the filter is usually controlled at the outlet either manually or by an adjustable float controlled weir which is provided in the filtered water outlet chamber to show the difference of level of the water above the sand and in the outlet chamber, i.e., the head under which the filter is working. The filter water outlet chamber is a small compartment built between the filter and clear water tank.

The rate of filtration is generally 100 to 150 (with 200 max.) litres per hour per sq. metre of filtering media. The size of beds should be calculated accordingly (based on 4 beds and not 5—the 5th one being extra).

Minimum layer of sand should be 30 cm and for sands less than

60 cm the rate of filtration should be not more than two litres per hour per cm depth of sand.

In a newly cleaned filter flow is rapid but as the filter works, the biological film formed at the top becomes thicker, frictional resistance is increased and the flow is reduced. As a rule, slowly the filtration takes place, the purer will be the filtrate. *Filtering head* is the difference between the water level over the sand bed and the filtered water level in the outlet chamber. This difference of water levels is about 10 cm to 15 cm in a freshly cleaned filter which gradually increases as the filter gets clogged. *Loss of head* through the filter sand is the frictional resistance of the sand to the passage of water which is made up of two parts : the frictional resistance of the sand itself, and the resistance due to the accumulation of dirt on the surface of the filter. It is affected by sand size and uniformity etc.

When the resistance in the filter (due to sand and clogging) i.e., loss of head, is equal to the total depth of water on the filter, the operation will stop. The loss of head should not be greater than the depth of the filtering sand ; when it becomes excessive and before a negative head is formed the filter should be cleaned. The level of the filtered water at the outlet chamber should not be below the level of the surface of the filter sand.

Negative Head. When a filter is clean, there is small loss of head in the filtering media. Due to filtration, clogging occurs in the top sand layer and friction losses increase greatly therein. When the loss in the top layer becomes greater than the head of water above the sand bed (due to choking), and the level of the water at the outlet is below the level of the surface of the filter sand the column of water below acts as a draft tube, resulting in partial vacuum. This condition is known as "negative head" and when excessive, allows air to escape from solution in the water and lodge in the sand. This interferes considerably with the filtration, and if this formation cannot be avoided, vertical air pipes are provided through the sand. If arrangements can be made to refill the beds with filtered water from below (after cleaning or emptying for any purpose) this will prevent entrapping of air in the sand.

Filtration is performed by the thin biological film (which is a layer of slime or gelatinous substance) formed on the top of the filter sand, which coats the sand grains sometime after the water is let on it which arrests suspended matter and most of the bacteria. After some flow when the filter gets clogged, about 5 cm top sand is removed by scrapping off, washed with clean water and reused. Provision should be made for drawing off the raw water from the top of the filter to waste when it needs cleaning. When a filter is to be cleaned the water should not be drawn down in the sand to a level more than 30 cm below the surface of the sand. Filters must be operated continuously at all times otherwise they will provide a favourable breeding place for bacteria and flies and make the filter water worst. About 12 per cent of filtered water is required for washing ; provision has to be made for its storage.

The filter should be filled with filtered clean water from below, if possible, for at least 10 cm above the sand before unfiltered water is admitted for the first time : when the normal level above the sand is reached the filter should be allowed to stand and settle for at least 12 hours. Until the biological film of sediment has formed it is unsafe to use the water for supply and the filtrate should be run to waste for at least 12 to 15 hours at 1/5th of the normal filtration rate. The water may then be turned into filtered water reservoir, but only at reduced rate of say 1/3rd normal, which can be increased to the normal maximum rate in the course of the next 3 or 4 days. After a filter has been cleaned or when a new filter is installed, sand acts as a mere strainer till the biological film has been formed.

Cleaned or resanded filters should be brought up gradually to the full filtering rate and maintained, as far as possible, at a constant rate until the head reaches the maximum. The loss of head while filtering ranges from 60 cm or less at the beginning of a run to the allowable limit which may be about 120 cm. Loss of head is low in the beginning of the filter run and mounts up gradually. Cleaning is done usually once a month or after a longer period as required. All materials should be cleaned and washed or changed at least once every six years.

The finer the filter sand, the more efficient is its filtering action and bacteria removal property, but the quicker it will clog greater will be the head necessary to pass water through the bed. Fine sands which would be rejected as unsuitable in cold countries are successful in India on account of the higher water temperatures.

A break-pressure chamber of small size is built on the inside corner of each filter compartment into which the raw water enters at a low level and is gently introduced on to the filter surface by means of one or more pipes discharging vertically upwards which are fixed flush with the top of the bed. A valve is fixed on the raw water pipe outside each compartment for control of the flow. Where several filters are operated in parallel, the flow through each must be regulated to distribute the load. Generally one spare bed for every four or five in use will be required.

Slow sand filters are quite efficient and cheap if raw water load as regards colour, bacteria, algae and turbidity is low. Colour removal efficiency is about 50 per cent, bacterial removal efficiency is about 95 per cent ; typhoid and cholera germs which are the the deadliest may not be removed through these filters. Raw waters with over 100 ppm turbidity cannot be handled by slow sand filters. Maximum permissible turbidity is 50 ppm which is reduced to 5 ppm. Turbidity over 40 ppm may produce unsatisfactory effluent ; average turbidity should be less than 30 ppm for economical and continuous operation. Usually the water receives no preliminary treatment other than plain sedimentation except when the raw water is very turbid. Turbid waters should be passed at rapid rates through preliminary or rough filters before passing on to a slow sand filter, no coagulants are generally used.

Slow sand filters are useful as final process after rapid sand filtration of very polluted waters. They have simpler mechanism, need less supervision and less running cost than rapid sand filters. There is less

lossof head, are very suitable for small towns where land is available but initial cost is high. Slow sand filters, however, are less commonly used than rapid sand filters.

Activated Charcoal (carbon) is highly porous and possesses a high power of absorption but has no purifying effect, its only function being to absorb certain taste-producing compounds whether due to decaying organic matter or chlorine, and to make the water sweeter. It can be used in granular form as a filter bed, or more generally as a fine powder, instead of sand.

Colloidal and dissolved organic or inorganic matter, iron. manganese, and dissolved gases are retained by it. It has also good colour removal properties, and is usually employed before sedimentation where required. It reduces chlorine demand of treated water, and helps coagulation if added before filtration. Activated carbon is a proprietary product and is sold under varions trade names.

Rapid Sand Filters or Mechanical Filters

A rapid sand filter is a water treatment plant that includes provision for pre-treating the water by coagulation and sedimentation for settlement of suspended solids before filtration, with sterilisation as the final process.

Advantages : Initial cost less ; high rate of filtration and small space required, with arrangements for cleaning the sand without removing it from the filter ; greater efficiency with highly turbid or coloured waters.

Disadvantages : Maintenance more expensive ; expert knowledge for handling machinery required ; cleaning difficult which has to be done very frequently ; not so efficient for bacteria removel as a slow-sand filter.

There are two main types of rapid filters : the *gravity* type and the *pressure* or hydraulic type. Rapid filters are now replacing slow sand filters, especially for capacities over 4.5 million litres per day as the slow sand filters require an area about 40 to 60 times of that required by rapid filters of the same capacity.

Rapid Gravity Filters are usually open tank filters in which the difference in level between the water in the tank and the filtered water channel or sump is available for forcing the water through the filter. This type of filter is used for most large installations. Gravity filters are considered to give better results than the pressure filters. Each unit is about 6 to 9 metres long and 3.5 to 6 metres wide and 2.5 to 3.5 metres deep with water depth of 1.5 metres or more over the sand bed, and are made of concrete or masonry. The filter bed consists of 55 cm to 60 cm of graded gravel in shallow layers in sizes ranging from 40 mm in the bottom layer to 3 mm size in the top layer. Above the gravel is a bed of sand 60 cm to 75 cm thick.

For high rate filters, an effective size of sand not less than 0.5 mm and a uniformity co-efficient not greater than 1.5 is recommended for Indian conditions. A rapid filter requires a coarser and more uniform sand than used in slow sand filters. The fine and coarse layers of sand

and gravel should be so placed uniformly that the sand does not enter into and mix up with gravel during the back-wash, nor enter into the under-drain system. General arrangements are as for slow sand filters. A filter depth should be designed for a maximum loss of head of 2 metres without causing negative back pressure. Various types of under-drains are provided. The general method of their operation is as follows :—

Raw water is dosed 5 to 25 grams per 1000 litres depending on the turbidity of the water. Average dose is 15 grams with a coagulant, generally alum, and allowed to settle for a period varying between 4 to 6 hours, after which it is admitted to the filters which are usually worked at the rate of about 100 litres per min. per sq. metre of the surface area of sand (equivalent to over 50 times the rate admissible with slow-sand filters). The filtration head in rapid filters varies from 1.5 to 3.5 metres. Rapid filters are usually found more satisfactory in India, where the necessary skilled supervision can be given. Gravity filters are cleaned by filtered water, pumped or otherwise forced upwards through the sand, called back-washing.

The wash-water is generally stored in an elevated tank at height of 9 to 12 metres. The wash-water required varies from about 3 to 4 per cent of the water filtered, depending on the frequency of the washings. Washing is done after 2 to 3 days. Wash-water is applied for about 5 minutes. Before washing, shut off the filter and lower the water level to the level of the sand. After the filter has been washed, open inlet valve only about 1/3rd to 1/2 for about 12 to 15 minutes till the filter starts working ; only clear water should be allowed into the outlet chamber.

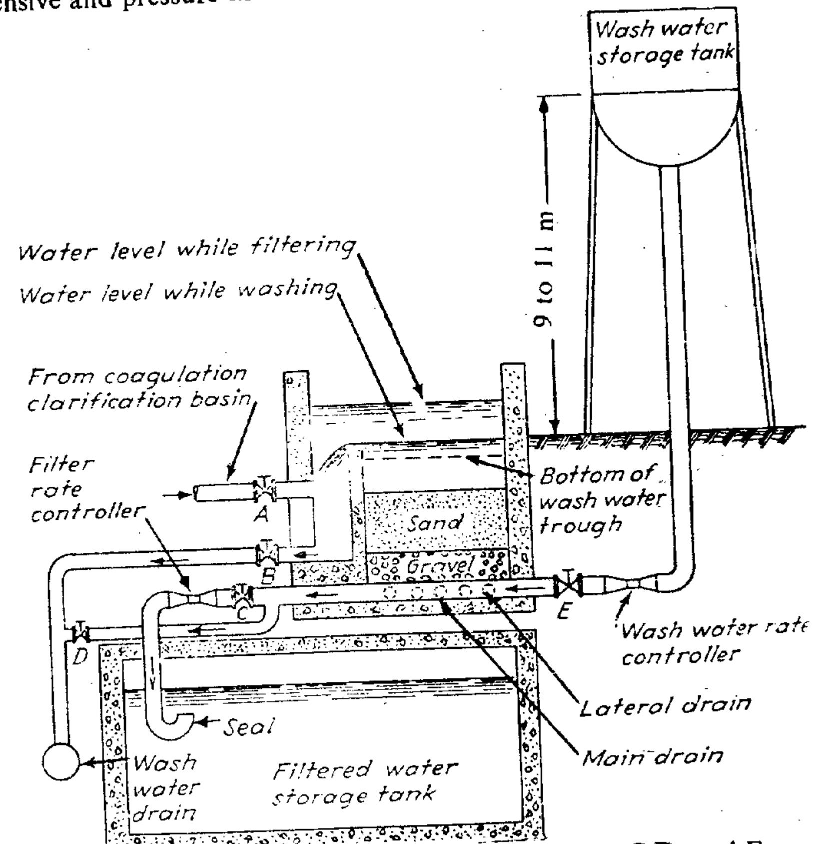
Pressure Filters. A pressure filter (also called hydraulic type filter) is a type of rapid sand filter enclosed in a water-tight container, about 1.5 to 3 metres in diameter and through which water passes under pressure. Sand bed is 45 to 60 cm in thickness with effective size and uniformity co-efficients as recommended for Rapid Gravity Filters. It may be located between the pumps drawing water from the source of supply and the filtered water storage reservoir. The water is generally given a small dose of coagulant before it reaches the filter. Pressure filters are not considered reliable for the removal of bacteria, therefore they are not generally used for municipal water supplies when the water is considered contaminated. These filters are generally used for small projects for filtering softened waters at industrial plants ; for filtering swimming pool waters ; for treating water for re-circulating ; and for the softening of and removal of iron from ground waters. Rate of filtration is 6,000 to 12,000 litres per hour per sq. metre of filter area. Details regarding these filters can be obtained from the manufacing firms for the particular requirements. A minimum of two filters should be provided.

Choice between Gravity and Pressure Filters

When the sequence of treatment involves settling, pressure filters are seldom used, because all but the smallest settling tanks are open and the open gravity filter follows logically. There are exceptions to this,

particularly when small plants are involved and, in such cases, the settled water may be pumped through pressure filters. The reason for using pressure filters is then based on the ease of assembly on site and simplicity of operation.

If settling tanks are not necessary, the question becomes one of economics. Where the plant can be installed on the hydraulic gradient of the system, gravity filters can be used, but this is sometimes difficult and expensive and pressure filters are then justified.



Diagrammatic section of a rapid sand gravity filter. A,B,C,D, and E are valves. Valve D permits waste of filtered water. The seal in the effluent pipe is necessary to keep the pipe full at all times so that the rate controller will function properly

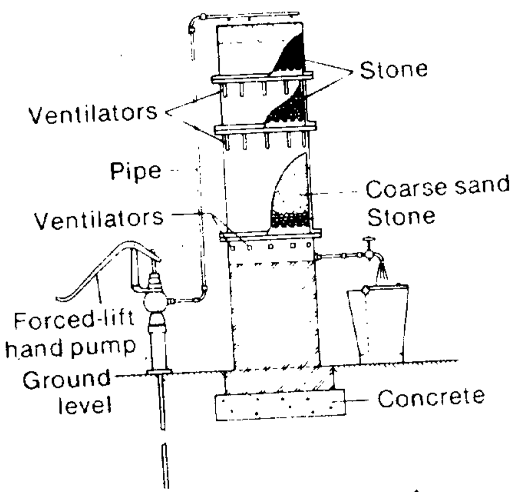
Aeration of Water

Aeration is the process of mixing extra air with water and has a number of useful functions and helps in the improvement of chemical and the physical characteristics of water. Aeration removes carbon dioxide and other objectionable gases and the water obsorbs more oxygen from the air which helps in the reduction of organic matter and oxidation of certain dissolved mineral compounds. Aeration is not adopted as a general practice but only when it is specially needed for iron and manganese removal. Cost involved and the head lost are weighed with the advantage of aeration. Aeration is often done at an early stage of water treatment for removing excess iron from ground waters, and odours and tastes from surface waters. Corrosiveness is also removed to some extent, while iron and magnesium are removed to a considerable extent. Aeration of water is a very simple process and can be effected in a number of ways.

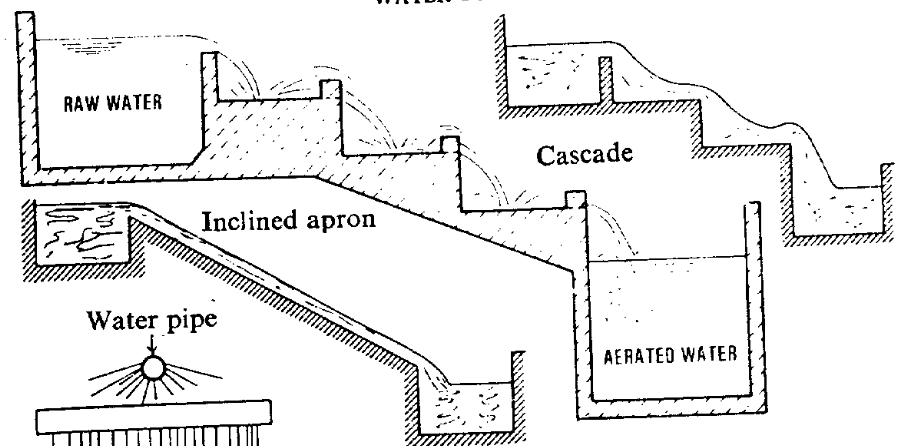
Cascades. Water is let to flow down a series of 3 or 4 inclined steps or weirs, or one long weir, over which the water tumbles in a thin sheet, discharging as a fountain into an open reservoir or a settling tank. This requires a head of 1.5 to 6 metres.

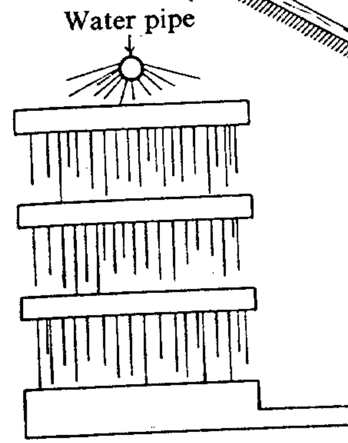
Allowing the water to fall from a pipe from a good height into a collecting basin will bring sufficient aeration. Passing the water through open channels is also effective.

Iron Removal Unit



Hand-operated unit for Iron and Manganese removal has been developed by National Environmental Engineering Research Institute, Nagpur, for rural works. The unit consists of four cylinders placed on top of each other as shown in the illustration. A 150 mm layer of assorted 20-50 mm stones is placed in each of the top two cylinders. In the third layer a 50 mm deep layer of 10-20 mm stones supports a 300 mm layer of coarse sand. The gravel becomes coated with oxides of iron which help in the further oxidation of iron and manganese. Water is collected in the bottom cylinder and is withdrawn through a 12 mm tap into a bucket.

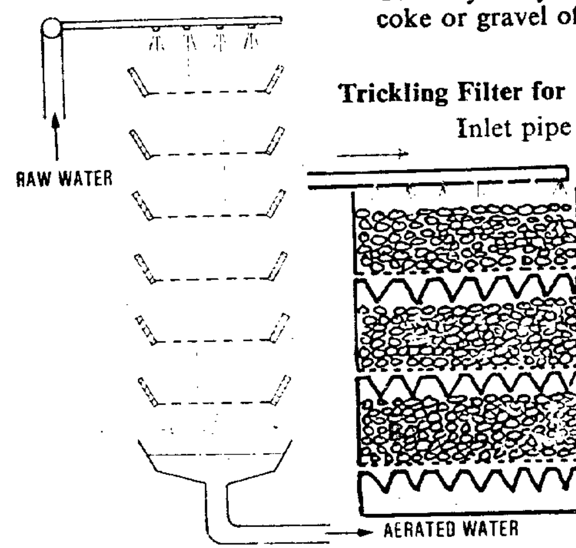




Tray Aerators. In these water falls through a series of trays perforated with small holes, 5-12 mm in diameter at 25-75 mm centres. The area of the trays required varies between 0.015 and 0.045 sq. m/cu. m of water passing through each hour. Tray aerators are often built in stacks of four to six trays giving a total height of 1.2 to 3 m. The trays may be filled with layers of coke or gravel of about 50 mm size.

Trickling Filter for Aeration & Iron Removal

Inlet pipe with sprinklers



The approximate rate of flow through this type of filter is up to 1600 litres/hour per sq. metre area. The number of trays should be found by trial.

Spray Nozzles. Water is pumped through pressure nozzles (through numerous small jets) to spray in the open air, as in a fountain. (A nozzle of 2.5 cm of orifice will discharge about 300 litres per minute to a height of about 215 cm at a 4.5 kg pressure at the throat.) A very good dispersion is obtained when the water is revolved by guides as it approaches the jet.

Compressed air is blown through diffuser porous plates or perforated pipes at the bottom of settling tanks and small air bubbles pass upwards through the water.

Trickling beds. Water is passed through perforated pipes over beds of coke, slag, or stone about 60 cm thick. These may be contact filters or pressure filters.

An iron removal plant has been designed by "Central Public Health Research Institute, Nagpur."

3. STORAGE OF WATER

Pure Water Storage Reservoirs

By storing water the quality may be impaired by the growth of microscopic organisms which produce tastes, odour and colour. Ground waters and treated waters stored in open reservoirs are more susceptible to such growths than natural surface waters. The quality of stored waters may be improved through sedimentations, and through the beneficial effect of oxidation and sunlight, but the danger from algae growths is so great that purified waters ought to be stored in the dark. Microscopic organisms, with a few exceptions, are dependent upon light for their growth. Therefore, all reservoirs built for filtered water should invariably be covered, and ventilators with wire gauze covers should be built into the roof or just under the roof above water level.

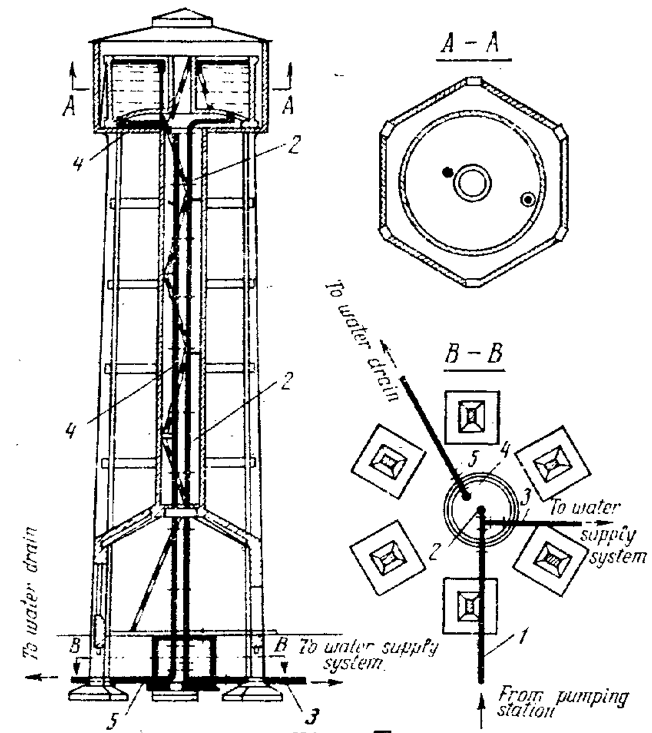
A storage reservoir is built to collect water from filters and store it until it is pumped into the service reservoir or the distribution system. It usually holds from 1 to 3 days or more of the average daily demand to meet fluctuations. One day's storage reservoir is better from sanitary point of view. Where water is obtained from several sources it may be only half day's supply.

The term *service reservoir* is used for a tank which is made for the storage of pure water after treatment before distribution to the consumer, and it provides for fluctuations in the demand. The term *impounding reservoir* is generally used for a tank which stores untreated surplus rainwater for use in dry seasons

Usual economic depths of ground reservoirs :

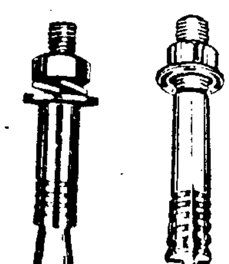
Capacity (million litres)	2	5	7	9	15	23	36	45
Economic depth in metres	3.5	4.0	5.0	5.5	6.0	6.5	7.0	7.5

Such reservoirs should preferably be built half in and half out of ground, the material of excavation being used for banking material for the walls and covering over roof slab to keep the water cool. Losses due to evaporation and absorption, etc., should be added to the capacity of the tank. This has been explained in Section 17. For design of masonry reservoirs see Section 7.



Water Tower

1-Inlet from Pumping Station; 2-Riser Pipe; 3-Outlet; 4, 5-Overflow Pipe



Concrete Anchor Bolts
← for Foundations

Service or Distributing Reservoirs are built to provide storage to meet the fluctuating demands and to serve as balancing tanks to maintain the pressure during the hours of greatest demand. They meet emergent demands due to fire and failure of pumps and also reduce the hours of pumping. Storage is necessary if the hourly rate of peak demand exceeds the safe pumping rate. The capacity of a service reservoir depens upon a number of factors (and the optimum capacity is debatable) such as variation between maximum and minimum demands, pumping capacity and the hours the pumps are designed to work and the stand-by arrangements. Where the pumps are designed to work for 8 to 10 hours a day during peak hours and they meet the peak demand in full, the storage capacity of the reservoir should be 14 to 16 hours average daily demand. Where the pumps work for about 16 hours at a uniform rate (at lesser rate than the peak demand rate), the minimum storage should be 6 to 8 hours average demand. These are only rough estimations. (See under "Distribution System.")

Where a service reservoir is provided pumps and filters can be of smaller size with capacities equal to the average rate of demand instead of maximum rate of demand. Distribution reservoir is connected direct to the distribution system and built in a position which is ralatively central to the max: consumption area and which has a tendency towards the pumping station rather than away from it so as to keep the friction in the pumping mains to a minimum. (Also see under "pressure-equalizing reservoir.")

ELEVATED TANKS OR WATER TOWERS

Overhead reservoir are termed "Water Towers"

Design of Tanks. Design of water tanks has been discussed in detail in Section 8—"Reinforced Cement Concrete".

Economical Forms of Reservoirs. For a single-compartment seservoir of a given depth the most economical form in materials is in the order—circular, elliptical, square and rectangular. For a reservoir to be sub-divided into two compartment by a mid-wall the greatest economy will result when the breadth is approximately equal to two-thirds of the total length.

For small circular tanks a height equal to or somewhat greater than the diameter is an economical ratio. For larger capacities the tank should be relatively large and shallow.

Where the tank is fixed on columns, bending moment due to wind with tank empty should be considered and each column must be well anchored to the foundations with a strength of anchorage equal to the maximum uplifts due to wind acting on empty tank. Min : size of anchor bolts is 6 mm dia. Steel rod cross stays should be provided inside steel tanks to hold the opposite sides together.

The cost of water towers is balanced against the cost of boosting, i.e., the capital cost of the tank and its maintenance against the capital cost of the (boosting) pumping plant and its running cost and maintenance. Ground storage tanks with booster pumps are less costly than elevated

tanks but are not as reliable as high up reservoir which provide gravity flow.

It would be more convenient to have a number of small reservoirs keeping in view the pumping and mains expenses, the town being divided into a number of districts or zones according to the levels and the population. Each zone should have its own service reservoir and distribution system but all should be interconnected if possible. (See below).

Level of Service Reservoirs. An ideal would be where the supply of the topmost floor of the highest building in the zone is assured by gravity alone, and in the case of fire jet from the hydrant commands all the parts of the highest roof. Allowing for the friction losses etc. in the water mains the reservoir should be high enough to supply water under the pressures as detailed in "Supply Pressure." If practicable, all the service reservoirs in the system should be built at the same level so that one may feed into the other in case of a breakdown.

Whenever it is possible to avoid no distribution mains or connections should be taken directly from the rising main and all water should be pumped in the first instance to the reservoir direct. Where it is essential to build the reservoir at the opposite end of the town from the pumping plant as mentioned below, and where for direct pumping a long rising main would have to be laid parallel to a service main with consequent duplication of costs, a combined rising and distribution main may be provided. In a position like this a reciprocating type of pump should be fixed and not centrifugal.

A pressure equalizing reservoir placed on the distribution system at a distance from the pumping station serves its purpose with better effect than one placed close to the pumping station, and which should preferably be situated on the opposite side of the highest consumption district from the pumping station. During peak hours, the district will be fed from both sides, a condition that will reduce the loss of head in the water mains to about one-quarter what it would be without elevated storage. The inlet should be so arranged, and provided with a float operated switch, so that the pumps stop working and inlet closes when the water in the reservoir reaches the top water level. An over-flow pipe should also be provided which may consist of vertical pipe with a bell mouth, placed a few centimetres above the top water level and connected to the scour pipe below the sluice valve which controls the sluice valve.

Break Pressure Tanks in Hills. Provide with capacity of about 2,000 litres, fitted with a ball-valve at the inlet, and the outlet should be at least 75 cm below the inlet.

All reservoirs, except the very small ones, should be divided into two or more compartments with vertical walls, and valves and piping should be arranged in such a manner so that either compartment can be used independently ; a sluice valve should be fitted on the outlet pipe from each section. All drinking water tanks should be covered and ventilated. Small cisterns upto 150 litres capacity should be provided with hand holes 15 cm dia. and, the bigger sizes with man-holes.

Fittings for Tanks. (See under "Cisterns" in this Section). All tanks, reservoirs or cisterns need an inlet, outlet, overflow and wash-out pipes. The main supply pipe or inlet is connected at the top and the outlet or down service pipe connected on the opposite side of the inlet pipe 5 cm to 15 cm (lesser height for smaller tanks) above the floor of the tank so as not to take any sediments with it which might have collected at the bottom. The outlet pipe should be fitted with a bell-mouth and strainer the aggregate area of which should be at least double the area of the inlet pipe. A stop tap or stop-valve should be provided on the supply pipe outside the tank. A *wash-out pipe* (also called *draining pipe* or *scour pipe*), min : 40 mm dia., with a stop-valve or a socket and a plug for cleaning and emptying the tank must be provided and made flush with the tank bottom at its lowest point. A waste pipe known as an overflow or warning pipe carrying off the surplus water when being over-filled (in case of the ball-valve getting out of order) must always be provided, and should be fixed to the tank a little below the level of the inlet and should discharge through the external wall. The overflow pipe should be of larger size than the inlet pipe and of not less than 25 mm dia. A back-flap should be fitted to the end of the pipe outside the building to prevent the entry of insects and birds ; or fitted with mosquito proof coupling with small perforations. The over-flow pipe can also be connected with the wash-out pipe. A float volve is used to control the water level in the tank. A water level indicator need also be provided.

Painting Water Tanks. Galvanized iron or lead lined tanks should not be used for drinking water. Steel tanks should be painted with leadless paint or a quick-drying bitumen paint with non-inflammable base, specially manufactured for the interior of water tanks. (Also see at page 15/50

Storage of Water in Buildings

Provision is required to be made for storage of water for one or more of the following reasons : (i) To provide against interruptions of the supply caused by repairs to mains, etc ; (ii) To tide over periods of intermittent supply.

Type of Building		Storage in Litres
Dwelling house	per resident	70
Hostels	"	90
Hotels	"	135
Commercial buildings		
without canteens	per head	35
with canteens	"	45
Restaurants	per meal	7
Day schools	per head	25
Boarding schools	per resident	90
Nurses homes and medical quarters	"	135

If the supply is intermittent and the hours of supply are irregular, it is desirable to have a minimum storage of half a day's supply for overhead tanks. The capacities recommended for various types of occupancies are given below (IS : 2065).

Type of building	Storage required
Tenments having common conveniences	900 litres per WC seat
Residencies ditto.	270 litres for one WC and 180 litres for each additional seat in the same flat
Factories and workshops	900 litres per WC and 180 litres per urinal
Cinemas, public assembly halls, etc.	900 litres per WC and 350 litres per urinal

In case the of multi-storey buildings where the height of the fittings or storage tank is such as will not permit of their being fed with the available pressures in the water mains, adequate pumping arrangements have to be made. The house service pumps are usually of the centrifugal type driven by electric motors. In cases where pumping is necessary, storage shall be provided either at the ground level or partially buried underground. This storage tank should have a minimum capacity of 50 per cent of the overhead storage tank.

Fire Storage

Multy-storeyed buildings above 25 metres height have to be provided fire storage in addition to domestic needs, adequate to fight a fire at the rate of 2250 litres/min :, as a normal fire fighting tanker cannot cope up with fires beyond an elevation of 25 metres. This limit, however, varies from place to place depending upon the normal height of ladder available with the local fire brigade service for extinguishing of fire.

The tank capacity for fire storage may be of 100 kilo litres, where the supply is intermittent so that it is adequate to fight a fire in the premises at the rate of 2250 litres/min: for about 45 minutes, The over-flow from the fire fighting tank should flow into the suction tank to maintain a continuous circulation in the static fire tank.

The fire fighting pumps may be located in the basement to have a positive suction head and designed to deliver 2250 litres/min : with a terminal pressure of 3 kg/sq. cm at the topmost floor so as to obtain the hydrant 900 litres/min : discharge with a jet of about 6 metres.

On the fire fighting rising main, hydrant tees of 60 mm may be provided at every landing of each staircase. A small 20 mm tapping may be provided at each landing with a wheel valve and adequate length of hose pipe for fighting small fires due to electric short circuiting etc.

(Manual on Water Supply and Treatment—Ministry of Works and Housing).

Flushing Storage. All flushing cisterns should be supplied from storage tanks through downtake pipes and not directly from the supply pipes. Recommended flushing storage for different classes of buildings :

Rectangular Pressed Mild Steel Tanks

Minimum Nominal Thickness of Plates

Depths of tank metres	Description of Plates	Thickness of Plates mm
1.25	Bottom, sides	3.00
2.50	Bottom and first tier of sides	6.00
	Top tier of sides	5.00
3.75	Bottom and first tier of sides	6.00
	Second tier of sides	6.00
	Top tier of sides	5.00
5.00	Bottom and first tiers of sides	8.00
	Second and third tier of sides	6.00
	Top tier of sides	5.00

Pressed steel tanks are not recommended for depths greater than 5 metres.

Tanks made for hot liquids, no plate shall be of thickness less than 6 mm.

Pressed steel tanks shall be either 1.25 m, 2 50 m, 3.75 m, or 5.00 m deep. The size of tanks shall be specified as multiples of the normal dimensions of 1.25 m.

Tanks 2.5 m deep or more shall be provided with a mild steel internal access ladder.

Closed top tanks shall be provided with a manhole of diameter of not less than 45 cm if circular, or 45×45 cm if square.

If an underground storage water is to be built, it must project at least 30 cm above the highest flood level. If this is not practicable, the manhole cover should be raised to this level. Provision shall be made for the draining of the tank fully when necessary.

If large storage is required of say more than 5000 litres, it is better to provide a number of small tanks or in compartments so inter-connected that each can be isolated for cleaning and repairs without interfering with the supply. Even in small installations it is better to provide two small tanks coupled together. All the tanks should have their own independent fixtures like ball valves, over-flow and draining pipes etc. Each tank may be connected to a common pipe and distributing pipes taken from this common pipe. Stop cocks provided at suitable places.

Cisterns or Small Water Storage Tanks. Small rectangular galvanized

iron (or mild steel) tanks can be made of the following specifications :—

Capacity Litres	Depth cm	Angle Frame mm	Sheet Thickness mm (gauge)
up to 200	up to 60	35 × 35 × 5	1.50 (16)
200 to 450	,, 75	35 × 35 × 5	2.00 (14)
450 to 1000	,, 90	35 × 35 × 5	2.50 (12)
1100 to 1800	,, 120	40 × 40 × 5	2.80 (11)
1800 to 2500	,, 140	45 × 45 × 5	3.15 (10)
2700 to 3000	,, 140	50 × 50 × 6	3.55 (9)
3500 to 4500	,, 140	50 × 50 × 6	4.00 (8)

Bottom plates are 1.5 mm thicker. Each side, end, and bottom of the cistern should be in one piece. The flanged sheets (or plates) are fixed to angle iron frames by means of 6 mm diameter rivets at 40 mm pitch for small cisterns and 8 mm rivets at 35 mm pitch for bigger tanks. The top angle iron frame may have only 65 mm pitch. Tanks above 1000 litres should have 20 mm dia. galvanized wrought iron stay rods, one fixed to angle framing at top and two in the body of the tank for extra strength. Holes for riveting should be drilled and not punched. White lead should be applied to the joints before riveting Welding is better than riveting. The sheets welded to form a tank will not have angle iron frame.

Cisterns are provided with 350 to 400 mm dia. holes at the top with hinged covers. The covers can be made of galv. sheet iron with angle iron frame. The cover should be just loose but close fitting to keep out dust, but it should not be air-tight.

Fittings for Cisterns. (Also see under "Fittings for Tanks") Every cistern should be provided with rising main inlet ; 40 mm dia. galv. iron scour pipe ; 25 mm dia. galv. iron over-flow pipe (or warning pipe).

The rising main is connected to the storage cistern with a ball valve near the top, which disconnects the supply when tank is full upto a specified level, that is, to the point of overflowing. The ball valve is a simple arrangement which permits the entry of water when the cistern is empty and disconnects the supply when the cistern is full. It consists of a hollow floating ball made of copper, plastic or hard rubber, 110 mm in diameter, attached to an arm which is so pivoted that the end near the pivot closes the orifice of the main when the ball is raised to the required height of water in the cistern and opens the main as soon as the ball drops with the fall of water level as it is drawn off, through the distribution pipes.

The ball valve should be fixed to the tank independent of the inlet pipe and set in such a position that the body of the ball valve cannot submerge when the tank is full up to the water line. The ball valve should be so adjusted as to limit the level of the water in the tank to 25 mm below the lip of the over-flow pipe. Free surface should be about 15 cm above the max: water filled level.

For an average residential house the main service pipe from the tank shall not be less than 25 mm, with 20 mm branches to the baths and 15 mm branches to lavatory basins, kitchen sinks, WC cisterns, etc. The outlet pipe should be fixed 50 to 75 mm above the bottom of the tank and provided (on the inside) with copper gauze strainer. A stop cock (or stop valve or stop tap) should be provided on every outlet and inlet pipe, as near the tank as practicable, to facilitate cleaning and repairs. Where possible, it is always advisable to run dringing water supplies direct from the main service line before it enters the tank.

Mild steel tanks should be painted with a priming coat of read lead and two coats of bitumastic paints. (See under "Painting Water Tanks" at page 15/46) Water tanks should be made watertight without the use of putty.

Storage Capacity of Cisterns. The storage capacity should be calculated to cover 2 to 3 days' supply in the case of intermittent systems and equal to one day's consumption for constant services. The actual capacity of a cistern is only about 3/4th to 7/8th of the nominal capacity since space has to left above the water-line for pipe connections.

Cleaning of Water Storage Tanks. After the water has been drained off completely, the whole of the interior should be scrubbed down with bleaching powder which has been dissolved in buckets to the consistency of a thin cream.

Wooden Cisterns. Well seasoned sound timber not less than 2.5 cm thick should be used with tongued and grooved joints. External iron tie rods should be fixed to resist the outward thrust of the water. Internal tie rods (or stay rods) should be fixed in large cisterns. Wooden cisterns should be lined with lead sheets of not less than 25 kg/sq. metre weight. Tacks securing the sheets to the internal surface of the wood should also be provided for cisterns over 1.25 metres deep.

Insulation of Water Storage Cisterns. The cisterns can be surrounded with a box of wood leaving a space of about 5 cm which can be filled with some insulating material such as granulated cork, slag wool, or even saw dust.

Hot Water Cylinders and Tanks : Can be of the following thickness of iron sheets :—

Capacity in Litres	100 200	200 to 400	400 to 500	500 to 1000
Thickness in mm	2.0	3.0	3.5	5.0

4. DISTRIBUTION OF WATER

Consumption or Demand of Water. Rate of water demand depends upon many factors and local conditions such as : the size of community (i.e., population) to whom water is to be supplied—the bigger the community the greater the supply ; the economic conditions and standard of living of the community to be served ; pressure under which water will be supplied consumption increases with increased pressure) ; water

charges ; whether the supply will be metered ; if sewage system will be provided ; if provision is to be made for watering of gardens and lawns. The water consumed varies widely in different houses from day to day and month to month. Maximum demand is during mornings and evenings; hours of peak demand are 6 to 9 in the mornings and 5 to 8 in the evenings. It is considered that half the daily demand of 24 hours is consumed in 6 hours and the balance half in the remaining 18 hours.

The maximum hourly demand may be from 2 to 3 times (or even more) the average hourly demand and which may be 20 to 50 per cent more during summer months—but all the services are not used at the same time. The maximum seasonal (summer) demand may be 1.3 to 1.5 times the average annual demand ; there is less seasonal variation on coastal towns in India than in the Northern towns due to extreme climates Then, the smaller the population the greater the percentage of total flow that may be drawn in a short time.

Metering all services reduce the consumption to about half of the consumption without meters. Although the value of metering for reducing excessive use is fully recognized, the capital charge of installing meters and the operating charges for their maintenance and reading and billing have proved deterrent to the expansion of domestic meter use. Metering reduces the water pressure available at the outlet on account of frictional loss in the meter.

Fire Demand. It is estimated that only about one per cent of the total water supply is actually used to fight fires, but a substantial provision has to be made The greater the population the greater the possibility for outbreak of fires. Fire demand has usually to be met from storage,

Future developments and increase of population should be contemplated for 30 years hence. Experience shows that consumption of water keeps on increasing gradually by 10 to 25 per cent independently of any increase in population and consumption per head also increases substantially as improved housing and working conditions are provided. The chief reasons for increased consumption are the requirements of industry and agriculture, rising standards of personal hygiene, together with an increase in population. An important additional item is "wastage".

Density of Population for Design of Water Supply

Densities vary widely within a town, general range being from 50 to 200 persons per hectare in the sparsely built-up areas. In four to six storied buildings with apartments like those in Calcutta and Bombay, with wide roads, the population may be 500 to 750 persons per hectare. In the closely built-up residential areas in Old Delhi, density of population has been reported to be 2200 to 2700 persons per hectare.

Loss Through Leakage and Wastage

A very important factor to be considered while estimating the total demand is the loss due to leakage and wastage. A metered system reduces the wastage to a considerable extent but still the aggregate amount of leakage, apart from misuse, may be from 10 to 25 per cent of the total output in a hundred per cent metered and well maintained water supply system. An increase of pressure also increases the wastage and leakage which is approximately in proportion to the square root of the pressure. Where the pressures are high and efficient methods of waste prevention are not adopted, the wastage may be as much as 50 per cent. A system consistently losing 25 to 30 per cent of its water will have about 9 to 10 per cent revenue loss.

Leakage in Water Mains. There is always some leakage in cast iron pipes with lead joints due to expansion and contraction from temperature changes. Settlements also cause movement in the joints. Old and badly caulked joints will have more leakage. There are leaks in every distribution system, no matter how well it is laid. The amount of water wasted from a water main per day under a head of 30 metres may be 600 litres from a 1 mm hole, 1,600 litres from a 1.5 mm hole, 10,000 litres from a 3 mm, and 40,000 litres from a 6 mm hole. A dripping house tap may waste as much as 100 to 200 litres per day and a tap, not under heavy pressure, running full 5,000 to 10,000 litres per day. Increase of pressure increases the leakage approximately in proportion to the square root of the pressure.

The most serious leakages are those which take place underground and which may continue for months or years without giving any surface indications. *A leakage allowance of 4,000 to 6,000 litres per 24 hours' flow per kilometre per 100 mm dia. of pipe may be taken for estimating,* depending upon lengths of pipes used, pressure and other factors.

The following table gives approximately leakage for a length of 1,000 metres of pipes line, in litres per hour with a pressure of 7 kg/sq. cm:

Dia. of pipe in inches	3	4	6	8	10	12	16	20	24	30	36
Dia. of pipe in mm	80	100	150	200	250	300	400	500	600	750	900
Leakage in litres	130	180	240	330	390	440	600	750	900	1100	1400

When estimating water requirements and the corresponding peak demands, there is usually less harm in over-estimating than in under-estimating, because in the former case the capital expenditure does not increase in direct proportion to the supply catered for, whereas in the latter case it is often extremely difficult to rectify the position except at considerable extra expense.

Tracing Leakage. It is rather difficult to detect leakage in the underground mains. A metal rod having a sharp-pointed end is generally thrust into the ground along the pipe line and withdrawn to find out whether or not its point is wet. Sound of escaping water can be determined by placing the ear against the rod. Waste-water meters are also used for detecting leakage. The meter is placed in a small chamber at the head of a supply zone and the supply is passed through it at night. This meter registers flows at all hours on a drum with square paper. An instrument

somewhat like a doctor's stethoscope is sometimes used. During the hours of least consumption at night, valves are closed in turn at the principal branches. Stem of the stethoscope is placed against the spindle of the valve, flow through the valve is indicated by a sizzling sound. Other instruments are also available called—pitometer, equaphone, sonoscope. Where chemicals are employed : one mg of fluoresein is sufficient to colour 1.5 tonnes of water ; one mg of indigo dissolved in sulphuric acid and mixed with supply, will colour 15 kg of water.

Water Supply for Residences. The requirements regarding water supply, drainage and sanitation for residences shall assume that minimum supply of 135 litres (about 30 gallons) per head per day is assured, together with a full flushing system (IS : 1172). Of this 135 litres, about 40 litres is for WC flushing system. Provision of 180 litres per head is not much on the whole. This quantity is considered where water meters are installed in residences. Total requirements of a town inclusive of provision for industrial and public purposes may be about double of this.

The following rates per head per day are considered as minimum for domestic and non-domestic needs (Manual on Water Supply published by Ministry of Works and Housing, New Nelhi.)

(i) For communities with population up to 10,000 — 70 to 100 litres
(ii) Ditto. with population 10,000 to 50,000 — 100 to 125 litres
(iii) Ditto. with population above 50,000 — 125 to 200 litres

Where the entire supply of a town is given through street "standposts", and where wells exist in addition, 45 litres per head per day may be taken. One standpost for each 150 persons should be provided and at distances not more than 200 metres apart. Pilgrim population may be provided at 25 litres per head per day.

Water Requirement of Buildings other than Residences (IS : 1172)

Type of Building	Consumption/per head/day Litres
1. Factories where bath room are provided	45
2. Ditto. no bath rooms ditto.	30
3. Hospitals per head	
(a) No. of beds not exceeding 100	340
(b) No. of beds exceeding 100	455
4. Hotels (per bed)	180
5. Offices	45
6. Restaurants (per seat)	70
7. Cinemas, theatres, (per seat)	15
8. Schools :	
(a) Day schools	45
(b) Boarding schools	135

Water Supply Requirements for Railway Platforms and Bus Stations (IS : 1172)

(a) Junction stations where bathing facilities are provided—70 litres/capita ; and where no bathing facilities are provided—45 litres/capita.

(b) Terminus stations—45 litres/capita, with or without bathing facilities.

(c) Intermediate stations—45 litres/capita where bathing facilities are provided and 23 litres/capita where no bathing facilities are provided.

The numbers of persons shall be determined by the average number of passengers handled by the station daily. Staff and vendors likely to use the facilities should be added.

Public purposes :

Road watering	... ...	140 litres per 100 sq. metres of road surface
Sewer flushing	... ...	4 5 litres per head

Irrigation

Road-side trees	...	28,000 litres per km
Public parks and private gardens	...	20 to 30 litres per 100 sq. metres

Unfiltered water supply, where available, can be taken for the above public purposes through separate mains and hydrants.

In addition to the above allowance has also to be made for animals, public standposts, bathing tanks, swimming pools, dhobi ghats, and industries, etc. It is estimated that each cattle requires 45 to 65 litres per day. For washing cars, 135 litres per washing per car per week is taken (i.e., 20 litres per day per car).

Roughly, for industrial purposes it will be 20 to 25 per cent and for public purposes 10 per cent of the total demand.

Distribution Methods. In a water supply scheme the distribution system is the most expensive item in respect of both initial cost and maintenance, and which may be 60 to 80 per cent or even 90 per cent for small schemes, of the total cost of a project. Therefore, the advisability of service reservoirs, or water towers, or the provision of '*booster pumps*' for separate pressure zones, should receive careful and detailed consideration. Where surface levels vary appreciably, the system of separate zoning at different pressures should be adopted. It is preferable to fix 'booster pumps' in high levels for boosting water rather than to have more pressure in all the mains. Booster pumps do away with provision of water towers at high levels. A booster can be so arranged which will come into operation whenever the pressure falls below a predetermined limit.

Boosting Water to Increase Pressure or Supply in a Distribution System. Where it is found necessary to increase pressure in part of a distribution system high lift centrifugal pumps (called booster pumps) can be installed to augment the pressure and supply. A booster pump is designed for the extra head of water required plus losses due to friction ;

pressure can be increased up to about 50 per cent. (It happens very often that pressure in a distribution system becomes deficient due to increased demand or extra demand at high level places than previously anticipated and laying of new pipes becomes necessary. In such cases, booster pumps are very helpful.) Booster pumps are best located on the track of the main on a by-pass arrangement, with valves on its both sides of the by-pass, rather than at pumping stations.

In addition to sluice valves to control a booster station, non-return or reflux valves are provided between the suction and delivery points (of the direct main) and also on the booster main (return by-pass) to prevent damage to the main or pump by water hammer.

There are three general systems of water distribution by pipes : (*i*) Gravity; (*ii*) Direct pumping ; (*iii*) Combination of the both (*i*) and (*ii*).

(*i*) **Gravity Distribution** is where the source of supply is above the town level so that sufficient pressure is available at all times in the mains. This is the most reliable method. Gravity conduits are laid on nearly uniform slope following the hydraulic gradient of the line while pressure conduits can be constructed at any elevation below the hydraulic gradient and may be laid even a little above the hydraulic gradient for short distances in the form of syphons.

(*ii*) **Direct Pumping Without Storage.** This system is used in cities where the ground is flat, with it there is no distributing reservoir but water is pumped directly into the mains at all times as needed. The rate of pumping has to be varied according to the demand. Pumps of varying capacities should be installed so that the varying demands can be met by operating one or more of the pumps in combination. The capacity of the pumps must be at least 3 times the average rate of consumption, and in small works 4 or 5 times as great. Duplicate sets should be provided with one-third and two-third capacities. Stand-by pumps must be provided for use in case of failure of one or more units. Stand-by engines should preferaly be steam or diesel driven units mointained for emergency in case of electric failure. Direct pumping is the least reliable system and should be used only for small supplies.

(*iii*) **Distribution by Pumps and Elevated Storage Tanks called Combined System.** The water from pumps enters and leaves the bottom of the storage tank through a common pipe whereby the distribution system can draw from the tank when the pumps are not operating. When the pumps are operating, the distribution mains take as much as is needed and the excess water is discharged into the tank and stored to be drawn upon during the periods of high consumption to augment the water pumped. With this system pumps can all be of the same size and rating which facilitates maintenance as spare parts are all of the same size and thus result in economy. Installation should be capable of meeting the pumping requirements with at least one unit out of service. This method allows uniform rate of distribution and pumping and meets demands during emergencies and break-downs, therefore, is fairly reliable.

Design of a Distribution System

A detailed survey of the town including the longitudinal strip of land lying between the area of distribution and the supply intakes is required with all levels accurately done from which contour maps can be prepared. A topographical map is drawn showing all street intersections, high and low lying areas, densities of population, location and arrangements of the pipe lines, required discharge or draw-off at each connection, and other relevant structures, etc. Scale for the map may be 1/2,500 to 1/5,000 according to the area.

Size of Distribution Pipes. There is no direct method of determining the size of pipes for a distribution system. The determination of water pressure losses in a distribution system consisting of network of pipes of various diameters permitting flow by many routes is complicated, and pipe sizes are often assumed for the desired maximum rate of flow expected at any time, and loss of head due to friction is worked out and the residual head arrived at. Diameter of a pipe can be fixed by assuming velocity as recommended earlier (60 to 120 cm/sec. and gradient 1 to 3 per 1000) and with the help of formulae, tables and "chart" given for flow in pipes. In working out a project it is necessary to start at the consumer's end (where the pressure required is fixed) and work backwards. Analyse the whole system for the pressure losses at all important points. Various methods have been devised for analysing pressures in a distribution system but they are all complicated and usually do not give accurate solutions. The following simple methods may be taken as a guide :

The most difficult points to supply are—which are at a distance from the reservoir or pump and are above the level of the hydraulic mean gradient.

When pipes are connected at each end to a large main, 50 to 80 mm pipes may be up to 150 metres length and 100 mm pipes up to 600 metres length, but on dead ends lengths may be limited up to 100 metres and 400 metres respectively. This is where no fire hydrants are fixed on the lines. Where, however, fire hydrants are fixed, no pipe should be smaller than 150 mm dia. and which may be only 100 mm up to 180 metres length if its each end is connected to a large main. Probable maximum demand for each district or a particular area of distribution is calculated and the size of the main which can deliver the estimated quantity is fixed. Bends and fittings are converted into straight lengths of pipes.

In the case of the ring system of distribution, in practice it is sufficient to assume that the maximum loss of head in the ring main from the point where it is fed to the neutral point, is that due to a flow equal to half the peak flow fed to the ring, passing through a length of pipe equal to one-half of the circumference of the ring.

The distribution system is divided into sections by drawing lines at right angles to the main feeders and the maximum demand estimated for each section. The required drafts ore assumed to take place at a few concentrated points. A simplified plot of the network is made by eliminating all small pipes except where they are important connecting links. Usually 150 mm and smaller pipes can be eliminated in medium-sized systems, and much larger pipes in larger systems. If the simplified grid system includes two or more parallel mains serving the areas in question, these can be converted to an equivalent single pipe. If the area is served by a single line, the capacity of the pipe and pressure drops can be taken from the appended tables direct.

Pipes must be laid at hydraulic mean gradient to maintain a given flow in a pipe of given cross-section.

Systems of Lay-outs of Pipe Lines

There are four systems of laying the distribution pipes :

(*i*) Dead-end, Free-end, or Tree system ; (*ii*) Grid-iron or Reticulation system ; (*iii*) Circular or Ring system ; and (*iv*) Radial system. The first two systems are generally used for small town supplies.

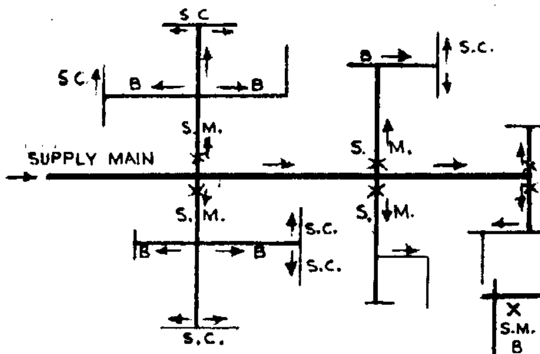
Dead-end System. In this system a central trunk main is laid which takes its supply from the service reservoir and to which are connected a number of sub-mains or branches in both directions, and which have no inter-connections except at the beginning, thus making dead ends. The trunk main is gradually reduced in size. For want of circulation, the water at the dead ends remains unused which stagnates and may contaminate the whole supply. Scour valves have to be installed at each dead end for periodical washing. Supply has to be stopped when a pipe breaks down in any section. This system is suitable for old towns that have developed in an haphazard manner and there is no regular lay-out of streets, and where there is a central road with by-lanes. This is a simple and cheap method but far from ideal.

Grid-iron System. Pipes are laid in the shape of net work resembling a grid-iron, connected at intervals at street intersections and there are no dead ends. Water is supplied from more than one direction and which circulates continuously throughout the system. Repairs can be done without cutting off the total supply as water would be available from all directions. Large quantity of water is available for fire. The system is suitable for regularly planned towns of rectangular lay-outs and where roads have no dead ends. This system requires a large number of valves, longer pipe lengths and of bigger diameter.

Radial System. This system is suitable for towns with radial system of roads. Centrally laid separate reservoirs are provided for each district or zone and the supply pipes are laid radially taking off from the central reservoirs and joining with a sub-main laid around the boundry of the district.

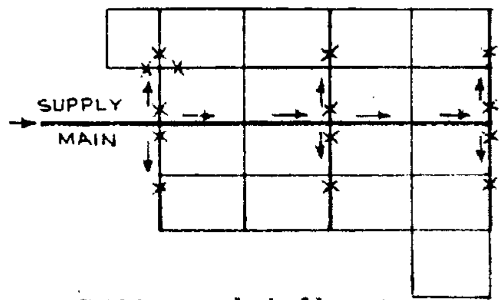
Ring System. The supply main is laid along the periphery of the area of the distribuion district and branches are connected cross-wise to the main and also to each other. This gives good pressure both for general supply and fire in addition to the advantages mentioned for grid-iron system. This system is suitable only for well planned towns with systematic lay-out of roads.

The type of lay-out system of pipes to be adopted depends upon the

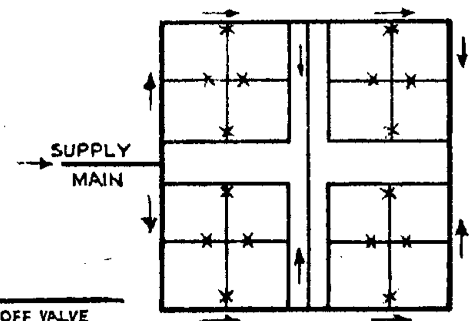


Dead-end method of layout

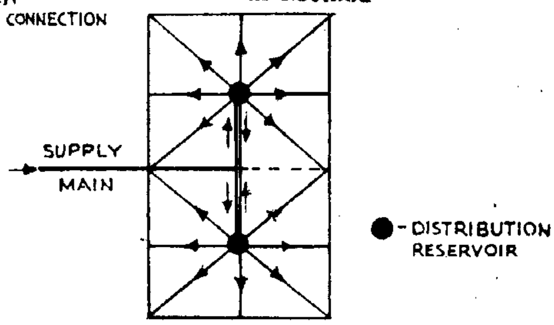
X CUT-OFF VALVE
S.M. SUB-MAIN
B BRANCH
S.C. SERVICE CONNECTION



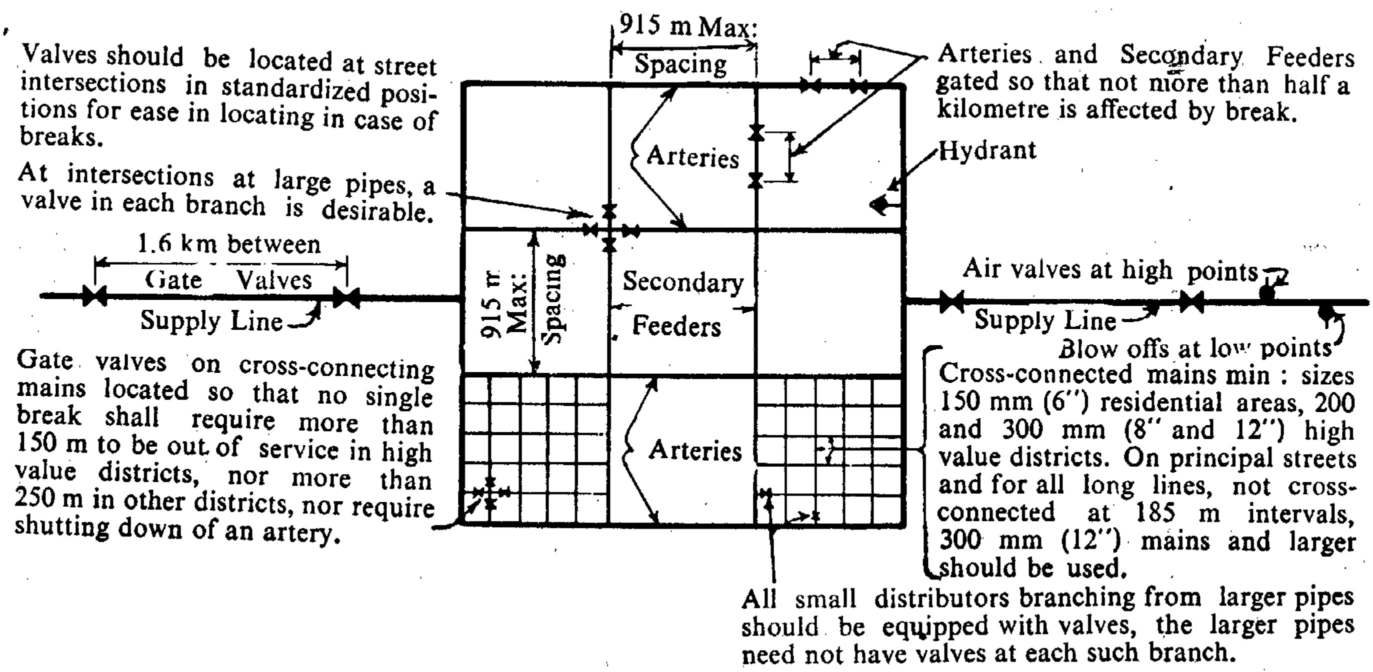
Grid-iron method of layout



Circular Method



Radial Method



topography of the town, and the location as well as the elevation of the source of supply. Each method of lay-out mentioned above has its own advantages and disadvantages. Single system of lay-out is perfectly satisfactory and a combination of two or more systems has generally to be adopted according to the lay-out of the town. It has been stated earlier that the town should be divided into high and low level zones or districts. These level zones should be further divided into separate supply blocks consisting of about 2000 to 3000 houses each in built-up areas. The lay-out of the system should be so designed as to have the large sub-mains passing through portions of the greatest demand and connected to the supply main in as direct a route as practicable.

In long narrow districts, the most economical plan is to run a trunk main through the centre of the district, of gradually decreasing size according to the reduced demand with small branch mains taking off from it. In wide roads where supply will be required on both sides, it is better to lay duplicate service mains, one on each side of the road, and mains and branches interconnected as far as possible so as to avoid dead ends.

Where the supply districts are more or less square and the streets are laid at right angles, the mains should either pass through the centre of or laid round the periphery of the distribution districts. As far as possible, large mains should be provided at intervals of from 1 to 1.5 kilometres apart and cross connected and the areas filled in with smaller pipes so as to form a grid or network of pipes. No district should depend upon a single main, and supply to any point should be ensured from at least two directions, and circulation of mains should be aimed at so that water is available from all directions.

As far as practicable, the underground service pipe should be laid at right angles to the main and in approximately straight lines to facilitate location for repairs. A stop-valve should be provided in the service pipe in an accessible position just outside the building, so that the supply may be readily shut off in case of trouble and for repairs. Where the service pipe is of less than 50 mm bore, all the stop-valves should be of the screw-down type.

Rules for fixing of valves have been explained under "Valves, Meters and Taps," in the pages following.

Types of Supply—*Intermittent and Constant Systems* :

A supply is called constant when it is supplied for 24 hours. When the supply is limited to certain hours of the day, it is termed intermittent service.

When supply is stopped all the water rushes to the low level mains from the high levels creating vacuum. Foul gases enter through taps which are left open and also filthy foreign matters find their way into the mains through leaky joints and hydrants etc., and consequently make the water unhealthy. In the case of a constant supply the danger of drinking water becoming contaminated is lessened owing to a direct supply from the main and the avoidance of large storage cisterns. Also the pipes,

constantly under pressure, are less liable to deteriorate, while with the intermittent supply, a vacuum may be created causing a strain on the pipe joints which may lead to gradual leakage. It is also found in practice that a water main or pipe which is always filled with water has a much longer life than the charged intermittently with water and air. For fire fighting service, a constant supply is much more helpful.

It would appear that there is less wastage with the intermittent supply and waste (due to leaks and where taps are left open) during the remaining period is prevented, but the position is otherwise. Water is stored in the houses which is all thrown away as soon as fresh supply is resumed, thus giving an enormous amount of wastage.

In the intermittent system, the sizes of pipes needed for the mains and the distribution must necessarily be greater as the day's requirements are concentrated to a period of 6 to 8 hours and the supply needed per hour may be four times the average. However, the advantage of the intermittent system is that, in those towns where the supply is scanty and the head available is poor, the town can be divided into zones and the water allowed in each zone at different hours of the day.

A 24 hours supply should be preferred and introduced where practicable.

Supply Mains

Mains fall into three groups according to the primary purposes which they serve, viz., trunk, secondary and service mains. A "trunk main" is usually an important pipe which carries water from one place, e.g., a pumping station or reservoir, to another main or to a district where the water will be used. As far as possible, a trunk main should not be used for supplying to the consumers direct, connections should be restricted to secondary and service mains. Service mains are used to supply premises in the streets in which they are laid, and their carrying capacity need be no larger than is necessary to meet the very local demands plus any anticipated future extra connections due to developments. Secondary mains form the links between trunk and service mains and can be used to supply the large consumers direct.

Size of Supply Mains. Mains should be capable of a rate of flow sufficient to satisfy the combined maximum demand of all the services to be supplied and should be sized liberally to provide for possible reduction in the carrying capacity due to incrustations that might occur in course of time. It is always difficult to determine how far it is desirable to allow a larger size of pipe on account of the incrustation. In some cases pipes corrod rapidly, in others they may remain permanently as smooth as they were first laid. A common rule (for pipes subject to incrustations) is to calculate the size of the pipe required when clean and add 250 mm to the theoretical diameter, and then to supply the commercial size which is next larger to that dimension. The carriage capacity of the new pipes should, however, be 30 to 45 per cent more than the estimated capacity.

Possible increase of water consumption due to any unforeseen demands or population expansion have also to kept in view. But unnecessary incaease in the size of mains will increase the cost of the project considerably. All these maximum demands of the separate services may not occur simultaneously. Therefore, it is generally recommended that the carriage capacity of the trunk mains should be made three times the average hourly demand for less than 5,000 population, 2.5 times for populations of 5,000 to 50,000 and which may be twice only for populations above 50,000 (subject to the rules given in the last para). Service pipes and feeders may be made twice to 1.5 times the average consumption. This will meet the normal peak demands. Average demand must include all the anticipated demands discussed above.

Recommended Velocities for Water Mains

Pipe dia.	Velocity in m/sec.	Pipe dia.	Velocity in m/sec.
100 mm	0.75 to 0.91	500 mm	1.40
150 ,,	0.81 to 1.22	650 ,,	1.69
200 ,,	0.91 to 1.32	800 ,,	1.92
250 ,,	0.99 to 1.52	900 ,,	1.95
320 ,,	1.09 to 1.67	1000 ,,	2.00
400 ..	1.27 to 1.83	Service pipes	2.13

Velocities in water mains should not exceed 3 metres/sec., but velocities should be high enough to prevent deposits of silt in the pipes and should not be less than 0.61 to 0.75 metres/sec. Ordinarily velocities in excess of 1.5 metres/sec., or friction losses of more than 3 in one thousand are uneconomical in water supply practice. Some engineers recommend velocity of flow to be not more than 1.8 meters/sec. in main pipes and not more than 0.91 metres/sec. in distribution pipes. With high velocities in small mains loss of head by friction is excessive. "Head" gives pressure and pressure gives velocity. Friction of water in pipes increases as the square of the velocity.

Pipe diameter can be fixed by assuming a velocity from the above table and with the help of formulae and flow tables given in the pages following.

Minimum size of cast iron water mains (trunks) is 100 mm (4") for mains fed from one side and 80 mm for mains fed from both ends. No branch feeders or main distribution pipes should be less than 50 mm (2") diameter.

Storage arrangements and *pumps* may also be designed for the same capacities as the mains, which may be only for 1.5 times the average demand where pumps work for 24 hours. Sedimentation tanks and filters are generally designed for average (prefer 1.5 times) rate of demand, which should include all the anticipated demands discussed earlier with extra units as described under "filters".

Lay-out of Supply Mains. Trunk mains should be so planned as to avoid main streets and points of traffic congestion, and should generally be laid on one side of roads. As far as practicable all important mains should be laid in duplicate, or which can be fed from more than one side,

so as to avoid the risk of supply being totally cut-off in case of a burst or repairs.

Two-Main System. Two-main distribution pipes are provided at some places instead of single-main pipe. Twin parallel lines are safer than a single line and should be used for under-water crossings or in other places of special danger or where settlement is likely. A bigger main, say of 150 mm, is laid on one side of the street and a smaller main of 50 or 80 mm on the other side of the street. These two mains are connected to each other at the ends of every block. This system will require shorter lengths of service pipes and is more convenient for giving service connections of consumers which is through the smaller main, and also for repairs. But unless there are particular hazards single large diameter pipe, made secure against possible failure, is more economical than two smaller lines.

The pipe-lines should follow in general the profile of the ground and should be laid well below the hydraulic grade line as far as practicable; rising of the pipe-line above the hydraulic grade line at any point would create siphonage and a negative pressure in the pipe. No section of the pipe-line should be more than 6 metres (max: 7.6 metres) above the hydraulic gradient at that section, as otherwise vacuum will be formed and flow will cease. Where a water main goes above the hydraulic grade line by more than 6 metres, pumping will have to be resorted to, to keep up the pressure. High pressures at steep slopes can be avoided by breaking the hydraulic grade line with over-flows or auxiliary reservoirs or by installing relief valves.

All mains should be divided into sections by the provision of sluice-valves (explained under "Valves, Meters and Taps"). Air relief valves should be provided at all summits, and "washouts" or "blow-off" valves at low points between submits, unless adequate provision is made for the discharge of air and water by the presence of service connections and fire hydrants. 'Dead-ends' to mains should be avoided if possible, and service mains should be arranged in 'ring' formation or interconnected at every 300 metres in the form of a network. A ring main is not necessarily a complete circuit, it is more than a series of cross connections but a complete circle is the ideal. Where it is not possible to avoid a dead end, frequent flushing out will be necessary for which a washout or a fire hydrant should be provided. Wash-outs should not discharge directly into a drain or sewer, or into a manhole or chamber directly connected thereto; an effectively trapped chamber must be interposed into which the washout should discharge through a flap-valve.

Mains should be laid with a cover, measured from the top of the pipe to the surface of the ground of not less than one metre under road-ways, and not less than 76 cm (min : 61 cm) elsewhere. Mains should not be laid in ground liable to subsidence, but where such ground cannot be avoided, special precautions should be taken to minimize damage to the piping.

Service Connections from Mains

The pipe extending from the street distribution main to the house or consumer's meter is known as the "*service pipe*" ; that portion of it lying within the consumer's premises is termed the "*supply pipe*".

It is often necessary to take a service connection off a main. To do this without stopping the flow, special equipment is used. The operation consists in drilling and tapping a hole in the main, and afterwards screwing in a brass ferrule within a watertight box ; the service pipe is attached to the ferrule. Whenever possible mains should be tapped at the side and not at the top. The curvature of the pipe restricts the size of the drilled connection that can be taken out of it and which should not exceed one-third the diameter of the main. Over 50 mm holes are not generally tapped into the mains to avoid weakening them. The effect of the holes drilled for ferrules in weakening the main can be reduced by drilling the holes at 45-deg. to the vertical. The maximum size of ferrules which can be used safely is limited to 15 mm with 80 mm mains, 20 mm with 100 mm mains, and 25 mm with 150 mm mains. The ferrule is 3 mm to 6 mm less in diameter than the pipe and the pipe can then be stepped to increase the discharge. Screw-down ferrules are used.

For connections up to 25 mm a solid drill is used and for larger holes, a hollow cup drill is employed. When larger services are required a tee has to be introduced into the run of the main.

The ferrule should be so set in the main that the service pipe leads off in line with the main before curving round right-handed into its proper course, it allows for any settlement of the pipe, which will then tend to tighten rather than loosen the ferrule in the main. A short length of (say about 45 cm) lead pipe or flexible copper tubing, called "goose-neck", is used to make a flexible connection between the service pipe and the ferrule cock to reduce the possibility of the service pipe and ferrule joint breaking through settlement of service pipe. The connecting pipe and service pipe should be given gradual offset and a support of brickwork near the ferrule. Wiped lead joints are used for connections.

For making connections in asbestos cement mains, a special saddle with a flat boss is first bolted round the main with a rubber washer between the main and the boss. The boss and the pipe are then drilled and tapped together, giving a continuous thread through the boss and the wall of the main.

A stop-cock is generally inserted on the service pipe between the ferrule and the house service in a cast iron box with cover. This serves as a shut-off for the building.

Service pipes of 50 mm bore and upward shall be connected to special T-branches which have to be inserted into the line of the main. Special branch pipes shall also be used for service pipes of less than 50 mm bore where the bore of the main is not greater than thrice that of the service pipe.

Cutting cast iron and stoneware pipes. A line is marked around the pipe where it is to be cut. With a chisel held radially on the line, strokes of moderate strength are given by means of a hammer in quick successions

moving the chisel each time a little further round the line. This must be done very carefully, and heavy blows avoided.

For cutting cast iron pipes, use of a pipe cutting machine is recommended, or the electric-arc cutting method may be permitted using a carbon or steel rod. The flame cutting of a pipe by means of an oxyacetylene torch should not be allowed.

House Services For an average house with about 15 inhabitants the service main should be of 25 mm diameter with 20 mm and 15 mm branches. For smaller houses no domestic service pipe should be less than 20 mm diameter except when the pressure is very high, it may be 15 mm diameter. For unfiltered water minimum size is 25 mm.

Supply Pressure. All domestic supply taps should have residual head of 1.5 metres minimum (and which may be 0.61 metre minimum at all cisterns) ; but a residual head of 6 to 7.5 metres should be aimed at to ensure ample supply under good pressure. Low pressure carries with it germs of diseases to the house taps.

Avoid pressures above 70 metres of water head in the trunk mains. Where houses are not more than 2 storeys high a maximum pressure of 50 metres of head may be adopted.

Delivery from the taps should be aimed at as follows :

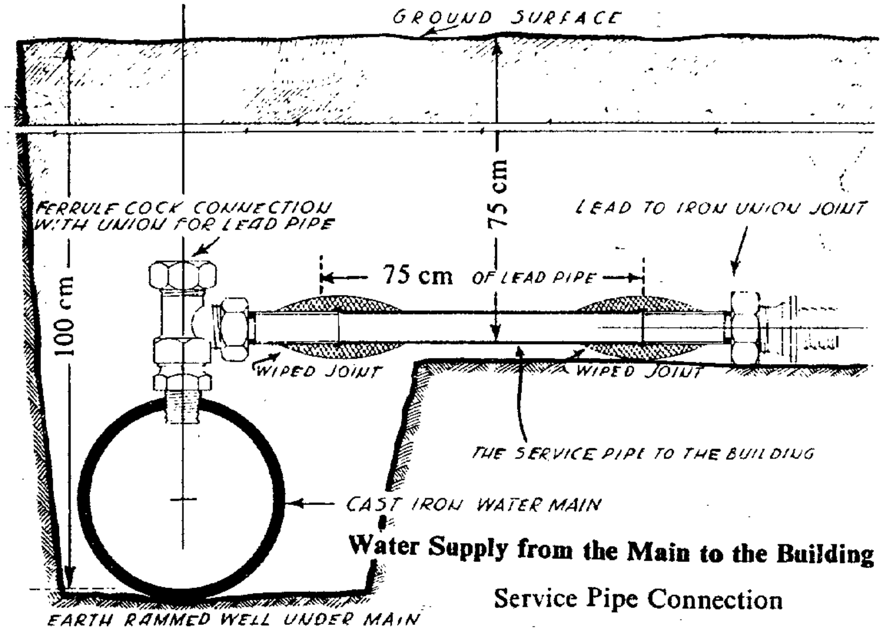
Dia. of pipe in mm	15	20	25	32	40	50
Flow in litres/minute	10	20	30	45	70	110

Recommended Size of Service Pipes in mm for House Connections

Class of Building	Length of service pipe main to house point in metres			
	30	15	7.5	3
Ordinary single family dwelling 2 to 3 storeys and not more than 10 rooms.	32	25	25	20
Larger dwelling with about 16 rooms.	40	32	32	25
Four apartment building with about 24 rooms.	50	40	40	40
Twenty five apartment building with about 100 rooms.	50	50	40	40

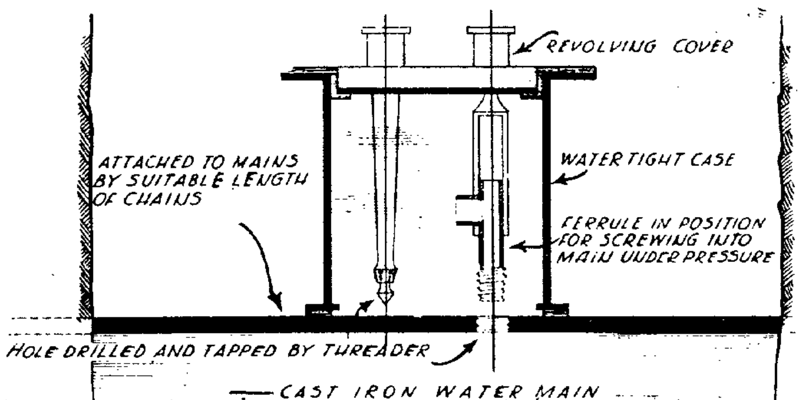
Where it is necessary for a pipe to pass through a wall or a floor, a sleeve shall be fixed therein through which the pipe is passed.

Service pipes generally are of galvanized wrought iron. Lead pipes are more expensive and heavy although very lasting ; they are liable to

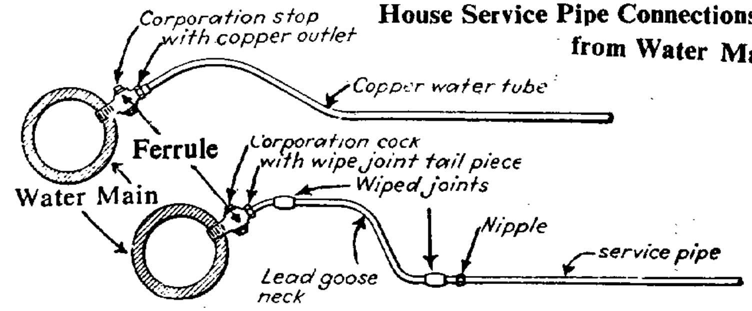


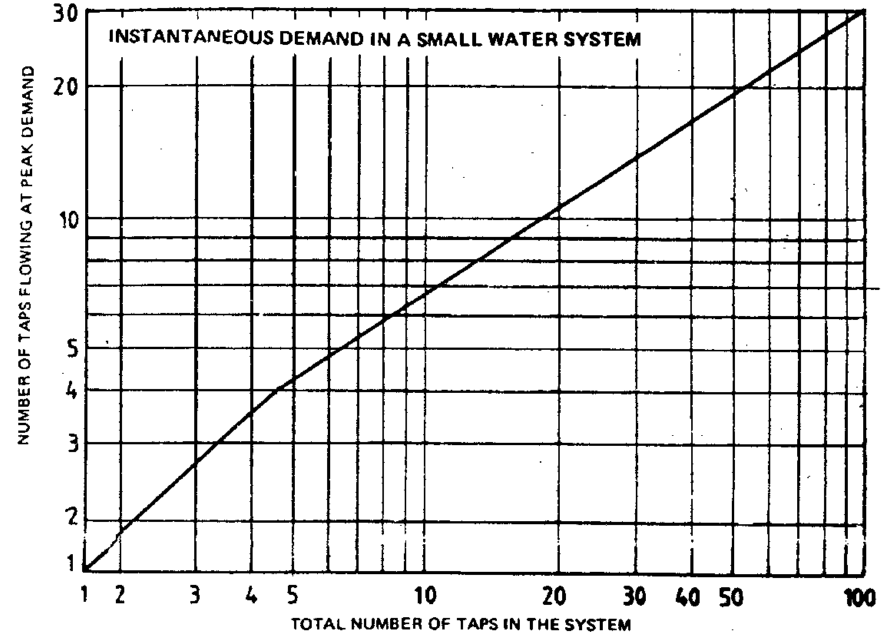
Water Supply from the Main to the Building

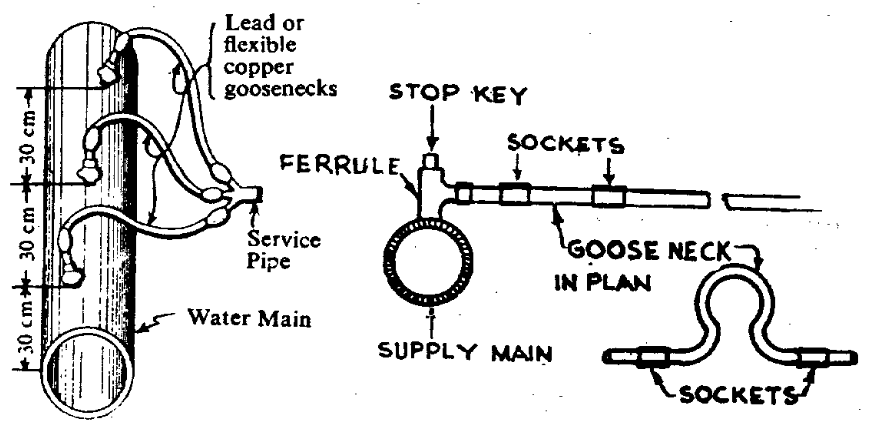
Service Pipe Connection



Under-Pressure Main Tapping Apparatus







Number of Connections of Various Sizes that can be Fed from a Main Line
that will together have the same discharge as the main line

Dia. of Delivery Main	Diameter of Branch Pipe in mm														
	15	20	25	32	40	50	65	80	100	125	150	200	250	300	350
	Number of Branch Connections														
15	1														
20	3	1													
25	6	2	1												
32	10	3	1.7	1											
40	16	6	2.7	1.6	1										
50	32	12	5.6	3.2	2	1									
65	56	20	10	5.6	3.6	1.7	1								
80	88	32	16	8.9	5.6	2.7	1.6	1							
100	181	66	32	18	11	5.6	3.2	2	1						
125		115	56	32	20	10	5.6	3.6	1.7	1					
150		181	88	50	32	15	8.9	5.6	2.7	1.6	1				
200			181		66	32	18	11	5.6	3.2	2	1			
250			316		115	56	32	20	10	5.6	3.6	1.7	1		
300						88	50	32	16	8.9	5.6	2.7	1.6	1	
350						130	74	47	23	13	8.3	4.0	2.3	1.5	1
400								66	32	18	11	5.6	3.2	2.0	1.4
450								88	43	25	16	7.6	4.3	2.7	1.9
500								115	56	32	20	10	5.6	3.6	2.4
600									88	50	32	16	8.9	5.6	3.8
700										74	47	23	13	8.3	5.6
750										88	56	27	16	10	6.5

Dia. of Delivery Main	Diameter of Branch Pipe in mm															
	200	250	300	350	400	450	500	600	700	750	800	900	1000	1100	1200	1500
mm	Number of Branch Connections															
400					1											
450					1.3	1										
500					1.7	1.3	1									
600					2.7	2.0	1.6	1								
700					4.0	3.0	2.3	1.5	1							
750					4.8	3.6	2.7	1.7	1.2	1						
800	32	18	11	7.9	5.6	4.2	3.2	2.0	1.4	1.2	1					
900	43	25	16	8.5	7.6	5.6	4.3	2.7	1.9	1.6	1.3	1				
1000	56	32	20		10	7.3	5.6	3.6	2.4	2.0	1.7	1.3	1			
1100	71	41	26			9.3	7.2	4.5	3.1	2.6	2.2	1.6	1.3	1		
1200	88	50	32				8.9	5.6	3.8	3.2	2.7	2.0	1.6	1.2	1	
1500	143	88	56				15	10	6.5	5.6	4.8	3.6	2.7	2.2	1.7	1

To know what number of pipes of a given size are equal in carrying capacity to one pipe of a larger size: At the same velocity of flow the volume delivered by two pipes of different sizes is proportional to the square of their diameters. Thus, one 100-mm dia. pipe will deliver the same volume as four 50—mm pipes. With the same head, however, the velocity is less in the smaller pipe, and the volume delivered varies about as the square root of the 5th power (*i.e.*, discharge varies as $\sqrt{\text{dia.}^5}$. The table has been calculated on this basis. The figures opposite the intersection of any two sizes represent the number of the smaller-sized pipes required to equal one of the larger. Thus, one 100—mm pipe is equal to 5.6—50-mm pipes.

$$\text{Number required} = \sqrt{\left(\frac{\text{Dia. of larger pipe}}{\text{Dia. of smaller pipe}}\right)^5}$$

cause lead poisoning as explained elsewhere. Wrought iron pipes are cheap and light and are easily jointed by screwing them to coupling.

Prohibited Connections. A service pipe shall not be connected into any distribution pipe, such connection may permit the backflow of water from a cistern into the service pipe, in certain circumstances, with consequent danger of contamination. No service pipe shall be connected to any WC or urinal. All such supplies shall be from flushing cisterns which shall be supplied from a storage tank.

Resistance to flow by fitttings, valves, etc. The pressure, volume, and velocity of water passing through a pipe are affected, in addition to the friction on the sides of the pipe, by the friction in passing through valves and fittings. The estimates of this additional friction by different authorities vary considerably. The following two schedules are commonly used, which can be taken as a good working guide to estimate the friction losses caused by fittings or valves in a run of pipe, each fitting or valve causing additional friction equal to an additional length of straight pipe.

Globe valve, open	275	times the internal pipe dia.
Angle valve, „	150	„ „ „
Gate valve, „	9	„ „ „
90 deg. elbow	30	„ „ „
45 deg. „	20	„ „ „
Tee, straight through	16	„ „ „
Tee, through side outlet	60	„ „ „

Fittings	Equivalent length of straight pipe in metres
Sluice valve, full-bore type	4.6
Non-return valve, full-bore type	6.0
Foot valve, full-bore type	6.0
Strainer	12.0
Bends	4.6*

*This length is taken for 225 mm (9″) dia. and 90 deg. bend, 250 mm (10″) dia. and 45 deg. bend, 300 mm (12″) dia. and 22 deg. bend. The length of pipe equivalent is increased with larger pipes and decreased with smaller ones.

Except in congested areas, the head losses through fittings and gate valves are relatively small in relation to the pipe friction loss. When the length of pipe is greater than 1000 times its diameter, the loss of head due to valves and fittings may be disregarded.

Strength of Pipe Design

When a thin pipe is subjected to internal pressure the stresses induced are of two kinds : tangential or circumferential (hoop stress) and longitudinal. The internal diametral pressure which produces tangential stress tends to extend its diameter and burst the pipe, and the longitudinal stress tends to elongate the pipe. Tangential stress is resisted by longi-

tudinal joints or circumferential reinforcement, and longitudinal stress by circumferential joints or longitudinal reinforcement. The longitudinal stresses develop only where the pipe changes in size or direction or has a closed end. The longitudinal stress is half the tangential stress, therefore circumferential joints need only be half the strength of the longitudinal joints.

The internal pressure in a pipe is worked out from the head of water it has to stand plus pressure due to "water hammer". A pipe must withstand the highest internal pressure to which it is likely to be subjected.

For the design of concrete pipes, the head of water is multiplied by four to be on the safe side. Thickness of wall of the pipe to resist circumferential tension is based on the composite section, i.e., area of concrete plus equivalent area of steel. For method of design see under "Design of Circular Water Tanks" in Section 8—Reinforced Concrete.

External Loads Over Mains generally arise from trench filling and from superimposed loads due to traffic. If a pipe is laid on a good, uniform continuous bed and the cover does not exceed the normal depth (about one metre) no special strengthening to resist external loading is necessary. When, however, a main is laid under deep cover or is subjected to heavy superimposed surface loads at less than normal cover, the pipes need strengthening. Laying of heavier pipes can often be avoided by careful bedding and trench filling as described elsewhere.

Information Regarding Interchangeability of the Old (Inch) Sizes and the New Metric Sizes for Cast Iron Pipes :

Nominal Bore		Mean outside Dia. of Barrel Old	Mean Socket Bore Old	Mean outside Dia. of Barrel New	Mean Socket Bore New	Clearance (cauling space)	
Old	New					New Spigot into old Socket	Old Spigot into New Socket
inch	mm	mm	mm	mm	mm	mm	mm
3	80	95.5	114.8	98	116	8.4	10.2
4	100	121.9	142.2	118	137	11.6	7.5
5	125	149.9	169.2	144	163	12.6	6.5
6	150	177.3	196.6	170	189	13.3	5.8
8	200	232.2	251.5	222	241	14.7	4.4
10	250	286.0	305.3	274	294	15.6	4.0

(i) New Metric size pipes are interchangeable among its classes for all diameters (that is, they have same external barrel diameter and internal socket diameter for a particular size). But old inch sizes above 10 inch nominal diameter are not interchangeable among A, B class and C, D class as external barrel diameters and internal socket diameters for class C and D are greater than class A and B.

(ii) For sizes bigger than 250 mm (10″) new metric spigot into old (inch) socket gives caulking spaces more than what is good for a joint. Old (inch) spigot into new metric socket is not possible after 100 mm diameter as the caulking space is less than the practical minimum.

(iii) In case of pipes where interchangeability is not possible double adapters may be used.

(iv) The increased diameter due to a bead on the spigot where this occurs must be allowed for. Figures in the table are based on plain barrel dimensions and diameters of barrel and socket.

Relative Discharging Capacities of Full Pipes

(Discharge varies as $\sqrt{d^5}$ approx.)

Dia. of Pipe	Relative discharging power	Dia. of Pipe	Relative discharging power
1	1.0	14	733
1.5	2.76	15	871
2	5.66	16	1024
2.5	9.90	17	1192
3	15.6	18	1375
4	32	20	1789
5	56	22	2270
6	88	24	2822
7	130	26	3447
8	181	28	4149
9	243	30	4930
10	316	32	5793
11	401	34	6741
12	498	36	7776
13	609	38	8870
		40	10150

Water Hammer, Surge Pressure or Concussion is the momentary pressure produced by the sudden stoppage of a mass of moving water at high velocity in a closed conduit or pipe, and is usually much in excess of, may be anything up to 15 to 20 times, the original hydrostatic pressure. This is caused by quickly closing a tap or a valve, or the sudden starting of a pump. This, in course of time, may burst the walls of the pipe or cause damage to the fittings of the system.

If valves are closed slowly, hammer pressure is considerably reduced but this is not always practicable. Slow closing taps should always be used on high pressure service pipes. Rubber or rubberised tap washers cause water hammer. The most effective remedy is to fix an air vassel, which will act as a shock absorber at or near the closure points of pumps and plumbing systems. Pressure reducing valve can also be fitted on the service pipe before it enters the building. Water hammer can also be guarded against by the use of a tower surge tank or standpipe at the end of the line, or relief valves.

Pressure due to water hammer may be taken as follows in addition to the static pressure in pipes :—

Dia. of pipe in cm	8 to 25	30	40	50	60	75	90	100 to 150
Pressure in kg./sq. cm	8.4	7.7	7.0	6.3	6.0	5.6	5.3	4.9

For small pipes the operation of hydrants and large branches has a relatively great influence on the system. For pipes under heavy pressure, where hammer pressure would be small in proportion to the static pressure, no hammer allowance need be considered.

5. PIPES OF DIFFERENT METALS

Choice of materials for pipes. The selection of a particular material will depend upon the purpose of a pipeline, character of the water to be conveyed, nature of the ground in which pipes are to be laid and the cost of each class of pipe on the site. Galv. iron pipes are used for distribution systems and small services ; cast iron and asbestos cement pipes are suitable for water supply pipes of moderate dia. and where frequent branches and specials are required. Steel and pre-stressed concrete pipes are used for large diameters and high pressures. Reinforced concrete pipes are used for water mains carrying moderate pressures.

Cast Iron Pipes

Cast iron pipes are the most extensively used for water mains as they are known to have good durability (are not subject to corrosion), good strength, low cost of maintenance, and can be easily tapped for making service connections. The cast iron pipe has the disadvantage of heavy weight, high transport costs, high pipe laying and jointing costs because of short lengths, low tensile strength liability to defects, such as sand-holes and blowholes, and the roughness of the internal surface.

Cast iron pipes are generally suitable for working pressures of up to 130 metres of vertical head of water, and up to 1000 mm diameter. Above this size, the use of either steel or RC pipes should be considered which may be more economical. Diameters beyond 1200 mm are not convenient to handle and fix. Cast iron mains must be strong enough to resist a test pressure of at least twice the working pressure.

These pipes are attacked by soft peaty waters, and hard waters also tend to cause incrustation of calcarious matter which offer considerable resistance to the flow of water and consequently reduce discharge. Cast iron pipes laid in chemically impregnated soils, town refuse, ash and cinder heaps, are liable to heavy corrosion. Ashes, cinders, clinker and some slag, which are found in made-up ground, are strongly corrosive in the presence of water. Cast iron or ordinary steel pipes perish in about 30 years in salt impregnated areas or coastal areas but otherwise cast iron pipes have been found to be in good condition even after use for 200 years. A lining of cement mortar protects the internal surface of pipes from the corrosive action of waters.

Cast iron pipes have been standardised in three classes according to the pressures for which the pipes are manufactured. (See Tables). Class LA, Class A and Class B for centrifugally cast (spun) pipes, and Class A and Class B for vertically cast pipes.

Manufacture of Cast Iron Pipes

There are two principal processes: (a) By casting the metal into vertical moulds which is the usual foundry method of casting in fixed sand moulds. These are called *vertically-cast* or pit-cast, and pipes are made up to 1500 mm diameter and of different standard lengths up to 5.5 metres, (b) By centrifugal or spinning process, which are metal-spun or sand-spun. These are called centrifugally-cast (spun) pipes. The mould has a solid core in the centre equal to the inner diameter of the pipe, leaving an annular ring around the core equal to the thickness of the pipe. A cylindrical pipe is produced by pouring molten iron into a rapidly rotating mould which causes the centrifugal motion.

Spun-iron Pipes. The process of spinning gives a uniform dense texture to the pipe with closer grain in the metal and greater tensile strength, toughness and elasticity, allowing the use of a thinner wall than that of a cast iron pipe of equal strength. Spun iron pipes are about three-quarters to two-thirds of the weight of vertically cast pipes of the same class. Spun iron pipes are more compact, freerer from blow holes and, weight for weight, appreciably stronger. Inner surface is also smoother than that of the vertically cast iron pipes. These are manufactured from 50 mm to 750 mm diameter and of different standard lengths up to 5.5 metres. Spun iron pipes are now superseding the cast iron pipes. But these pipes are more liable to be affected by corrosion troubles because of thinner walls. Extra wall thickness gives additional beam strength to the pipe.

Advantages of using steel or wrought iron pipes should be considered where heavy thrusts or high tensile stresses are anticipated. In cases where pipes are subjected to high internal pressures, as in the case of inverted siphons tending to draw the joints apart, flanged pipes should be used.

All pipes are coated with some standard treatment before leaving works. For water, iron pipes are coated externally and internally, and for gas they are coated externally only as coal gas has a solvent action on the coating. The coating has a coal tar or bitumen base and is applied by a hot dipping process (or centrifugally) and thoroughly incorporated with the metal. Thicker coatings give better protection. The surface coating must be with a bright gloss forming a tough enamel. This treatment makes the pipe rust-proof and increases its life. Care should be taken while jointing pipes that the coating is not injured. (The term "Dr. Angus Smith's solution is often used in reference to coating solutions, but this is a misnomer as the original formula, patented in 1848, is not now in use.)

Dimensions of thickness of pipe barrel of Class A and B, and outside diameter of barrel also apply to flanged pipes up to 300 mm nominal diameter.

Centrifugally Cast (Spun) Iron pipes

Nominal Dia. of pipe mm	Outside Dia. of Barrel mm	Thickness of Pipe Barrel (wall)		
		Class LA Pipe mm	Class A Pipe mm	Class B Pipe mm
80 (3")	98	7.2	7.9	8.6
100 (4")	118	7.5	8.3	9.0
125 (5")	144	7.9	8.7	9.5
150 (6")	170	8.3	9.2	10.0
200 (8")	222	9.2	10.1	11.0
250 (10")	274	10.0	11.0	12.0
300 (12")	326	10.8	11.9	13.0
350 (14")	378	11.7	12.8	14.0
400 (16")	429	12.5	13.8	15.0
450 (18")	480	13.3	14.7	16.0
500 (20")	532	14.2	15.6	17.0
600 (24")	635	15.8	17.4	19.0
700 (28")	738	17.5	19.3	21.0
750 (30")	790	18.3	20.2	22.2

pipes have been classified according to their wall thickness. Class A pipes have 10 per cent greater thickness than Class LA, Class B pipes have 10 per cent greater thickness than Class A pipes.

Standard Lengths of Pipes

(A) Centrifugally cast pipes :

(i) Standard lengths of barrels for socket and spigot pipes : 2, 2.8, 3, 4, 4.88, 5, 5.5 metres

(ii) Ditto. for flanged pipes (end to end of flange) : 3.66, 4, 4.88, 5, 5.5 metres.

(B.) Vertically cast pipes :

(i) Standard lengths of barrels for socket and spigpt pipes : 3.66, 4, 4.88, 5, 5.5 metres.

(ii) Ditto. for flanged pipes (end to end of flanges) : 1 to 3 metres for 80 mm pipes and 1 to 4 metres for other sizes.

Hydrostatic Test Pressure for Cast Iron (Centrifugally Spun) Pipes

Class of Pipe	Hydrostatic Test Pressure at Works	Max : Hydrostatic Test Presure After Installation	
	Socket and Spigot Pipes		
	kg/sq. cm	kg/sq. cm	Head of Water
LA	35	12	120 m
A	35	18	180 m
B	35	24	240 m
	Flanged Pipes (up to 300 mm dia.)		
A	35	18	180 m
B	35	24	240 m

Hydraulic Test Pressure for Vertically Cast Iron Pipes

Nominal Diameter	Test Pressure in kg/sq. cm			
	Socket and Spigot Pipes		Flanged Pipes	
	Class A	Class B	Class A	Class B
Up to and including 300 mm	20	25	20	25
Over 300 mm up to 600 mm	20	25	15	20
Over 600 mm up to 1000 mm	15	20	10	15
Over 1000 mm to 15000 mm	10	15	10	10

Diameter of Flanges of Cast Iron Pipes (both Centrifugally Spun & Vertically Cast)

Nominal dia. of pipe —mm	80 (3")	100 (4")	125 (5")	150 (6")	200 (8")	250 (10")	300 (12")	350 (14")	400 (16")	450 (18")
Dia. of flange (outside)— mm	200	220	250	285	340	395	445	505	565	615
Nominal dia. of pipe —mm	500 (20")	600 (24")	700 (28")	750 (30")	800 (32")	900 (36")	1000 (40")	1100 (44")	1200 (48")	1500 (60")
Dia. of flange (outside)— mm	670	780	895	960	1015	1115	1230	1340	1455	1800

Conversion of Inch Diameter Pipes into (exact) Millimetres

Dia. in Inches	3	4	6	9	12	15	18	21	24	27
Millimetres	76	102	152	229	305	381	457	533	610	686
Dia. in Inches	30	33	36	39	42	48	54	60	66	72
Millimetres	762	838	914	991	1057	1219	1372	1524	1676	1829

Steel Pipes are made for water distribution purposes from 100 mm to 1800 mm or larger for special requirements and are generally used for long exposed rising mains, trunk mains, inverted siphons, and on bridges and other structures where strength and least weight are required, and also where pressures are high (above 7 kg/sq. cm). Steel pipes are seldom used in diameters below 600 mm for water supply distribution mains owing to the difficulty of making service connections because the wall thickness of a steel pipe is not sufficient to permit tapping without installing special saddles, or repairs to a pipe line in case of a burst. The removal of rivets or welds and the substitution of a new pipe takes an unusually long time. Large-diameters steel pipes are used frequently as transmission lines and principal feeder for long stretches.

Steel pipes are generally welded as welding has now entirely replaced the earlier riveted types, and can be made in many weights or thickness designed for particular service. Because of its greater strength and ductility, the wall thickness of steel pipes is much less than that of cast iron and may be about only one-half for small diameters and one-third for large diameters, consequently saving in weight, transport, pipe-laying and jointing costs. Necessary pipe thickness can be obtained for each section for different pressures. Considering the effect of age on the carrying capacity of pipes, it has been noted that steel pipes after 30 years' use deliver 20 per cent more water than cast iron pipes of the same bore.

Steel pipes usually have flanged ends and are made in long lengths, usually 5.5 metres or more, and thus have lesser joints. Special types of flexible and expansion joints are available with rubber gaskets or rings. Steel pipes up to 300 mm diameter are also made for direct coupling to cast iron mains of similar size. Such pipes can be made with bell and spigot joints for connection to cast iron pipes by welding extra rings of proper diameter around the ends of the steel pipes. Under most conditions the stresses produced by temperature variations are well within the strength of the steel. Expansion joints are necessary for long lengths of pipe lines. Steel pipes are less liable to breakage in transit. Owing to their flexibility, which enables them to adapt themselves to relative changes in ground level without failure, steel pipes are suitable for laying in grounds liable to subsidence. This is particularly true if the pipes are joined by a flexible joint, which forms an additional safeguard against failure. Steel has now largely superseded wrought iron.

A steel pipe is very much liable to the actions of acids and alkalies in water and a slight trace of these will produce rust and incrustations. Soft and acidic waters rapidly attack small-bore wrought iron and steel pipes. The presence of rivets further gives better nucleus to the process of rusting. Steel pipes are more susceptible to corrosion than cast iron pipes and must be protected against it inside and out, and also couplings and bolts. Steel pipes need frequent painting. Thinner walls of steel pipes and greater susceptibility to corrosion are likely to cause high maintenance charges and shortened life.

The wall thickness required for internal working pressure is determined from the following formula : $t=\frac{P\times d}{2\times S\times F}$

Where :

t=thickness of pipe wall in cm ;

P=internal water pressure in kg/sq. cm (which causes the hoop stress) ;

S=safe working stress for pipe wall material in kg/per sq. cm—may be taken at about 1100 for steel pipes ; F is factor for efficiency of longitudinal joints. F is usually taken as follows : For single riveted lap joints 0.60, for double riveted 0.70, for triple riveted lap joints 0.80 For a lock-bar joint or seamless pipe 1.00. Strength of welded pipe is taken at 90 per cent.

It is usual to provide wall (plate) thickness 3 mm in excess of the theoretical as insurance against corrosion and water hummer. The value thus found, is taken to the next larger thickness of the plate. In large pipes under small pressure the thickness of metal computed must still be increased to given the pipe the necessary stiffness.

Steel pipes are not adapted to withstand heavy external loads and a partial vacuum caused by sudden emptying of a pipe may cause collapse and distortion by the unbalanced external pressure. Therefore, these pipes are not suitable for underground works under heavy fills, and depth of cover must not be such that the weight of the earth will flatten and deform the pipe. For filling, firm material should be used carefully placed around the pipe and well rammed on the sides. Anchorages have to be built to keep the pipes from moving at all free ends and at all sharp bends. (This has been explained under Resistance to Thrusts—Laying of Pipe Lines).

Where a steel pipe is to be laid in a swampy or water-logged ground or soil containing deleterious earth or salts, the particular length of the pipe can be protected by first painting the steel pipe before laying and then wrapping round the pipe a length of jute cloth in the form of a bandage, so that this cloth will stick to the composition which has been freshly applied.

Steel pipes should be stored not over four tiers high with 2.5×30 cm wooden strips not over 120 cm apart between layers.

Value of the co-efficient of discharge for riveted pipes is taken 20 per cent lower than that for a full welded pipe because of the rivet projections.

Data for Steel Water Pipes

Thickness for Various Diameters and Working Pressures

Inside Dia. of Pipe	Wall Thickness (Plate)	Safe Working Pressure Head of Water	Max : Safe Span	Inside Dia. of Pipe	Wall Thickness (Plate)	Safe Working Pressure Head of Water	Max : Safe Span
mm	mm	m	m	mm	mm	m	m
600	5	160	10	1200	6	100	12
	6	200	11		8	135	13
	8	275	12		10	165	14
	10	330	13		12	220	15
					14	300	16
750	5	130	10	1350	8	120	13
	6	160	11		10	150	14
	8	220	12		12	175	15
	10	260	13		14	200	16
	12	350	14		16	250	17
900	6	130	11	1500	8	110	13
	8	180	13		10	130	14
	10	220	14		12	155	15
	12	300	15		14	180	16
	14	400	16		16	220	18
1050	6	110	12	1800	8	90	14
	8	155	13		10	110	15
	10	185	14		12	130	16
	12	250	15		14	150	17
	14	350	16		16	185	18

Theoretical plate thicknesses have been rounded off according to the thickness of steel plates manufactured. The table is good for estimating purposes. Exact thickness can be worked out from the formula given at page 15/78.

Welding and Cutting of Steel Pipes

Nowadays, steel pipes are usually jointed by welding. Welding is used instead of threaded couplings and flanged joints ; even when flanged joints are used for connecting pipeline fittings, the flanges at the ends of steel pipes are joined by welding.

Welding produces highly reliable and pressure-tight joints which is of prime importance when the pressure in the pipeline is great. The application of welding considerably saves metal.

There are three main kinds of welding : fusion welding, plastic pressure welding and cold welding.

Fusion welding includes manual metal-arc electric welding, automatic submerged-arc electric welding, gas shielded arc electric welding, and gas welding.

Pressure welding in the plastic state includes electric-resistance and electric-fusion contact welding, ,induction pressure welding, pressure arc welding, and gas-pressure welding. (See page 5/23).

Despite considerable improvements in pipe-welding engineering in recent years and the development of automatic pipe welding, manual arc welding is still widely used in pipeline construction especially for welding non-rotated pipe joints, i.e. in overhead joints, lap welding, and fixing of pipe coils.

Recently, metal-arc welding has been considerably improved. In arc welding, an electric arc flame is formed between the electrode connected to one terminal of the welding current supply source and the work being welded connected to the second terminal of the source. The heat produced by the arc (its temperature reaches 3000-4000°C.) melts the metal. Here the electrode metal mixes with the molten metal of the work being welded. In metal-are welding the arc gives off heat to a relatively small area ; the caving formed in the welded metal is called a pool or crater.

Cement Concrete Pipes

Cement concrete pipes (with or without reinforcement) are now coming into general use as they have many advantages over other materials. If properly made they are better than metal pipes in many respects as there is no rusting and incrustations inside the bore. These pipes are not suited to carry acidic or alkaline waters or temporary hard waters and waters charged with sediments. The former will make the bore of the pipes rough and the latter will leave deposits inside the pipes which become hard incrustations in the course of time. Neither these pipes can be laid in soils with concentrations of alkaline salts, but are useful in black-cotton soils. (This subject has been discussed in detail in Section 8—"Reinforced Concrete"). Cement concrete pipes are relatively less expensive and durable except in soils having a high percentage of salts mentioned above.

Joints in concrete pipes. Small pipes are often provided with spigot and socket ends and are jointed with cement like the stoneware pipes. Large size pipes have mortise and tension joints-butt ends with collars. Joints have also been described in the Section on "Sewerage."

Bends, junctions and specials are of cast iron as for cast iron soil pipes. To repair leakage from a joint the earth at the joint is dug out, collar broken and the pipes are cleaned with iron brushes. Form is then made under the joint and cement concrete filled in it from one side until it comes out from the other side. Concrete is then laid over the joint so as to make a complete collar. The thickness of the collar should be at least 50 mm to ensure water-tightness. Hemp soaked in cement slurry 1 : 2 is wrapped all round before a cement concrete collar is made around the joint. The spigot and socket shall be thoroughly wet before the joints are made.

RCC Pipes. Unreinforced pipes are not generally used for diameters

above 450 mm (with 600 mm dia. max:) (See also under "Culverts" in Section 19.) Steel fabric or wire cages are used, in single or double layers, inside and outside for gripping the concrete. Steel cylinders are also used instead of fabric. This core of steel acts as a water-tight layer. Concrete pipes develop cracks even if heavily reinforced, when the tensile stress in steel exceeds modular ratio times the ultimate tensile stress of the concrete used.

Concrete pipes are best suited for conditions where the pressure is low, and the danger of shock is small. A surge pipe or vacuum chamber is usually installed near the pump to reduce the danger of sudden shocks. They are not used for pressures above 50 metres head of water. For greater pressures Hume-steel pipes are used. These pipes will break if the foundations settle. They are heavy and difficult to transport and need great care in handling. Difficult to repair.

Manufacture of Concrete Pipes

Concrete pipes are either cast in vertical moulds by centrifugal spinning in which the ingredients are passed into raidly revolving cylinders which produce dense and homogenous concrete or are made by pressure process in moulds.

Reinforcement made in the form of cages should extend throughout the pipe barrel and wound round collapsible frames or drums. The cages shall consist of spirals or rings and straights of annealed wire, cold-drawn wire or mild steel rods and may be circular. When double reinforcement cage is used, the amount of steel in the outer cage shall be 75 per cent and 25 per cent in the inner cage. Additional spiral reinforcement of mild steel wire of the same diameter shall be closely spaced at the end of the pipe for a length of 150 mm to minimize damage during handling. The spacing of such end spiral shall not exceed 25 mm or half the pitch whichever is less. Diagonal reinforcement shall be provided at 15 per cent of longitudinals if the cages are not welded ; this will help in binding the cage securely.

The pitch of the circumferential reinforcement shall be neither more than 10 cm or four times the thickness of the barrel, whichever is less, nor less than the maximum size of aggregate plus the diameter of the bar used.

In the case of concrete, other than the controlled concrete, the mix for non-pressure pipes shall have a minimum cement content of 360 kg/cu. metre and a minimum compressive strength of 185 kg/sq. cm at 28 days in works tests. If mortar is used, it shall have a minimum cement content of 450 kg/cu. metre and a compressive strength not less than 160 kg/sq. cm at 28 days at work tests.

The mix for pressure pipes shall have a minimum cement content of of 450 kg/cu. metre and a minimum compressive strength of 215 kg/sq. cm at 28 days at works tests. If mortar is used, it shall have a minimum cement content of 600 kg/cu. metre and compressive strength not less than 200 kg/sq. cm at 28 days in works tests.

Data for Cement Concrete Pipes

Internal Dia. of Pipe mm	NP$_1$ Barrel Thickness mm	NP$_2$ Barrel Thickness mm	NP$_2$ Reinforcement Longdnl. kg/linear m	NP$_2$ Reinforcement Spiral kg/linear m	NP$_3$ Barrel Thickness mm	NP$_3$ Reinforcement Longdnl. kg/linear m	NP$_3$ Reinforcement Spiral kg/linear m	NP$_4$ Barrel Thickness mm	NP$_4$ Reinforcement Longdnl. kg/linear m	NP$_4$ Reinforcement Spiral kg/linear m
80	25	25	0.86	0.08						
100	25	25	0.86	0.17						
150	25	25	0.86	0.22						
250	25	25	0.86	0.71						
300	30	30	1.00	1.29						
350	32	32	1.00	1.75	75	1.25	7.25	—	—	—
400	32	32	1.00	2.25	75	2.15	8.25	75	1.71	7.74
450	35	35	1.25	2.75	75	2.40	9.00	75	2.14	10.62
500	—	35	1.25	3.22	75	2.45	10.50	85	2.14	13.38
600		40	1.25	4.90	80	2.45	14.50	85	3.42	27.18
700		40	1.78	6.05	80	2.95	23.75	95	3.44	29.17
800		45	1.78	9.10	90	3.60	26.50	100	4.46	37.56
900		50	1.78	11.35	100	4.35	32.75	115	5.91	49.12
1000		55	2.50	13.50	100	5.80	44.00	115	7.40	58.10
1100		60	2.50	16.30	115	7.40	52.50	120	9.45	78.75
1200		65	2.50	18.20	115	7.60	63.75	135	11.80	103.8
1400		75	3.36	22.60	—	—		140	17.70	148.6
1600		80	3.36	28.30				150	22.47	193.0
1800		90	3.36	36.00				—	—	—

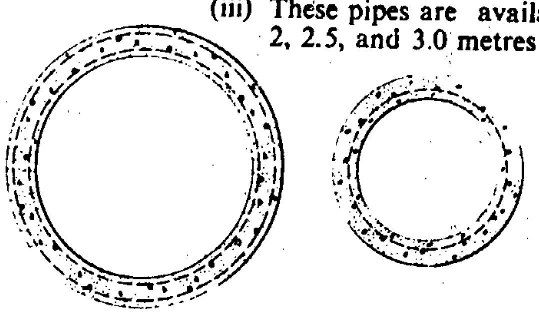
		P$_1$			P$_2$			P$_3$		
80		25	0.06	0.21	25	0.86	0.47	25	0.86	0.60
100		25	0.86	0.33	25	0.86	0.65	25	0.86	0.86
150		25	0.86	0.64	25	0.86	1.25	25	0.86	1.67
250		25	0.86	1.57	30	0.86	2.95	35	0.86	4.62
300		30	1.00	2.25	40	1.00	4.65	45	1.00	6.60
350		32	1.00	2.92	45	1.00	6.30	55	1.00	8.60
400		32	1.00	3.80	50	1.00	8.07	60	1.09	8.60
450		35	1.25	4.75	55	1.00	10.10	—	—	11.50
500		35	1.25	5.75	55	1.25	12.40			—
600		40	1.25	8.15	65	1.25	18.00			
700		40	1.78	11.20	—	—	—			
800		45	1.78	14.50						
900		50	1.78	18.00						
1000		55	2.50	22.50						
1100		60	2.50	27.50						
1200		65	2.50	32.50						

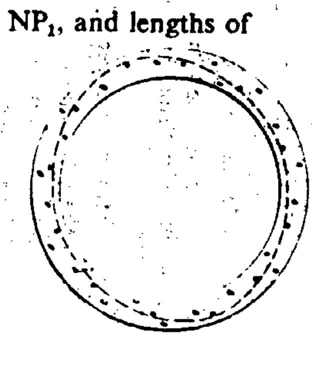
IS: 458—1971

(i) Reinforcement for Longitudinal steel has been based on a permissible stress of 1265 kg/sq. cm.

(ii) Spiral reinforcement is hard-drawn (or cold drawn) steel wires with permissible stress of 1400 kg/sq. cm.

(iii) These pipes are available in lengths of 1 metre for class NP$_1$, and lengths of 2, 2.5, and 3.0 metres for other classes





Cross-section of Reinforced Concrete Pipes

The quantities of cement indicated above do not include the quantity of cement required for surface finish of the pipe if required.

The maximum size of aggregate shall not exceed one-third the thickness of the pipe or 20 mm, whichever is smaller. The concrete shall not be leaner than 1 : 2 : 4.

Unreinforced and reinforced concrete non-pressure pipes shall be capable of withstanding a test pressure of 0.7 kg/sq. cm (7 metres head of water).

Minimum cover of reinforcement for pipe thicknesses :—

Less than 30 mm	9 mm for spun and	12 mm for other pipes
30 to 75 mm	12 mm „	16 mm „
75 and over	18 mm „	18 mm „

Laying of Concrete Pipes

(See under Section 16—"Drainage and Sewerage") see page 15/166

Follow the general principles given for cast iron soil pipes.

Quantity of Cement and Gaskin per joint for RCC Pipes with collar, jointed with Cement Mortar 1 : 1.

Dia. of Pipe	Cement kg	Gaskin kg	Dia. of pipe	Cement kg	Gaskin kg
mm			mm		
100	2	0.10	600	20	0.72
150	3	0.18	700	24	0.83
250	5	0.29	800	27	0.95
300	7	0.36	900	30	1.08
350	9	0.40	1000	34	1.20
400	12	0.50	1100	37	1.30
450	15	0.55	1200	40	1.85
500	18	0.60	—	—	—

Cement Concrete Pipes

Classification according to IS : 458

Class	Description	Conditions where normally used
NP 1	Unreinforced concrete, non-pressure pipes	For drainage and irrigation use, above ground or in shallow trenches
NP 2	RC, light duty, non-pressure pipes	For culverts carrying light traffic
NP 3	RC, heavy duty, non-pressure pipes	For culverts carrying heavy traffic
NP 4	RC, Ditto.	For culverts carrying very heavy traffic, such as railway loadings
P 1	RC pressure pipes tested to a hydrostatic pressure of 2.0 kg/sq. cm (20 m head)	For use on gravity mains, the actual working pressure not exceeding 2/3 of the test pressure
P 2	RC pressure pipes tested to a hydrostatic pressure of 4.0 kg/sq. cm (40 m head)	For use on pumping mains, actual working pressure not exceeding 1/2 of the test pressure
P 3	RC pressure pipes tested to a hydrostatic pressure of 6.0 kg/sq. cm (60 m head).	Ditto.

Cement Lined Pipes are made by spreading 1:3 cement mortar evenly on the clean inside of the pipe and then revolving it. The standard average thickness of this coating is 1.5 mm for 150 mm dia. or smaller pipes, 3 mm for 200 mm to 400 mm pipes, and 5 mm for larger pipes. The outside of the pipes receive the usual coating. Such pipes stand corrosion exceedingly well. But these pipes are not very popular.

Asbestos-Cement Pipes are manufactured of a mixture of asbestos fibres and cement. Advantages claimed for these pipes are that the water carrying capacity remains substantially the same as when first laid as they have a smooth inside surface and are immune from attack from corrosive soils and alkalies or acids in water, and also tuberculation. These pipes are light in weight but they are fragile and cannot stand heavy impacts and blows during handling.

Asbestos-cement pipes can be cut easily with coarse-toothed hack-saw, drilled and tapped for service connections in the same manner as cast iron pipes, but have not the same strength and suitability for threading as iron pipes and any leakage at the thread will become worse as the time passes. Leaks are very difficult to detect and repair. Are liable to crack during drilling of holes for house connection unless done very carefully.

These pipes are generally supplied in lengths of 3 metres for pipes of diameter 100 mm and less and in lengths of 4 metres for pipe of diameters greater than 100 mm. Fittings and accessories such as bends, tees, reducers are of cast iron and of 'medium' or 'heavy' grade. Where the asbestos-cement pressure pipes are to be connected to any flanged cast iron fittings or specials, special cast iron adaptors are used.

Jointing of pipes and fittings are done with cast iron detachable joints. The detachable joint consists of two rubber ring sleeves, a cast iron collar, two cast iron flanges and sets of bolts, nuts and washers. (The number of bolts varies with the diameter of the pipes).

The joining is done by first sliping on the flanges, one each on the ends of the pipes to be jointed, followed by the rubber rings. The collar is placed on end of one pipe and the end of the other pipe brought in so that the collar is central to both the pipes. Then the flanges are moved in against the rubber rings and brought together and tightened with bolts. Mechanical couplings are also used. To join asbestos cement pipes with cast iron mains, the spigot end of the asbestos cement pipe can be caulked into the cast iron socket with molten lead in the usual manner described under cast iron pipes. These pipes have to be bedded on sand or light top soil.

Asbestos-cement pipes may be used for water mains up to 230 mm diameter, and they need a minimum cover of 1 metre. These pipes are not very popular except for use in small diameters and for rainwater pipes.

Asbestos-cement pipes are classified according to the test pressure as follows —

Class 1	5 kg/sq. cm	The maximum working pressure under which each pipe is used shall not exceed half the test pressure for that class of pipe.
Class 2	10 kg/sq. cm	
Class 3	15 kg/sq. cm	
Class 4	20 kg/sq. cm	
Class 5	25 kg/sq. cm	

Hume Pipes are manufactured by "The Indian Hume Pipe Co., Ltd., Bombay-I. These are RCC pipes and are used for drainage, irrigation, wells and culverts, etc. Standard diameters in metric are : 100, 150, 250, 300, 350, 400, 450, 500, 600, 700, 800, 900, 1000, 1100, 1200, 1400, 1600 and 1800 mm.

Standard diameters in inches are : 4, 6, 9, 12, 15, 18, 21, 24, 27, 30, 33, 36, 39, 42, 48, 54, 60, 66, and 72 inches.

Standard lengths are : Up to 250 mm in 2 metres or 6 ft. and over 250 mm in 2.5 metres or 8 ft.

Useful for test pressures of 2 kg/sq. cm, and 4 kg/sq. cm. The corresponding working pressures for these pipes may be taken as 50 per cent and 67 per cent for pumping mains and gravity mains repectively.

The latest method for jointing these pipes is with rubber rings. Collars are also provided.

Where it is proposed to use these pipes, details should be obtained from the manufacturers.

Galvanized Pipes of steel or wrought iron are widely used for distribution systems and give good service where the water is hard. With soft active waters such pipes deteriorate rapidly and should not be used. Galvanized wrought iron pipes and fittings are used where exposed to corrosive conditions such as, the presence of sea water or salty air.

Approximate Weight and Thickness of Medium Grade GI Water Pipes per Metre Length :—

Nominal bore in mm	10	15	20	25	32	40	50	65	80	100
Thickness in mm	2.35	2.65	2.65	3.25	3.25	3.25	3.65	3.65	4.05	4.50
Weight in kg/metre	0.85	1.21	1.57	2.42	3.11	3.59	5.07	6.49	8.43	12.0

If the pipes are affected by the action of water, and once the surface coating is gone, the pipes rust very fast and the incrustation is very great. Friction is considerably increased and the bore considerably reduced. Jointing of GI pipes is easy and they are also easy to transport and manipulate.

Galvanized iron pipes are manufactured in sizes of from 8 mm to 100 mm (15 mm to 65 mm are common) diameter.

Weight and thickness of pipes differ slightly with different manufacturers.

Jointing Galv. Iron Tubes and Fittings. The screw threads of the tubes and fittings should be carefully preserved from damage before jointing, and should be cleaned and smeared with red lead in linseed oil. In cases where the joints require it, owing to slackness of screw threads, cotton threads smeared with red lead may be used to make a water-tight joint. Taps and dies should only be used for straightening screw threads which have become bent or damaged and should not be used for turning of the threads so as to make them slack, this procedure would result in a non-water-tight joint.

In case a GI pipe has to be embedded in walls or floors it should be painted with anti-corrosive bitumastic paint. The pipe should not come in contact with lime mortar or lime concrete as this pipe is affected by lime.

Pipes laid near electric tram lines, power transmission lines, electric railway or power houses should be provided with insulating joints at frequent intervals to guard against electrolysis.

Galvanic and Electrolytic Corrosion. Corrosion often sets up when two dissimilar metals in contact with each other are immersed in water. Such an action takes place between pipes and fittings of different metals such as with galvanized steel tanks and copper pipes, or zinc and iron.

Lead Pipes are not generally used for domestic water supply as most of the waters in India are plumbo-solvent and are liable to cause lead poisoning. These pipes are suitable for fixtures inside buildings because of the ease with which they can be bent to follow an irregular line, and may be used for flushing and over-flow pipes. Lead or lead-lined pipes are permanent and their capacity is not reduced by corrosion but they are attacked slowly by soft and very pure active waters and because of lead poisoning can be safely used only for waters demonstrated to be without action on them. Hard water, unless containing a large excess of carbon dioxide, does not dissolve lead but forms a protective coating on the inside of the pipes.

Lead pipes are liable to corrosion by contact with damp cement or lime or concrete, and also with certain timbers. When a lead pipe passes through a wall or concrete floor it must be protected by wrapping round it thin bituminous felt or brown paper tied securely.

Lead and lead-alloy piping are jointed with "wiped" solder joints to cast-iron, wrought-iron, steel or copper piping by the use of copper-alloy or brass screwed unions or ferrules. (See "Plumbing and Internal Fixtures" in Section 16).

Frictional Head Losses in Galv. Iron Water Pipes for Designing Water Supply Schemes

Flow in Thousand Litres per Day	Frictional Head Loss in Metres per 1000 Metres of Pipe Length — Pipe Diameter in mm							
	25	32	40	50	65	80	100	150
14	19.4	5.85	1.96	0.78	0.12			
16	24.8	7.45	2.50	1.02	0.16			
18	30.8	9.26	3.13	1.26	0.19			
20	37.5	11.3	3.80	1.53	0.24			
22	49.3	13.5	4.54	1.84	0.28			
24	52.6	16.8	5.33	2.16	0.33			
26	62.0	18.4	6.19	2.50	0.38			
28	—	—	7.93	2.87	0.44			
30			8.60	3.26	0.50	—	—	
35			10.7	4.34	0.67	0.24	0.08	
40			13.8	5.51	0.85	0.31	0.10	
45			17.7	6.90	1.06	0.39	0.13	
50			20.7	8.40	1.29	0.47	0.16	
55			24.8	10.0	1.54	0.56	0.19	—
60			29.0	11.8	1.81	0.66	0.22	0.03
65			33.7	13.3	2.04	0.76	0.26	0.04
70			38.7	15.7	2.40	0.87	0.29	0.04
75			44.0	17.8	2.73	0.99	0.33	0.05
80			49.5	20.0	3.18	1.12	0.38	0.05
85			55.9	22.4	3.44	1.25	0.42	0.06
90			61.7	24.9	3.83	1.39	0.47	0.06
95			68.1	27.6	4.23	1.54	0.52	0.07
100			74.9	30.3	4.65	1.70	0.57	0.08
105			82.0	33.2	5.09	1.85	0.62	0.09
110			89.4	36.2	5.55	2.02	0.68	0.09
115			97.0	39.3	6.03	2.19	0.74	0.10
120			105.0	42.5	6.52	2.37	0.80	0.11
125			113.2	45.8	7.04	2.56	0.86	0.12
130			—	49.3	7.57	2.75	0.93	0.13
135				52.8	8.11	2.95	1.00	0.14
140				56.5	8.68	3.16	1.06	0.15
145				60.1	9.26	3.37	1.14	0.16
150				64.2	9.86	3.59	1.21	0.17
155				68.3	10.5	3.81	1.29	0.18
160				72.2	11.1	4.04	1.36	0.19
165				76.8	11.8	4.28	1.44	0.20
170				81.0	12.4	4.52	1.53	0.21
175				85.5	13.1	4.77	1.61	0.22
180				90.6	13.8	5.03	1.70	0.23
185				94.8	14.5	5.29	1.78	0.25
190				99.6	15.3	5.56	1.87	0.26

Frictional Head Loss in Metres per 1000 Metres of Pipe Length
Pipe Diameter in mm

Flow	80	100	150	Flow	100	150
195	5.83	2.00	0.27	1075	46.4	6.44
200	6.11	2.06	0.29	1100	48.4	6.72
210	6.69	2.26	0.31	1125	50.5	7.01
220	7.29	2.46	0.34	1150	52.6	7.30
230	7.92	2.67	0.37	1175	54.7	7.60
240	8.57	2.89	0.40	1200	56.9	7.90
250	9.24	3.12	0.43	1225	59.1	8.21
260	9.94	3.35	0.46	1250	61.4	8.52
270	10.6	3.59	0.50	1275	63.7	8.84
280	11.4	3.84	0.53	1300	66.0	9.16
290	12.2	4.10	0.57	1325	68.4	9.49
300	13.0	4.37	0.61	1350	70.8	9.82
325	15.0	5.07	0.70	1375	73.2	10.2
350	17.2	5.81	0.80	1400	75.7	10.5
375	19.6	6.60	0.92	1425	78.2	10.8
400	22.1	7.44	1.03	1450	—	11.2
425	24.7	8.32	1.56	1475		11.6
450	27.4	9.25	1.28	1500		11.9
475	30.3	10.2	1.42	1525		12.3
500	33.3	11.2	1.56	1550		12.7
525	36.5	12.3	1.71	1575		13.1
550	39.8	13.4	1.86	1600		13.5
575	43.2	14.6	2.02	1625		13.8
600	46.7	15.8	2.19	1650		14.2
625	50.4	17.0	2.36	1675		14.6
650	54.2	18.3	2.54	1700		15.1
675	58.1	19.6	2.72	1725		15.5
700	62.2	21.0	2.91	1750		15.9
725	66.4	22.4	3.11	1775		16.3
750	70.7	23.8	3.31	1800		16.7
775	75.1	25.3	3.51	1825		17.2
800	79.6	26.9	3.73	1850		17.6
825	84.3	28.4	3.95	1875		18.0
850	89.1	30.0	4.17	1900		18.5
875	94.0	31.7	4.40	1925		18.9
900	99.0	33.4	4.64	1950		19.4
925	104	35.1	4.88	1975		19.9
950	109	36.9	5.12	2000		20.3
975	114	38.7	5.38	2025		20.8
1000	120	40.6	5.64	2050		21.3
1025	—	42.5	5.90	2075		21.8
1050	—	44.4	6.17	2100		22.3

See next page

For Head Loss in Asbestos Cement Pressure pipes and PVC pipes multiply the figures for head loss in galv. iron pipes by 0.71. For galv iron pipes and asbestos cement pipes inner diameter (bore) and for PVC pipes outer diameter is taken for calculations. Add 10% for friction losses due to fittings etc in addition to head loss. The customary practice is to take these into account as an equivalent addition length of pipe line. Commercial pipe size should be taken for most conditions; the size next to the theoretical size is chosen.

For bigger sizes of pipes calculate from the Nomogram at page 15/161.

Various pipe flow formulae are available to determine the head losses in relation to velocity in pipes and they all give varying results. The table is based on a combination of formulae including the Hazen and Williams' formula (as given at page 14/20) with roughness co-efficient "C" of 100 which applies to pipes in use for 15 to 20 years with normal waters. The resulting figures may be too small when very corrosive or incrusting water is encountered. This formula is not suitable when the co-efficient of "C" is appreciably below 100.

The resulting figures should be taken as only "approximate".

Relative carrying capacity and Head Loss for various values of "C" for use with the Nomogram. (Page 15/161) Head Loss is "slope"

C	A	B
40	5.46	0.40
60	2.58	0.60
80	1.51	0.80
90	1.22	0.90
100	1.00	1.00
110	0.84	1.10
120	0.71	1.20
130	0.62	1.30
140	0.54	1.40

1. To determine loss of head with value of "C" other than 100, multiply the loss of head found in the nomogram by the "A" value given in the table.
2. To determine quantity of flow with value of "C" other than 100, multiply the quantity af flow found in the nomogram by "B" value given in the table.

(Also see Nomogram at page 15/161)

The action of water on lead is tested as follows :—

A strip of sheet lead is scrapped to expose a bright surface, another strip is left covered with the film of basic carbonate formed by the previous action of moist air on the lead. The strips are allowed to stand for 24 hours or more in separate beakers of the water to be tested. If the surfaces of the strips are altered, acid action is to be suspected. Erosion will be indicated by the presence of white basic lead carbonate (insoluble) in the water, and solvency can be confirmed by removing the strips and adding to the water a solution of sulphuretted hydrogen, when a darkening of colour shows the presence of lead.

Lead pipes are not suitable for hot water supply since they sag when heated and are not liable to recover fully after expansion, and also the chemical action of water is increased by heating. Lead-alloy pipes are lighter in weight than lead pipes. Bending of a lead pipe is best done to any curve by filling it with sand.

Lead pipes do not require the usual fittings such as, bends, elbows like pipes of other metals but can be easily bent, bored, expanded and jointed. Lead pipes are not much used in India.

Plastic Pipes—Polythene and PVC (Poly-Vinyl-Chloride) **Pipes** These pipes being more flexible, corrosion resistant, light in weight, easy to handle and install, relatively cheap and available in long lenghts compared to metallic pipes, are very suitable in certain locations of plumbing systems. Can be extensively used in hotels and house service connections, and are well suited for use in mountainous terrains and undutaling areas. One tonne of PVC pipes replace 6-8 tonnes of galvanized or cast iron pipes. They are found to be 30 per cent more economical. These pipes, however, are not suitable for hot water systems. Certain grades of polythene and PVC pipes contain traces of objectionable soluble substances; these should be avoided by ordering material that is known to be suitable for water supply. The plasticised PVC is not recommended because of the uptake of lead by the water.

Polythene Pipes are generally available from 10 to 75 mm (1/2 to 3 inches) diameter in long lengths. Polythene tube is the ideal solution to all small-bore pipe runs. It is relatively cheap, easy to handle and lay and, being available in long lengths, less likely to have leaking joints. Polythene pipe can be supplied with a wide variety of fittings and can be coupled to other types of pipes. Polythene pipe is not completely resistant to attack by rodents and some insects. Polythene pipe laid underground should be bedded in sand or soil with no sharp stones present; laid overground should be adequately supported to prevent sagging.

Nominal bore in mm	25	32	40	50
Nominal outside dia. in mm	32	40	48	60

Polythene pipes can be bent cold to a radius of not less than 8 times their external dia. Bends of smaller radius can be made by hot bending. For hot bending the pipe is heated by immersion in boiling water or by application of blow lamp flame taking about 3 minuts to heat the pipe. When thoroughly softened the pipe is bent to the desired radius using bending spring inside the bore and held until the pipe becomes completely cold.

Jointing Normal gauge pipes are not threaded, but are used with compression flange joints. Jointing procedure is as follows :—

The end of the pipe is cut square leaving about 3 mm to 5 mm more than the finished length required. The approximate union is then inserten on the pipe. The pipe is heated slowly and uniformly by means of metal baffle held in front of the end of the pipe and heated with a blow lamp. The pipe should not be heated directly by a naked flame. On thus heating the end of the pipe swells outwards. Heating is continued till the thickness of the swelling is nearly equal to the wall thickness of the pipe. The pipe is then pressed strongly against a flange forming tool to shape the required profile. The forming surface of tool is given a smear of glycerine to prevent the pipe flange from sticking. The flange thus formed is between 1.5 to 2 times the wall thickness of the pipe. If a polythene pipe is to be jointed with a galvanized iron pipe of the same diameter, a galvanized iron bush is screwed on the galvanized iron pipe and the other end of the galvanized iron bush is screwed into the female union nut on the polythene pipe.

Rigid PVC Pipes are light to handle, resistant to corrosion and easily laid, usually available in long lengths which can be handled easily by one person. Joints are easily made and a wide variety of fittings are available.

Fittings such as tees, elbows, crosses and reducers are provided with male and female union nuts and have moulded flanges. The sizes of the union fittings of pipes or specials up to 40 mm dia. are one size, i.e., 6.5 mm larger than the nominal bore of the pipe or specials, and in the case of 50 mm pipes the size of the union fittings is 15 mm larger than the nominal bore of the pipe or specials. Other types of fittings can also be used.

Plastic pipes are classified as 2.5, 4, 6 and 10.

Test Pressures and Allowable Working Pressures

Class of Pipe	Test Pressure		Allowable Working Pressure	
	kg/sq. cm	Head in metres	kg/sq. cm	Head in metres
2.5	3.75	37.5	1.875	18.75
4	6	60	3.0	30
6	9	90	4.5	45
10	15	150	7.5	75

IS : 3076 & 4985

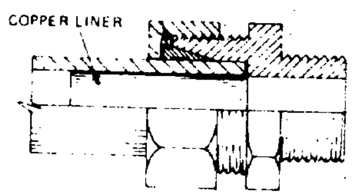
Pipe diameters (outer) generally available are 63 to 250 mm in standard lenghts of 3, 5 and 6 metres. Pipe sizes commonly used in water supplo systems are 90 and 110 mm diameters.

Following types of joints are used for jointing PVC pipes : (a) Cement Solvent joint, (b) Flanged joint; (c) Screwed or Threaded joint; and (d) Rubber Ring joint. Cement solvent joint is commonly used.

High Density Polythene (HDP) pipes are jointed by butt welding process. PVC or HDP saddle pieces are used for providing house service connections.

Reinforced Fibre glass plastic pipes for water transmission and sewerage are also now manufactured.

Polythene pipe connection to steel or copper pipe



Copper Pipes are generally used for hot water installations and for interior works in small diameters. Copper piping are jointed to other pipes by copper alloy screwed unions or ferrules.

Fittings for Service Pipes

In every case a ***stop-cock*** should be provided within the boundary of the permises between the street main and the building, situated in a convenient and accessible part of the premises, and it should be enclosed in a proper iron box with a hinged cover. In flats and tenements supplied by a common service pipe, a stop tap shall be fixed to control the branch to each separately occupied part.

Bends are manufactured to certain radii (about 20 sizes) varying from 32 mm to 500 mm internal radius and 100 mm to 1000 mm long. Slight bends can be made on the site by cold-bending a tube. Sharper bends can be made by hot-bending, but heat spoils galvanizing. Wrought iron pipes may be bent through angles not exceeding 45 deg. but pipes of more than 50 mm diameter cannot be bent to any great extent.

The following are special fittings :—

Unions—Sockets or pipe.

Elbows—Square or round.

Tees—All three pipes can be one diameter or with the stem of different diameters from the cross-piece.

Crosses—With all four pipes of one diameter, or with two pairs of different diameters.

Sockets—Plain or diminishing.

Caps—For closing a pipe end.

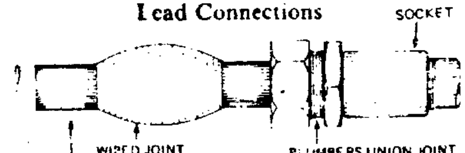
Plugs—For closing a socket-end.

Back-nuts—With long screws to prevent back movement of the socket and leakage.

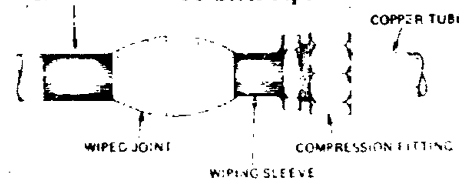
Nipples—For making junctions when a hole is tapped in a main for a branch pipe.

Flanges—For connections to tanks, etc.

Lead Connections



To Steel Pipe



To Copper Pipe

For cast iron pipes the standard specials are : 90°, 45°, 22-1/2°, and 11-1/4° bends, equal and reducing 45° branches tapers and collars. (BSS provide for 5-5/8 bends as well) A 90° bend is also called 1/4 bend, and a 45° bend 1/8 bend and so on.

Change in diameter and in direction should be gradual and not abrupt to avoid undue loss of head.

Fittings to be used in pressure pipes should be of 'medium' or 'heavy' grade.

6. VALVES, METERS & TAPS

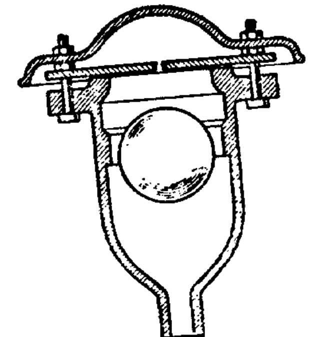
Sluice Valves, Gate Valves, Stop Valves or Shut-off Valves are the commonest and most important as they control the flow of water in a distribution system and are provided at intervals along the mains and all branches and street intersections, one on each line of pipe, so that each portion of the system can be isolated. They facilitate testing the line or shutting off portions of the supply lines for repairs. Valves must be numerous and should be spaced at short intervals in order to cause the minimum dislocation of the service if a portion of the pipe line has to be shut-off. Service mains should be fitted with a valve at each end of a street and not more than 0.4 km if it is a long one, and branches at 300 to 350 metres. Whenever a small pipe branches from a large one, the former is provided with a valve. A stop-cock or sluice valve is fitted on the outlet pipe from a reservoir. Sluice valves are used on cast iron mains and screw-down stop-cocks on wrought iron pipes. Valves on the "run" of large mains should normally be 0.8 to 1.6 km apart.

The normal size of a valve is generally between 1/2 to 3/4 of the pipe, the connection being made by reducers on either side—there is some loss of head. Generally 24" (or 600 mm) gate valve may be used on 30" (or 750 mm) and 36" (or 900 mm) pipes, 30" (or 750 mm) on 42" (or 1000 mm) and 48" (or 1200 mm) pipes and 36" (or 900 mm) gate valves on 60" (or 1500 mm) and 72" (or 1800 mm) pipes. Where head or elevation is not available, gate valve should be larger than indicated above. Large gate valves on high pressure line should have a by-pass to equalize the pressure on both sides of the gate before opening it. A small gate on the by-pass is first opened to equalize the pressure in the pipe on either side of the gate before the main gate is opened. This allows the main gate to be opened with less effort than would otherwise be required. The following sizes of by-pass are usually recommended :—

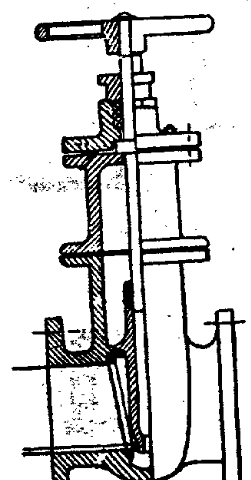
Valve size By-pass	up to 8" 1/2" to 3/4"	9" to 12" 1" to 1-1/4"	14" to 21" to 36" to 48" 2" to 3" to 4" to 6"

Valve size in mm By-pass in mm	up to 200 15 to 20	225 to 300 25 to 32	350 to 550 to 900 to 1200 50 to 80 to 100 to 150

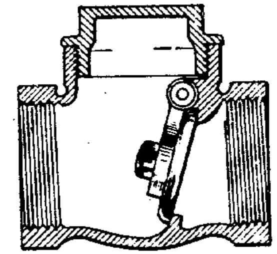
Gears are fixed on large gate valves and those under heavy pressure.



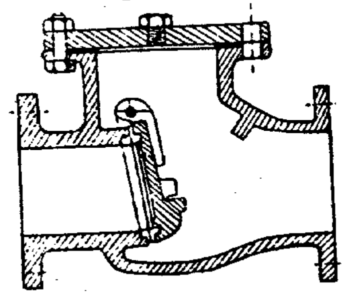
Air-Relief Valve

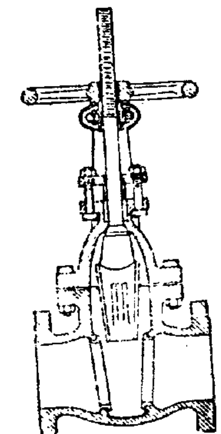


Sluice Valve

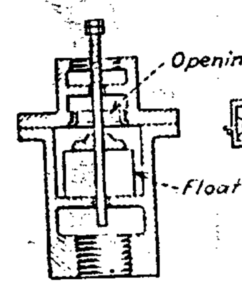


Swing Check Valve



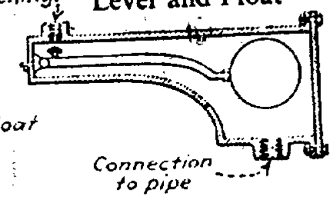


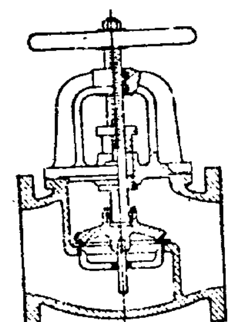
Gate Valve



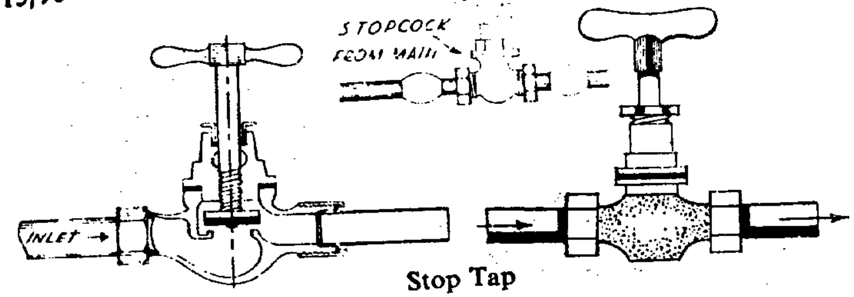
Automatic Poppet Air Valve

Lever and Float

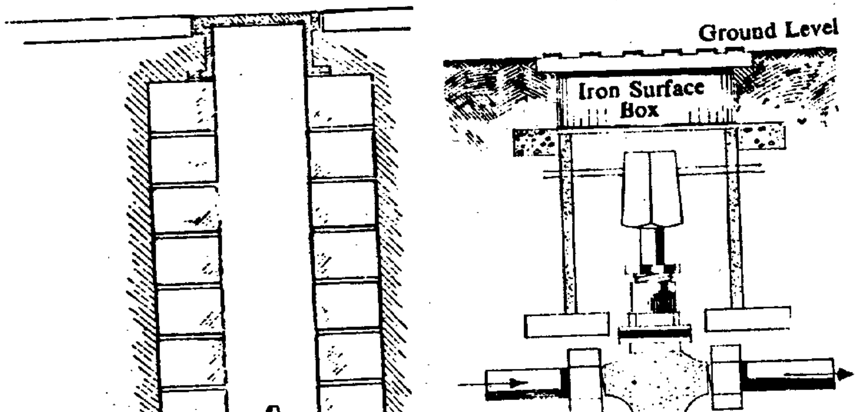




Globe Valve
To control rate of fl

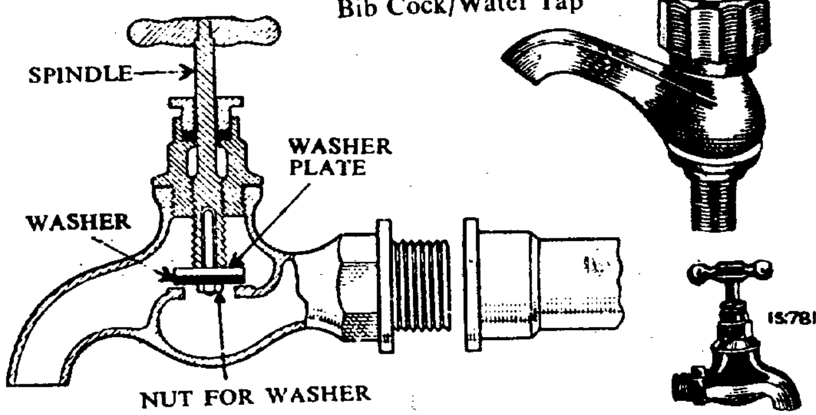


Stop Tap



Stop Tap Placed
Underground

Bib Cock/Water Tap



Each valve should be provided with a cast iron valve box, or a pit built around it ; generally over 300 mm size are placed in masonry pits. It is highly desirable that all valves in a system shut off in the same direction or confusion and damage will result. It is preferable to mark the direction of rotation on the valve box. Valves should be operated to some extent periodically so that they do not jam.

Reflux Valves, Check Valves, Non-return Valves, and Flap Valves. These valves are of many types, which open only in the direction of the flow and automatically close when a burst in the main occurs and the flow reverses its direction. These valves are placed at intervals (say 300 metres) in long pumping mains to prevent back pressure on the engine, on the rising mains at the foot of long upward inclines, in force mains just beyond the pumps to cut off automatically the back flow when the pumps stop running, at the bottom of the rising leg of an inverted syphon, and at points where a breakage would mean great loss of water by backward flow such as at the entrance to a reservoir. *Foot Valves* are similar to reflux valves but are usually placed in a vertical position on the end of suction lines of pumps. A *relief valve* must be placed at the pumping station on the delivery pipe to save the pumps from damage or strain in case of sudden stoppage of water flow due to valves being shut on the line or any obstruction, with consequent water hammer. With two or more pumps discharging into the same main, each of them should be provided with a non-return and an air-relief valve.

Air Valves or Air-Relief Valves are fixed on trunk and secondary mains at the highest points (wash-outs or scour valves are fixed at the lowest points between summits), on undulating water mains, on long stretches of nearly level mains, and at summits of all changes of gradient. Their function is to allow the accumulated air to escape when the pipe is filled and to permit the air to enter when the pipe is empetied. Air and vacuum-relief valves are essential on large diameter steel pipes which would collapse if subjected to a vacuum on the inside.

Such valves need not be fixed on service mains as entrapped air is released through the service pipes. When the pipes rise above hydraulic gradient it is useless to fix an air valve.

Air valves consist of a floating ball in a chamber which allows the air to escape and closes as soon as the air is expelled under pressure of water. At some places "double-acting" valves are used which have a larger and a small chamber with a ball in each. A large-orifice air-valve discharges displaced air when mains are being charged with water, but when air is liable to collect at summits under ordinary conditions of flow, small-orifice air-valves are required : Single air valves are usually provided on mains of 75 mm diameter and double air valves on mains of 100 mm diameter and above. The usual sizes are :

Size of main in mm	up to 100	125 to 200	250 to 300	400 to 500	600
Size of air-valve in mm	40	50	80	100	150

The most common fault in air-valves is the sticking of the ball on its seating, and air-valves on important mains should be controlled by sluice valves so that they can be removed without shutting down the main. A short connection with a stop valve should be given at the highest point and a valve on the main lower down. Air valves are usually bolted on to a standard flanged tee. The common practice in respect of placing of air-valves on long water mains while ascending or descending is at 400 to 800 metres intervals. A galvanised iron pipe is taken from the top of the main pipe on which an air-valve is to be fixed to the side of the road where a suitable site for the valve has been selected. A cock is fixed at its lower end and a screwed plug at the other. The air-valve is fixed in a suitable measonry pit for protection and a weep hole is provided for the escape of water which may pass the air-valve with the escaping air.

Scour Valves or Blow-off Valves (also called *wash-out*) which are ordinary sluice valves provided at the bottom of all depressions and dead ends to drain out the waste water or sediments collected. Scour valves are usually 80 to 150 mm diameter leading from the main from a flanged tee branch to a ditch with a suice valve control. They should be fixed in a manner so that the inverts of the outlets branch and the main are at the same level. In small mains a stop-cock or even a plug will be sufficient. A combined stop-cock and scour valve is frequently used for buildings.

Pressure-Relief or Safety Valves are sometimes fixed at the down-stream ends of long lengths of mains, or where water hammer is likely to occur, to relieve excessive pressure. These are automatic valves that close when pressure becomes excessive on the downstream side. They are heavily weighted spring controlled valves which open under pressures exceeding those for which they are set.

It is important that the correct type and size of valve is selected for the particular line or purpose. Valves are best connected to the mains with flanged joints so that they can be easily removed, repaired and re-inserted without disturbing the rest of the pipe line.

Ball Valves are float operated valves which control the flow of water and prevent over-flow. These valves are made suitable for certain pressures and the one meant for low pressure will leak where a high pressure valve should have been fixed. They are generally made for high pressure (HP valve), low pressure (LP valve) and full-way (FW valve). HP valve are used on storage tanks and flushing cisterns at ground floor level and they will remains closed against a pressure of 105 metres head of water. LP valves are used at a height not exceeding 11 metres above ground level and they will remain closed against a pressure of 35 metres head of water. FW valves are used above 11 metres height above ground level and they will remain closed against a pressure of 7 meters head of water.

Float Valves are used on distribution reservoirs or other tanks and filter beds etc., to maintain constant water levels. The valves are usually of the piston type actuated by means of a lever and float.

Water Taps : brass bib taps (or bib cocks) and stop taps (or stop cocks)

Bib tap is draw-off tap with a horizontal inlet and free outlet. Stop tap is a valve with a suitable means of connection for insertion in a pipe line for controlling or stopping the flow. They shall be of screw down type and shall open in anticlock-wise direction.

Standard sizes : The standard size of a bib tap or stop tap is designated by the nominal bore of the pipe outlet to which the tap is to be fitted.

The following sizes of bore are generally used :—

In inches	3/8"	1/2"	5/8"	3/4"	1"	1-1/4"	1-1/2"	2"
In mm	8	10	15	20	25	32	40	50

All water supply fittings, with the exception of ball taps should be tested for a pressure of 21 kg/sq. cm.

Bath taps for hot water should not be less than 5/8" or 15 mm bore and for all lavatory taps, not less than 3/8" or 10 mm bore.

Washers for cold water fittings are of specially prepared leather or rubber-asbestos composition and for hot water fittings of fibre or rubber-asbestos composition.

The minimum finished weights in kilograms of bib taps and stop taps shall be as follows :—

Bore mm	8	10	15	20	25	32	40	50
Bib taps	0.25	0.30	0.40	0.75	1.25	1.70	2.15	3.65
Stop taps	0.25	0.35	0.40	0.75	1.30	1.80	2.25	3.85

Meters. Meters are used to measure the quantity of water flowing through a pipe-line. A meter should measure accurately all flows through it whether large or small, with minimum loss of head, and should not clog with impurities in the water. There are two main types of meters—"positive" or displacement and "inferential" or velocity types. The ***displacement*** type of meter has a rotating disk or piston which is moved by the flow of water displacing a fixed quantity of water for each revolution and the numbers of displacements or motions are transmitted through a gear to record the amount of water passed on a dial. These types are very accurate and are generally used for small flows as required for domestic purposes, but are costlier than the other type. The ***inferential*** type of meter has disks or vanes like a turbine which are rotated by the flow of water in the meter and measure indirectly the quantity of water passing. The turbine as it rotates turns the hands on the dial. These meters are cheap but not very accurate and are generally used on main lines and for measuring pump discharges, etc. There are two types of dials—the decimal and direct reading.

Meters should be correctly chosen as to the type and size and tested for accuracy before putting them into service and properly set. Head lost through a meter is about 1 to 1.4 kg/sq. cm and this should be taken into consideration while calculating pressure for a distribution system. A meter should be fitted between two stop-valves and with unions. Where a meter has to be fixed underground a small massonry chamber with cast iron surface box with hinged or chained lids and RCC top slab can be built. The inlet and outlet should be in a horizontal position with the dial facing upwards.

Venturi Meters are used for large supplies and are fitted on the mains near the pumping station to ascertain the rate of flow through the pipe system. A venturi meter simply consists of an enlarged end converged to a short parallel throat, which again diverges to the full diameter. The difference in pressure at the inlet and the throat is measured by a pressure gauge. The meter must lie with its longitudinal axis horizontal and should have 10 or 12 diameters of straight pipe upstream of the meter. These meters are not reliable where the velocity of flow is small. Venturi meters work on the principle that as the velocity of flow increases, because of the reduction in dia. of the pipe the pressure falls. After leaving the throat the water loses its velocity and nearly regains all the original head.

Fire Hydrants

Fire hydrants may be located at distance apart of about 90 to 120 metres when the buildings are large and close together or built of inflammable material, and 300 metres or further apart in open areas. Areas of about 4,000 to 10,000 sq. metres are taken per hydrant according to the density of the population and importance of the locality as regards its protection against fire. Hydrants are provided at all street crossings. Water required for fire fighting for Indian towns has been estimated at one litre per head per day.

Fire hydrants are of two types : Pillar or Post Hydrants and Sunk or Flush Hydrants. Post hydrants are not much in use now. Sunk types are fixed below the road surface having a cast iron cover which is flush with the road surface. A vertical pipe is screwed on the sunk hydrant when required and hose pipe fixed on it. Fixing of hydrant should be done on the water main which should be at least of 80 mm dia. (prefer bigger size up to 150 mm dia.) The flanged end of the hydrant should be fixed to the flanged outlet of a tee in the water main. This can also be fixed by means of a flanged tail piece which may be connected to the water main by cast iron specials.

Fire hydrants should be of spindle type with 65 mm (2.5") outlet combind with sluice valve and should be located at street intersections. There are 2, 3 or 4-way hydrants. Common arrangement of hydrants is recommended to be one 115 mm (4.5") connection for fire engine pumper and two 65 mm connections for hose outlet. A min: flow of 900 litres per minute is necessary through a 30 mm 1-1/8" smooth nozzle with a pressure at the base of the tip of 3 kg/sq. cm which is effective to a height of about 21 metres above the ground with a horizontal line not exceeding 18 metres.

Minimum residual pressure heads required for fire hydrants is :—

With pumping engines . . . . 3.5 metres

For direct flow single storey buildings . . . . 20 metres

Residual pressure heads recommended at the fire hydrants for which provision should be made, if practicable, while designing a supply system :—

With pumping engines	10.5 to 14 metres
For direct flow	42 to 53 metres

The hydrant shall consist of the following components :

(i) one sluice valve, (ii) a duck-foot bend, (iii) a 65 mm male coupling instantaneous pattern, (iv) cast iron cap permanently secured to the duck-foot bend by means of a chain.

The flanged end of the hydrant shall be fixed to the flanged outlet of a tee in the water main by means of bolts, nuts and 3 mm rubber insertion or compressed fibre board 1.5 mm thickness. This can also be fixed by means of flanged tail piece which may be connected to the water main by CI specials.

Hydrants are also used for street washing, flushing sewers, watering gardens, etc.

7. LAYING OF PIPE LINES

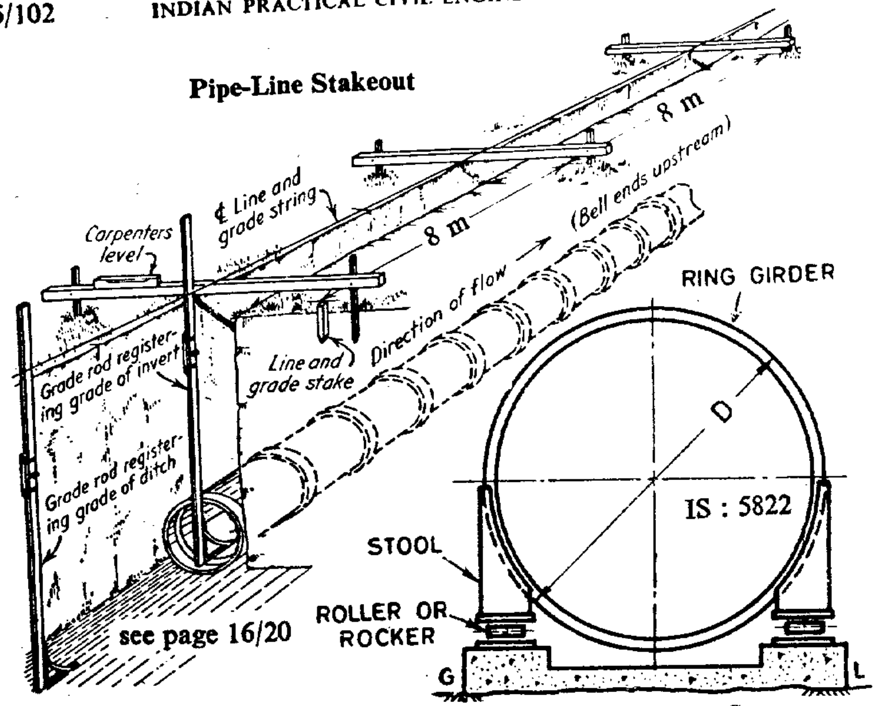
Pipe Trenches. The width of a trench shall be the nominal diameter of the pipe plus 40 cm, with 55 cm minimum in all soils, and which should not be less than 1 metre in case of rock. The trench should be sufficiently wide to allow space for timbering where required. The sides of the trenches should be as vertical as possible. If rock is met with, it shall be removed to 15 cm below the level of the pipe and the trench refilled with excavated materials and consolidated. (Prefer sand to make a cushion under the pipe.) The gradient is set out by means of bonning rods and depths marked. The depth of the trench shall not be less than 1 metre measured from the top of the pipe to the surface of the ground under roads and not less than 0.75 metre elsewhere, to safeguard against the effects of traffic and other superimposed loads. In cold climates, water pipes shall be laid below the frost line which may be as much as 1.8 metres.

Where wide excavation is not permissible such as in roads and streets, shoring will have to be resorted to in deep trenches. Trenches in soft soils require sheet piling and bracings to prevent collapse of the sides. (Also explained in Section 16.)

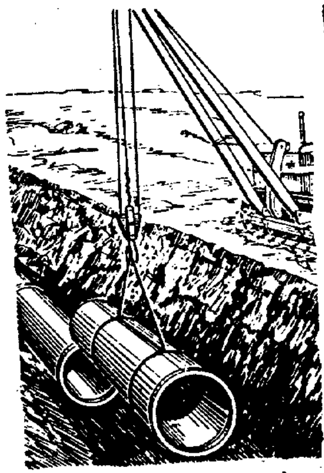
In some grounds where the finished surface of the formation becomes soft after levelling, a firm bottom may be obtained by spreading and compacting an 8-cm layer of gravel or broken stone over the trench bottom, which should be further excavated to receive this. The bed of the trench, if in soft ground or made up earth, should be well watered and rammed before laying the pipe, depressions, if any, shall be filled with earth and consolidated in 20 cm layers. It is important to excavate the trench to the correct width and depth at all points, any extra depths cut out at the bottom of the pipe-line should be made good with cement concrete not weaker than 1:10 (in the case of heavy pipes).

After the excavation of the trench is completed, hollows shall be cut at the required positions to receive the sockets of the pipes and which should be of sufficient depth to allow hands to pass for making joints. The socket holes shall be refilled with sand after jointing the pipe.

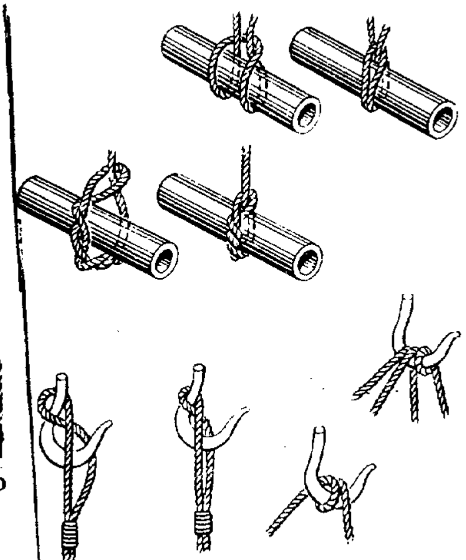
Pipe-Line Stakeout



Roller or Rocker Support



Lowering of pipes into trench with crane



Hoisting Slings and Knots

The excavated material shall be thrown on to one side of the trench and pipes stacked on the other side. The excavated material shall not be placed within one metre or half of the depth of the trench whichever is greater, from the edge of the trench.

Pipe Laying. Pipes up to 250 mm dia. may be handled without mechanical equipment, but in no case the pipes shall be rolled and dropped into the trench. For laying large pipes in position, or small pipes in deep trenches, some form of derrick or tripod with pulley blocks, or mobile crane will be required. Before laying the pipes shall be brushed and cleaned from inside to remove any soil that might have accumalated therein. After lowering, the pipes shall be arranged so that the spigot of one pipe is centered into the socket of the next pipe, and pushed to the full distance it can go. The pipe-line shall be adjusted to the levels required. The pipes shall be laid with sockets facing uphill or in the direction of the flow of water. Where the trench is on a slope, pipe laying should proceed in an uphill direction to facilitate joint making. Wedges are used for keeping a uniform joint space between the socket and the spigot.

Any deviation either in plan or elevation less than 11.25 deg. shall be effected by laying the straight pipes round a flat curve of such radius that minimum thickness of lead at the face of the socket shall not be reduced below 6 mm or the opening between spigot and socket increased beyond 12 mm at any point. A deviation of about 2.25 deg. can be affected at each joint in this way. Pipes of large diameter should be laid with a gradient of 1 in 48 (prefer 1 in 24) to ensure that air travels to the air valves. Work should be commenced from the end furthest from the source of supply and upwards from valves and other specials. Several sections can be laid at the same time. When pipes are laid on brick pillars, the socket should be well clear of masonry to allow the lead joints to be caulked. At the end of each day's work the last pipe laid shall have its open ends securely closed with a wooden plug to prevent entry of water, soil, rats and any other foreign matter into the pipe.

The alignment of the pipe line is checked as follows—a source of light (a candle or lamp) is placed in one manhole and a mirror in the other. If the pipe is properly laid, a true circle will be seen in the mirror. Horizontally, a deviation of up to 1/4 of the diameter of the pipe is permitted, no deviations are permitted vertically.

Water and gas mains should be laid as remote from electrical cables as possible. Water pipes should be laid on the other side of the street where sewage pipes are laid.

Refilling of Trenches. No ashes or clinker should be used for refilling the trenches. The filling should be free from sharp stones, and the filling material should not be thrown from a distance so as not to injure the pipes. In corrosive formations, efforts are often made to counter the risk of corrosion by surrounding the pipes with sand, gravel, chalk or limestone chippings.

Jointing of Pipes

Cast Iron Water Mains. Cast iron pipes are supplied with three types of joints : (i) Spigot and Socket—lead caulked (also called spigot and bell joints) ; (ii) Spigot and Socket—turned and bored ; and (iii) Flanged (bolted).

Spigot and socket lead caulked joint is the most commonly used for underground water mains. This joint lends itself to expansion and contraction under moderate changes of temperature and has certain amount of flexibility which enables the pipes to accommodate themselves to small settlements of the ground which may occur under the pipes. Curves of large radius can be built with straight pipes by deflecting each joint slightly as explained under "Flexibility of Lead Caulked Joints". These joints are not suitable for high pressure hydraulic pipe lines, and are liable to leakage when disturbed by ground movements or vibrations.

Spigot and socket joints in which cement is used as jointing material are rigid joints and semi-rigid or semi-flexible where lead is used. Rigid joints should not be used to connect a newly laid line to an old line as the settlement of the new line will cause a broken pipe.

Jointing of spigot and socket pipes may be done with any one of the following materials : (i) Molten lead (under dry conditions) ; (ii) Lead wool (under wet conditions) ; (iii) Portland cement ; and (iv) **Tarred or asphalted yarn** (for sewers only where considered necessary), mixed with **stone dust or cement.**

For packing threaded joints in pipe lines, use is made of hemp yarn coated with red lead ground with raw linseed oil or with zinc white. The yarn is wound evenly on the pipe end in the direction of thread cutting, after coating thread with red lead. When winding the yarn it is necessary to see that it is the free from breaks, does not extend beyond the pipe end, and does not clog the pipe.

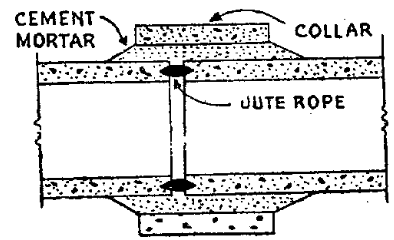
Lead Caulked Joints

The lead used for caulking shall be soft bluish grey pig lead free from admixtures of tin or other impurities. The spun yarn shall be clean hemp and soaked in hot tar or bitumen, cooled and dried before use.

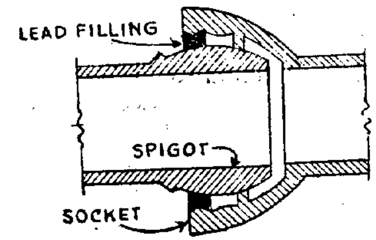
To lead the joints the interior of the socket and the exterior of the spigot must be thoroughly cleaned and dried, or else, the lead will spatter and blow out, and injury may result to the jointer. This method, therefore, requires special care in wet trenches. The spigot end should be inserted into the socket right up to the back of the socket and centered by tightly caulking in sufficient turns of tarred gasket or hemp yarn to leave unfilled half the depth of socket for lead. Two or three laps of spun yarn, twisted into ropes of thickness of at least 10 mm dia., will do. No piece of yarn shall be shorter than the circumference of the pipe. When more than a single strand is required for a joint, each strand shall be cut to sufficient length so that the ends will meet without causing overlap. When gasket or hemp yarn has been caulked tightly home, a jointing ring shall be placed round the barrel and against the faces of the socket.

Molten lead shall be poured in to fill the remainder of the socket. The lead shall then be solidly caulked right round the joint to make up

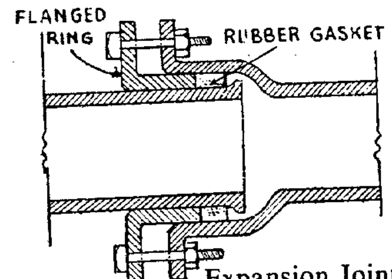
Joint for Cast Iron Pipes



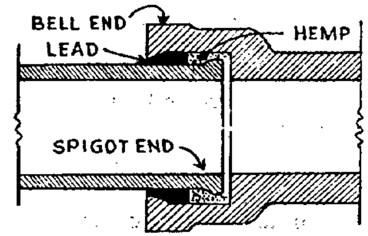
Joint between Hume Pipes



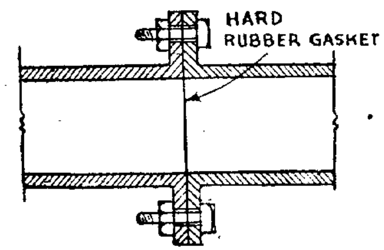
Flexible Joint



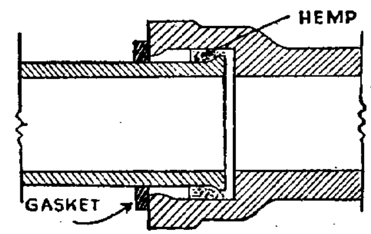
Expansion Joint



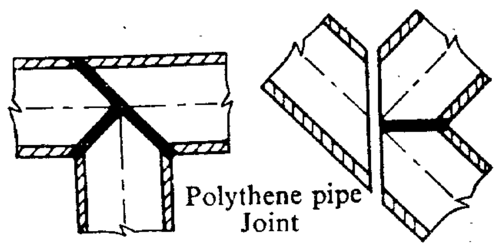
Bell & Spigot Joint



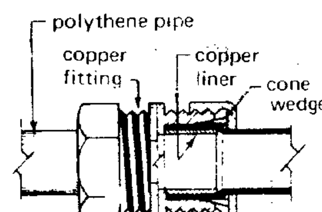
Flanged Joint



Gasket fitted to the bell end for pouring molten lead



Polythene pipe Joint



Polythene pipe Joint

for the shrinkage of the molten metal on cooling. The proper depth of each joint shall be tested before running the lead in by passing completely round it a wooden gauge notched out to the correct depth of lead. (Depth for the lead is about 25 mm to 40 mm according to the diameter of the pipe.) Asbestos rope is also used in place of yarn. Any material used must be free of oil, tar or any greasy substance.

The objection to the use of spun yarn is that in time it rots, tends to become infected with bacteria and which may contaminate the water. It must be disinfected before use.

Lead Caulked Joints with Lead Wool Yarn. This type of lead caulked is generally done when it is inconveient or dangerous to use molten lead for joints for example, in cases such as inverted joints or in wet trenches. The spun yarn shall first be inserted and caulked into the socket as described under jointing with molten lead. Lead wool or lead yarn shall then be introduced in the joint in strings not less then 6 mm thick and the caulking repeated with each turn of lead wool. The whole of the lead wool shall be compressed into a dense mass. The joint shall then be finally finished flush with face of the socket. The lead wool brought to site shall be properly protected and packed with wax paper or polythene sheet, to prevent oxidation.

Heating and Pouring of Lead. Lead shall be heated in a melting pot kept in easy reach of the joint to be poured so that the molten metal will not be chilled in being carried from the melting pot to the joint and shall be brought to a temperature which renders it thoroughly fluid so that when stirred it will show a rapid change in colour. Before pouring, all scum shall be removed, and each joint filled at one pouring. Sometimes a little powdered resin is sprinkled on the molten lead and the latter shaken. The resin acts as a flux and keeps the lead in a liquid form. If the pipe is too large for the joint to be filled from one ladle, two or more ladles can be used. Lead shrinks considerably on solidifying. Joints leak or 'sweat' slightly at first but tighten up in a short time.

The pouring of molten lead in the joints can be done by using proper leading rings of metal or asbestos,' with clamps, or if these are not available, make a wrapper of spun yarn (or a ring of hemp rope) worked up with clay having the consistency of putty. This should be about 80 mm wide and 20 mm thick and 100 mm longer than the circumference of the joint. Wrap this round the joint with the overlap on top leaving a V-shape large hole in it. Lead is poured in one operation only. Strip off the wrapper when lead has hardened and use it for the next joint. For large pipes it is also necessary to leave one or more air vents around the lower half of the joints.

When sealing joints with molten lead, care should be taken to protect them against penetration of moisture, because when moisture gets into liquid lead it spatters and can cause burns.

When lowering the kettle with molten lead into the trench, any worker in the vicinity of the place where the kettle is lowered should move away to a safe distance from it. The kettle with molten lead can be approached only after it is at the bottom of the trench.

Permissible Deflection at Joints. Wherever it is necessary to deflect pipe from a stright line, either in the vertical or horizontal plane, to avoid obstructions or where long radius curves are permitted, the deflection allowed at the joints shall not exceed 2.5 deg.

Cement Joints

The following procedure is recommended :

(a) The joint is first yarned with hemp yarn dipped in cement slurry. The yarn is first inserted to slight depth and well pressed in the same manner as for lead jointing ; (b) Cement mortar of ratio 1:1, with a water cement ratio not exceeding one part of water to 5 parts of cement (by weight) should be rammed into the joint by caulking tools ; (c) The filling to complete and caulked again ; (d) Joints should be kept wet for 24 hours after making. Use of lead joints at intervals is recommended.

Approximate Quantities of Material and Labour Required for Jointing Cast Iron Pipes of Different Diameters, Per 10 Joints

Nominal Dia. of Pipe	Lead	Spun Yarn	Fuel Wood	Kero-sene Oil	Labour		
					Fitter	Asstt. Fitter	Maz-door
mm	kg	kg	kg	litres	Number	Number	Number
80 (3")	19	1.0	19	0.4	1	1	2
100 (4")	25	1.8	29	0.5	1	1	2
125 (5")	30	2.0	38	0.8	1.5	1.5	3
150 (6")	38	2.0	43	0.8	1.5	1.5	3
200 (8")	50	3.0	57	0.9	1.75	1.75	3.5
9 inches	55	3.2	62	1.2	1.75	1.75	3.5
250 (10")	63	3.5	66	1.3	2	2	4
300 (12")	77	4.8	76	1.5	2.25	2.25	4.5
350 (14")	90	6.0	90	1.7	2.5	2.5	4.5
15 inches	98	6.7	100	1.8	2.75	2.75	5.5
400 (16")	110	7.5	110	2.0	3	3	6
450 (18")	130	9.5	125	2.8	3.5	3.5	7
500 (20")	150	10.0	130	2.8	4	4	8
21 inches	160	10.8	140	2.9	4.5	4.5	9
600 (24")	185	12.0	170	3.0	6	6	12
700 (28")	220	13.5	210	3.2	7	7	14
750 (30")	250	14.5	240	3.5	7.5	7.5	15
800 (32")	280	15.3	260	3.8	8	8	16
900 (36")	340	18.8	290	4.5	9	9	18
1000 (40")	410	20.5	350	4.8	10	10	20
1100 (44")	460	24.0	380	5.0	11	11	22
1200 (48")	500	26.0	420	6.0	12	12	24
1500 (60")	660	28.0	500	7.5	14	14	28
1800 (72")	900	31.0	600	8.5	16	16	32

There is about 5 per cent wastage of lead in melting. Quantities of lead and hemp are very variable and also the depth of joints. (The above given figures differ slightly from those given in IS : 3114.)

Weight of lead is 11.35 grams per cu. cm varying 15 per cent depending on purity.

When a section of a few hundred metres has been leaded, the caulking is put in hand. Each gang of caulkers should consist of at least four blacksmiths, all working on the same joint at a time. Lead should be caulked only sufficiently tight with a hand hammer weighting not less than 3 kg as a lead joint excessively caulked will become rigid and prone to leakage. Lead joints should be finished 3 mm inside the face of the socket. When working with lead wool, it is very important to use caulking tools of appropriate thickness to fill the joint space, and to thoroughly consolidate the material from the back to the front of the socket.

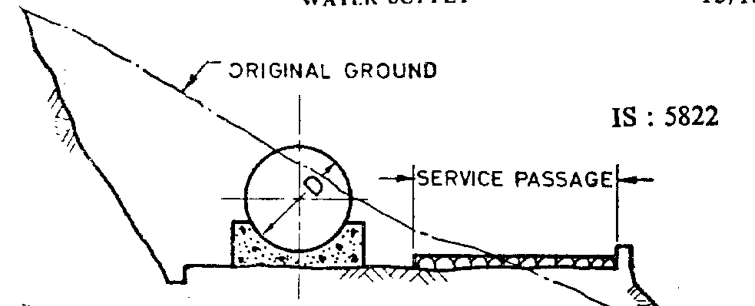
Mixtures of sand or fine stone dust and hot sulpher in equal proportions are sometimes used instead of lead (but that is not as reliable), and the same method and process is followed as for lead caulked joints.

Flexibility of Lead Caulked Joints. Poured lead and lead-wool joints permit some movement for expansion and contraction, and the pipes can also be laid on the sweep round curves the minimum radii of which are approximately : 67 metres for 80 mm pipes, 79 metres for 150 mm and 116 metres for 225 mm pipes. These figures are based on pipes 2.74 metres (9 ft.) long, for pipes 3.66 metres (12 ft.) long, the minimum radii should be increased by 33 per cent. If possible avoid sharp bends such as 90-deg. and 45-deg. and in soft grounds, it is better not to put two bends together but to separate them by a length of straight pipe.

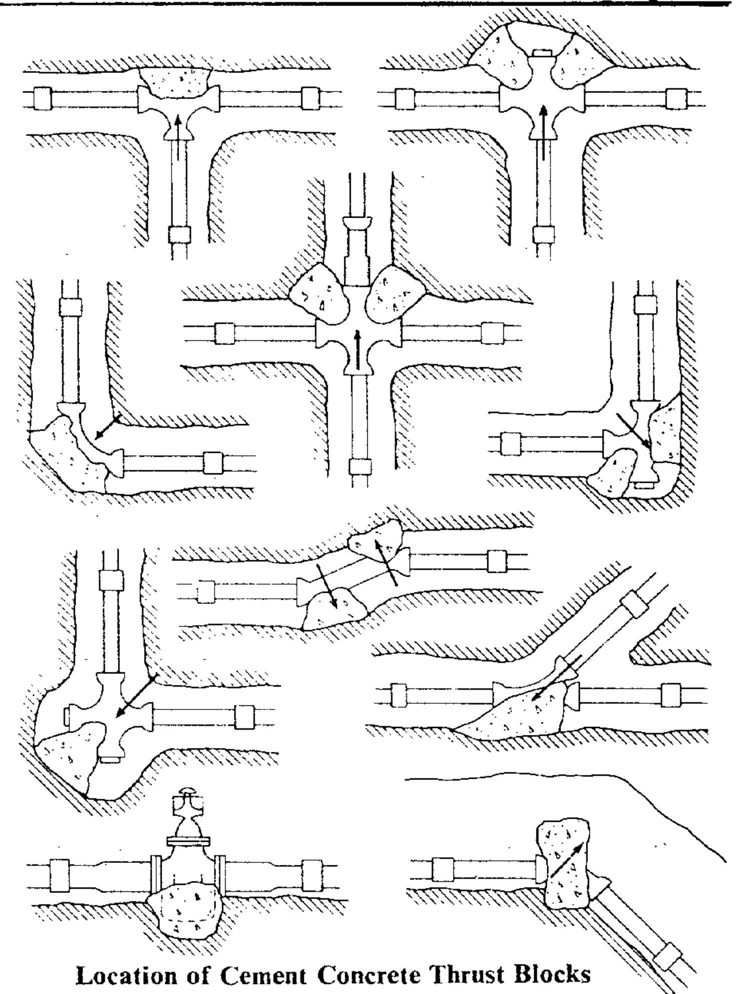
Quantity of Lead Wool or Lead Yarn Required for Different Sizes of Pipes

Nominal Dia. of Pipe	Weight of Lead-wool per Joint	Depth of Lead-wool per Joint	Nominal Dia. of Pipe	Weight of Lead-wool per Joint	Depth of Lead-wool per Joint
mm	kg	kg	mm	kg	kg
80	0.80	19	400	5.70	31
100	0.90	19	450	6.70	32
125	1.25	20	500	8.30	33
150	1.60	23	600	10.0	35
200	2.05	23	700	11.8	36
250	2.95	25	750	13.6	38
300	3.50	25	800	15.4	40
350	4.65	29	900	16.8	40

Flanged Joints. (Also called mechanical or bolted joints.) Pipes which are made with flanges are jointed together with bolts and nuts. Such joints are used in pipes for (i) sluice valves, air valves, hydrants or meter



SECTION IN PARTLY CUTTING AND EMBANKMENT



Location of Cement Concrete Thrust Blocks for Underground Pipes & Fittings

connections which may have to be removed for repairs, (ii) vertical inlet and outlet pipes of reservoirs, (iii) for pipes where inlet pressure is high or resistance to longitudinal force is required as in the case of inverted siphons, (iv) in suction pipes of pumps where a close joint is wanted to prevent air being sucked in, and (v) where there are vibrations. Flanged joints, in general, are widely used for inside work, overhead pipe lines, or for connections in confined or vertical positions, but are generally not employed for underground mains.

Packing ring or gasket (washer) or rubber, leather, compressed fibre board or guttapercha, smeared with graphite paste or mixture of red and white lead is placed between the flanges of both the pipes and nuts tightened in opposite pairs. Size of gasket is 1.5 mm thick for 200 mm pipes and smaller, and 2.5 to 3.0 mm thick for bigger pipes.

In cast iron pipes the flanges are cast as part of the pipe, and in steel pipes flanges are welded-on.

For joints in small diameter wrought iron steel and cast iron piping, copper-alloy screwed unions or ferrules are used.

Resistance to Thrusts. To resist the outward thrust due to water pressure pipes should be firmly fixed near the joints by strong brackets or concrete blocks. Pipe joints do not take tensile forces readily. Thrus blocks are installed on the opposite side of the direction from which thrust is expected whenever the pipe line (i) changes directions, as at tees, crosses and sharp vertical or horizontal bends, (ii) changes size, or (iii) at dead ends. Anchor the valves as well if thrusts due to high pressure are expected. Where the hydraulic thrust is in an upward direction, anchor-blocks of sufficient weight should be provided to which the pipes should be secured with steel straps. The thrust blocks are made after the joints have been caulked with lead. On a steep slope the cumulative weight of the pipes may be so great as to injure the joints. To safeguard this the pipes should be anchored at intervals say, every third length of pipe, to masonry pillars. Specify flexible glands for pipe that pass through rigid masonry such as walls or footings.

Direction of Cracks and other Defects in Pipes. All pipes and fittings should be tapped with light hammer (about 2 kg. weight) to sound for any cracks, blow-holes or sand-holes or any other defects. In case of doubt about cracks, confirmation may be obtained by pouring a little paraffin on the inside of the pipe at the suspected spot ; if a crack is present, the paraffin seeps through and shows on the other surface. Also examine the thickness of the metal and see that the bell and spigot is well formed. The damaged portion of the cracked pipe may be cut at a point not less than 15 cm beyond the visible extremity of the crack, with a cutting machine or a diamond pointed chisel. (Also see under "cutting cast iron and stoneware pipes".)

Testing of New Pipe Lines for Leakage

After a section of the pipe line has been laid and jointed, it shall be tested for water-tightness before being covered in. Sections of about

300 metres length and between valves are taken for testing at a time, as the work proceeds. The trench may be partially back-filled except at the joints which shall be left open. One open end of the pipe line is closed by fitting a water-tight expanding plug of which several types are available or a valve or blank flange or cap The pipes are slowly filled with water so that all air is expelled from the pipes through hydrants and blow-offs, by providing a 25 mm inlet with a stopcock and allowed to stand full of water for at least 24 hours, and then tested under pressure. If necessary the pipes should be tapped at high points to expel the air. The test pressure shall be 5 kg/sq. cm or the maximum working pressure plus 50 per cent, whichever is greater. The pressure can be applied by means of a manually operated hand force pump, or hydraulic pressure pump, and the pressure maintained for half an hour. If the trench has been back-filled, the duration of the pressure test shall be at least one hour.

If the section of the pipe line to be tested terminates with a sluice valve, the wedge of the valve shall not be used to retain the water instead the valve shall be temporarily fitted with a blank flange cap, or in the case of a socketed valve, with a plug, and the wedge placed in the open position while testing.

When the joints are made with lead, all such joints showing visible leaks shall be re-caulked until tight. When the joints are made with cement and show seepage or slight leakage, such joints shall be cut out and replaced. Redone joints shall be re tested.

While under-going test pipes are struck with a small hammer to detect any leakage through cracks. All pipes, fittings valves, hydrant, and joints are carefully examined for defects. The pipes shall remain full of water until all tests have been made. When a pipe is laid on a steep gradient, the test should be carried out at the lower end of the pipe.

Leakage tests on cement joints shall be made after at least two weeks and on sulpher compound joints after four to five weeks.

No pipe installation shall be accepted unless the leakage (evaluated on a pressure basis of 10 kg/sq. cm) is less than 230 litres per 24 hours per kilometre per 25 mm dia. of pipe of 3.66 metre length, and proportionate for other lengths of pipes.

Sterilisation of Completed Lines. Before being put into service for domestic use the entire water supply line should be chlorinated with a liquid chlorine solution-hypochlorite of lime (bleaching powder) may be used. The dose should not be less than 50 ppm of available chlorine and the time of contact not less than 8 to 12 hours—a residual of not less than 5 ppm shall be produced in all parts of the line. Some authorities say that sterlisation shall be considered to have been achieved if a chlorine residue of not less than 10 ppm remains in the water after 24 hours' standing. During the chlorination process all valves and accessories shall be operated. The pipe line shall be thoroughly flushed with clean water after the test.

Corrosion of Water Mains

Special Features of Corrosion

When the metal of a pipe wall becomes weak by corrosion, partial or complete fracture of the pipe often takes place as a result of movement of the surrounding soil. The main cause of serious trouble from corrosion of buried pipes is localised corrosion which may result in "pitting". Pitting may lead to perforation of the pipe within a short time because the pipe might burst before the wall has been fully perforated by pitting owing to earth movements from internal water pressure. The thickness of the pipe wall has a direct bearing on the life of a pipe; the thicker the wall, the longer is the life in the same enviroments. Corrosion trouble has been more marked in pipes of 9-in. dia. and under. Leaky joints, undected pipe fractures, alternate dryness and wetness of the site, and inadequate trench drainage accelerate corrosion. Presence of deleterios salts in the soil in which pipes are laid or saline sub-soil water is often found to be highly corrosive. Acidic and soft waters rapidily attack wrought iron and steel pipes. (Also see under-"Pipes of Different Metals"—Cast Iron Pipes) Damaged coating or presence of pin holes in the protective coating are an important cause of corrosion, which is generally due to rough handling of the pipes. Cutting of screw threads on galvanized iron pipes breaks the zinc coating and in certain soils leaves the piping very prone to attack from outside. Corrosion in iron tanks is due to debris collected at the bottom, and drilling of holes.

Remedial Measures Against Corrosion of Pipes: Corrosion is due to either corrosive sub-soil in which the pipes are buried or corrosive water flowing through the pipes. A survey should be made of the route of the pipe line as regards the corrosive nature of the soil and endeavours made to determine the most favourable locations. Provide protective coatings of a greater thickness than normally given. Lining pipes with cement or bitumen provides an adequate remedy. Flexible joints are better than rigid joints in soils subject to settlements. Corrosiveness of water is controlled by adding lime or soda ash to raise the pH value as explained under "Water Softening". Corrosion is not uniform throughout the pipe but rather occurs in spots and produces "tubercles" of iron oxide that project into the waterway and reduce the carrying capacity of the pipe.

Storage of cast iron pipes over long periods should be avoided whenever possible as both tar and bitumen coatings deteriorate in sunlight, rain or intense cold. When such storage is unavoidable, pipe stacks should be covered over as far as practicable.

No supply pipe or distributing pipe of wrought iron or steel should be laid under the ground unless it is properly protected from corrosion. This can be done by covering with earthenware pipes properly jointed, or hessian wrapping impregnated with bitumen may be used. Pipes can also be laid surrounded by sand on a bitumen filled wooden trough.

Cleaning of Pipes of Distribution System

The pipes are first flushed through washout valves, hydrants, air

valves or by opening a pipe joint and then inserting bleaching powder at the rate of :

0.113 kg for 80 mm (3″) pipe	1.135 kg for 225 mm (9″) pipe
0.227 kg for 100 mm (4″) pipe	2.043 kg for 300 mm (12″) pipe
0 454 kg for 150 mm (6″) pipe	4.082 kg for 450 mm (18″) pipe

for every 90 metre length of pipe.

The water is then run through until the chlorine in solution comes out at the far end, which can be detected by its smell. As soon as smell is felt, the pipe should be shut and allowed to stand full of the solution for at least 12 hours at the end of which time it is flushed out until all traces of chlorine have disappeared.

Scraping of Pipes. Although scraping of pipes increases their carriage capacity but the scraped pipes tend to become encrusted again more rapidly than before owing to the damage caused to the coating. The water is discoloured for sometime after the scraping has been done. If scraping is undertaken, either the pipes should be lined afterwards or scraping done again at frequent intervals.

8. PUMPING WATER

The diameter of pipes through which water is pumped is of great importance especially for long lengths. If pipes of too small diameter are used, the power required may be considerably increased, and if the diameter is too large, there may be wastage of power. (In small size pipes "head lost" is very much increased for the same flow).

Pumping is generally done for about 15 to 19 hours in a day where much capacity for storage is not available, Therefore, pumps must be capable of pumping the whole supply in that time and 25 per cent extra for seasonal variations. Total storage required to balance fluctuations in the distribution demand may be taken equivalent to 6 to 8 hours supply. As already described, the max: demand may be three time the average hourly demand especially during summer months, therefore in cases where the pumped water is let out directly into the mains without an intermediate service reservoir, pumping capacity should be three times the average demand per hour. Where pumping is done only for 10 hours in a day, and service reservoir of adequate capacity is provided, the capacity of the pumps should be four times the average hourly demand.

Calculating Horse Power for Water Pumps

1 British horse power is taken=550 ft.-lbs/sec. (That is, 550 lbs. lifted 1 ft. in one second)

1 Metric horse power is taken=75 kg-m/sec. (That is, 75 kilograms lifted 1 metre in one second)

1 Metric h.p.=0.73 kilowatts

Brake h. p.= $\frac{Q \times w \times H}{75 \times \text{pump efficiency}}$ which is the h.p. absorbed by the pump.

The efficiency of a pump is the ratio of the power output to the power input. If a direct-coupled electric motor is used, approximately 10 per cent should be added for the transmission losses, and if a diesel engine is used to drive the pump, then about 35 per cent will be necessary to allow for the extra losses. Therefore the *pump h.p.* which is the net h.p. actually required may be about 1.5 times of the indicated horse power.

Q is water to be raised by the pump in cu. metres per second ;

w is weight of water per cu. metre = 1000 kilograms ;

H is the total pump head through which water is to be raised. (Static head plus head lost due to friction in both suction and delivery pipes—including losses in bends, valves, sudden contractions and expansion etc.—For losses due to friction etc., add one-third to two-thirds of the static head).

Economic diameters of pumping mains. The following economic diameters of pumping mains for 16 to 24 hours pumping daily are usually recommended :—

Litres/hour (in thousands)	27	54	109	163	215	295	445	590	900
Bore of pipe in mm	150	225	250	300	350	400	500	600	700

The velocity for economic pumping should lie between 85 to 135 cm per second. Choosing too small a diameter is more uneconomical than erring by choosing too large a diameter.

Slip of the Pump : There is always some leakage of water past the valves and pistons of every pump which reduces the discharge below the theoretical amount.

Static Head : Is the vertical height through which water is raised—measured from the water level in the well to the delivery point.

Static Delivery Head : Is the vertical distance from the water level at delivery to the pump centre.

Static Spring Water Level : Is the level to which the water rises in the tube-well when the well is not being worked.

Critical Velocity : Is the maximum velocity at which water can be drawn from a well without disturbing the sand in the medium. If water is withdrawn at a velocity greater than the critical velocity for that particular strata the well tends to sink and is liable to collapse.

Discharge head is the vertical distance from the centre of the pump to the centre of the discharge outlet where the water is delivered.

Suction Lift or Suction Head is the vertical distance from the level of the water supply to the centre of the pump. To this should be added the "draw-down" to arrive at the total *working lift.*

The pressure of the atmosphere will support a column of water about

34 ft. (10.36 metres) high at sea level and it, therefore, follows that the maximum height to which water can be theoretically lifted by creating a perfect vacuum in the suction pipe is 10.36 metres. Pumps lift water by generating a partial vacuum which permits the atmospheric pressure to force the water up the suction pipe. It is not, however, practically possible for the pumps to raise water through that height as the pumps can never create a perfect vacuum, because of friction losses. Therefore, a pump is not used for a total suction height of more than 7.31 metres and which too generally varies from 4.57 to 6 metres according to the make of the pump. For design purposes it is usual to assume 4.88 metres for centrifugal pumps and 6 metres for reciprocating pumps. Suction lift is reduced approximately 30 cm for each 300 metres of altitude above sea level owing to reduced atmospheric pressure. Increase of temperature also reduces suction lift.

Pumps should be placed as low as practicable near the source of supply and may be in pits to minimise the length of the suction pipe so that allowable suction lift is not exceeded and best suction conditions are obtained ; motors can be fixed high up connected by vertical shafts to reduce the danger of flooding. When the water is more than 7.31 metres below ground (or pit) level a pump of the "deep well" type has to be installed. Suction lift should, as far as possible, be avoided. Where suction lift must be provided provision has to be made for priming the pumps, or vacuum pumps installed to exhaust air from the suction lines.

Suction and Delivery Pipes. No point of suction line should be at a higher level than the suction eye of the impeller, as this will form air pockets. (This may, however, be permitted where a vacuum pump is used.) Horizontal suction line is practical up to 300 metres length and the suction pipe should be given a gradual rise of at least 1 in 180 (towards the pump) to prevent air pockets forming which seriously interfere with the flow of water. The suction line should be made perfectly airtight paying particular attention to all the joints and it should be so arranged that any air in it can be easily let off through suitably placed valves. The end of suction pipe should be submerged in water at least three times its diameter to prevent admission of air.

Suction pipes are usually larger than delivery pipes. Usual practice is to have the suction pipe at least one commercial size larger than suction connection size of the pump ; the pump is connected to suction and discharge lines by means of reducers. The diameter of the suction pipe is 7 mm bigger for 25 mm pipe, and about 10 mm bigger for 32 mm to 80 mm pipe than the delivery pipe and for 100 mm and above, the size of the suction and delivery pipe is generally the same. On long suction lines the pump should be driven at slower speed and should have a vacuum chamber to assist the suction and prevent pounding. Both suction and delivery pipes should be free from bends as practicable.

In order to keep the water in the pump whilst standing and to facilitate priming a foot-valve is necessary which should have an area two or more times the area of the suction pipe, depending upon how much foreign matter is carried in the water. A foot-valve should be used at the supply end of an horizontal suction line.

A strainer has generally to be fitted with the foot-valve at the end of the suction pipe (in the case of river pumping) to prevent entrance of objects which may cause stoppage or injury to the pump, and which should be at least 60 cm away from the sides and bottom of the tank or well. The strainer should have a total area of holes at least equal to four times the area of the suction pipe. In the case of the tube-wells where the strainer tube is directly connected with an horizontal centrifugal pump, foot valve is fitted between the strainer and the pump (except in the case of very small pipes) to hold priming water in the pump. Where no strainer is necessary, the suction pipe should preferably have bell-mouth entry.

A shut-off valve and check valve is placed on each discharge pipe to prevent backward surges of pressure which might injure the pump, should the pump stop. A sudden power failure or pump stoppage may also cause serious water hammer and pipe failure. On the outlet side of the check valve a sluice valve is usually placed to permit repairs to the check valve and to allow throttling if the pump is of centrifugal type. The delivery valve must never be closed on a displacement pump (rotary and reciprocating type) or dangerous pressure will arise.

Rapid fluctuations on the vacuum gauge or mercury column indicate air leaks in the piping or pumps.

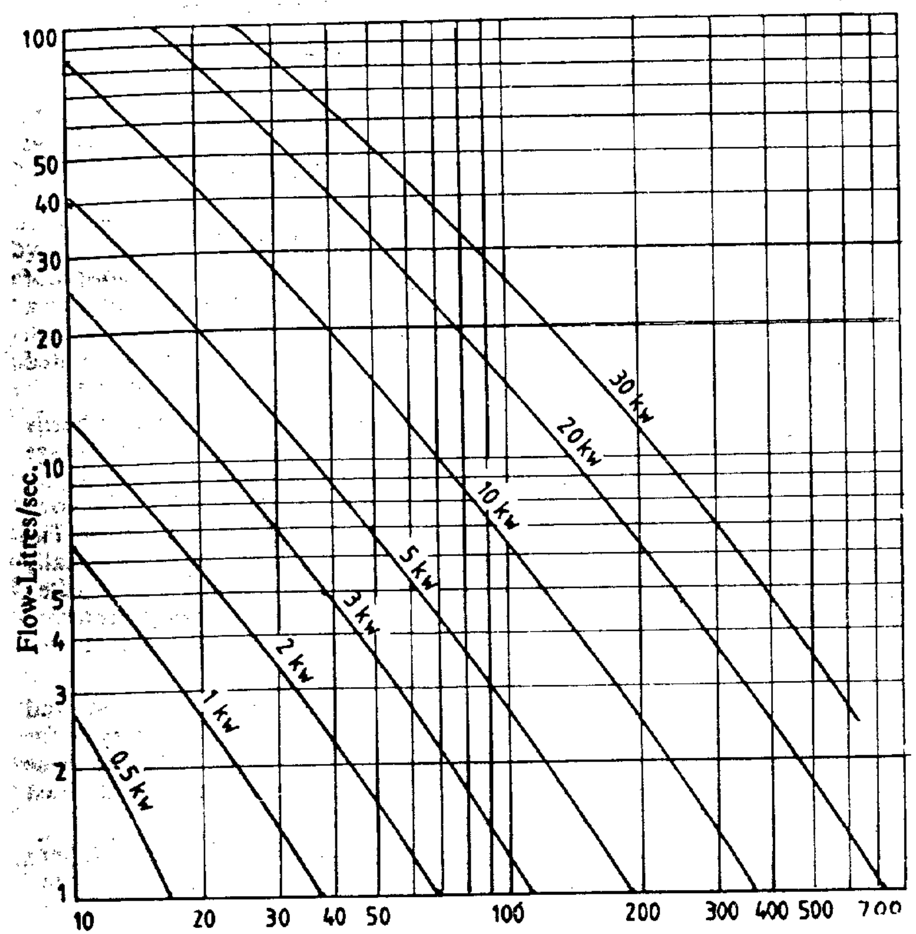
9. WATER PUMPS

Types of Pumps. The pumps are broadly divided into three main types : Reciprocating, Rotary and Centrifugal. In the reciprocating type a piston (or plunger) alternatively draws water into the cylinder on the intake stroke and then forces it out on the discharge stroke. Reciprocating pumps are used for the highest pressure and the smallest quantities.

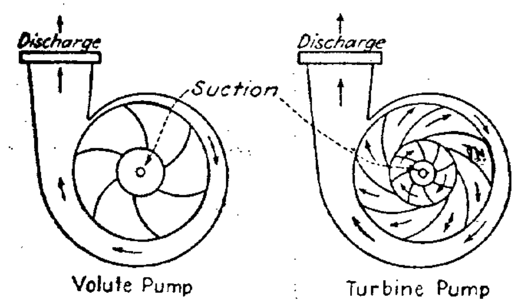
In the rotary type two rotating pistons or gears mesh together and draw water into the chamber and force it practically continuously into the discharging pipe. Rotary pumps are used for small quantities and medium pressures.

The centrifugal pump has an impeller with racial vanes rotating swiftly to draw water into the centre of the pump and discharge it by centrifugal force. Centrifugal pumps are used for almost all quantities and pressures.

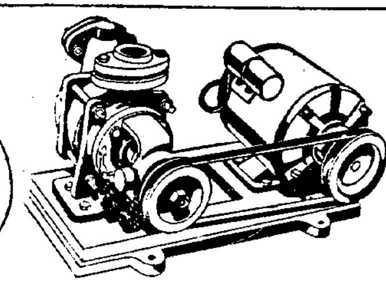
Reciprocating and rotary pumps are classified as 'Displacement' type of pumps, and centrifugal pumps as 'Velocity' pumps. Pumps having separate prime-movers are called *power pumps* and where the prime-movers (steam, gasoline or diesel engines) are incorporated with the pumps and form into single units, these are known as *pumping engines*. Direct coupled pumping sets are cheaper than belt operated sets and also require less power for operation. High-speed pumps and motors usually cost less than low-speed units, but low-speed units are quieter in operation with less vibrations and longer life.



KILOWATT DEMAND OF PUMPS REQUIRED TO PROVIDE VARIOUS FLOWS AT DIFFERENT PUMPING HEADS



Centrifugal Pumps



Electric Motor Pump

Pumps for Civil Engineering Projects

High-head, self-priming centrifugal pumps for feeding concrete mixers and jet drilling holes ; diaphragm and plunger pumps for sludge and sewage pumping ; low-head models for dewatering purposes.

Power for Working Pumps

Choice for Prime Mover

(*a*) *Steam Engine* : Steam power is more reliable and more fool-proof than any other engine power, but it is very clumsy to construct, requires a lot of space, has innumerable moving parts requiring replacement at high cost, needs constant attention and is suitable only for large plants located where fuel is cheap. Therefore, it has become now almost out of date.

(*b*) *The Diesel Engine* : Is an internal-combustion engine which burns low-grade fuel oil (diesel or kerosene). The diesel plant has the advantage of being completely self-contained, and is more efficient than any other type of engine. It is quite reliable and takes up the load atonce. It is, however, rather expensive in first cost and requires skilled attention. The internal combustion engines are noisy in operation and there is usually considerable vibrations for which a special isolative foundation is often necessary. Steam engines and turbines usually run more quietly. Internal combustion engines are not suitable for use with loads with wide fluctuations.

(*c*) *The Gasoline Engine* : Because of their high operating costs and low first costs and ease of starting they are generally used as stand-by or for emergency service with electric power motors. Provision has to be made for starting. The gasoline engine chosen should have a capacity of at least 25 per cent in excess of the need.

(*d*) *Electric Power* : Where available is the cheapest, efficiency is very high and the plant is very compact and is put on or off in a movement. Electric motors are silent in working, free from nuisance of smoke and occupy very little space.

The squirrel-cage induction motor is the type most widely used for driving pumps. AC motors are more efficient than DC but are not suitable for installations in which the speed of operation varies. Care should be taken in making selections that the mortor is large enough to avoid overload and not too large that power is wasted. As there is always possibility of failure of the electric power, a Dietel or Gasoline plant should be kept as a stand-by for emergency.

Fuel Consumption of Engines/Motors per Horse-Power per Hour : (approx.)

Steam	Diesel	Petrol	Electric
2 kg coal	1/4 litre	1/3 litre	1 unit

Lubricating oil consumption is about 1/10 to 1/15 of the fuel consumption.

Pumping Machinery Efficiency

Electric motors : 93 to 95% ; Diesel engines : 70 to 80% ; Steam engines : 60 to 70%. Larger pumps are more efficient.

Centrifugal Pumps

A centrifugal pump consists of a rotating impeller or runner (which has a number of spiral vanes) keyed on to a central shaft which is driven by a power unit housed in a circular casing ; water is admitted at the centre and discharged at the periphery. It utilizes the centrifugal force imparted to the water by the rapidly rotating runner. The impeller rotates rapidly sucking in water at the centre of the runner (or eye) through suction pipe, and which is whirled out and discharged at its outer periphery by centrifugal force. By passing the water in stages successively through a series of impellers any desired lift can be obtained.

A single-stage pump is one in which the total head is developed by one impeller. *A multi-stage* pump has two or more impellers acting in series in one casing and on one shaft in order to generate a greater head than can be obtained from a single-stage pump. The shaft may be vertical or horizontal and the pumps are correspondingly known as virtical or horizontal pumps. Horizontal pumps are cheaper, easily maintained and are more popular.

A centrifugal pump is relatively low in initial cost and also maintenance, compact, simple in construction and simple to operate, light in weight, occupies small floor space for its foundation and is very dependable and durable. It starts quickly and is suitable for both electric and steam turbine drive. The rotor may be driven by belt directly coupled to an electric motor. A centrifugal pump is capable of delivering large quantity of water as compared to its size and can handle sandy, gritty and muddy waters without injuring the pump The clearance between the impeller and the casing are varied to suit the conditions for which the pump is to be made. For clear water pumping the clearances are kept as fine as possible as that gives maximum efficiency. Numerous variations in designs have been developed to meet the great variety of pumping conditions encountered in practice.

In a centrifugal pump the volume of water that can be handled or pumped and the head or pressure developed depends upon the speed of the impeller and the width of the blades and the clearance between the blades and the casing. The horse-power absorbed is proportional to the cube of the speed. High lifts can be obtained with small pumps running at high speeds or with large pumps running at slower speeds. A centrifugal pump works at a maximum efficiency if the head does not fluctuate. These pumps have efficiencies of 55 to 85 per cent, the large pumps being more efficient. Single-stage centrifugal pumps are used for largest quantities against low and medium heads. For high heads and relatively small volumes, either high speed or multi-stage pumps have to be used. Multi-stage pumps are much used in deep wells where the impeller diameter must necessarily be small for submerged operations.

Ordinary single-stage centrifugal pumps have lifts limited to 18 metres, depending on the speed of the motor, (but generally 12 metres is estimated). Multi-stage pumps are used for high lifts, usually the head is increased 30 to 45 metres per stage. Special design single-stage pumps can be had for about 36 metres lift. Placing appropriate number of stages side by side in the pump casing they can be made for lifts even up to 300 metres. Where there is rubbish in water, small turbine pumps are liable to be blocked, at such places the pump should be of the single-impeller type or, if the speed is not suitable, of the three-throw type.

A *foot valve* is fixed at the foot of the suction pipe. It is a one way check valve, its function being to admit water into the suction pipe but prevents it flowing back. Where the pump has been out of operation for long periods it may not work for air may have accumulated in the pump casing ; in that case open all the air-cocks and prime the pump with water. Where the pump is placed below water level, no priming is required and no foot-valve need be fixed. A self-priming pump is advantageous since the foot-valve can be dispensed with.

While purchasing a centrifugal pump, the head against which it has to operate (total pumping head-suction lift, discharge head etc.) and the discharge required must be accurately specified, and also possible variations in head due to variations in water lavel of the supply and variation in pressure in the discharge pipe system. Pumps are generally selected by standard sizes, brake horse-power and speed. The size of a centrifugal pump is specified by the diameter of the discharge pipe.

Points to be Observed when Starting a Centrifugal Pump

A centrifugal pump should not be run dry but must be "primed" i.e., filled with water, before it is started, or the impeller will merely rotate in the air without drawing in any water. For priming, the delivery sluice valve is opened slightly and also the air-cock which is kept open until all the air has been expelled and the water has started flowing freely when both the air-cook and the delivery sluice valve are closed. Self-priming pumps with automatic priming devices are now available.

Reciprocating Pumps

Reciprocating pumps are displacement class of pumps and are also called *Piston and Plunger type pumps*. In the simplest form of a reciprocating pump, a 'bucket', 'piston', 'plunger' or elastic 'diaphragm' is connected to a crankshaft or driving rod which moves back and forth in a 'cylinder' or 'barrel'. These pumps are single-acting or double-acting. A single-acting pump has an intake and an outlet valve and draws water into the cylinder on one stroke and discharges on the return giving an intermittent supply. A double-acting pump has two sets of intake and outlet valves and water in drawn in and discharged on both up and down strokes. All water is raised by "suction" until it enters the pump barrel after which it is lifted to a higher level. Reciprocating pumps are either lift pumps or force pumps. *Hand-worked* pumps are reciprocating pumps.

It is easier to raise a given quantity of water with double barrel pump than with a single barrel if the capacity of a single barrel is equal to both the double barrels.

Reciprocating type of pumps usually need a slower prime mover than centrifugal pumps. It is better to use a large pump driven at a slower speed.

The diameter of the suction pipe should be half of the pump barrel (and not less) but the delivery pipe should be smaller. The height of the lower valve above water level should not exceed 7.3 metres otherwise the pump will fail to raise water.

Pumps of the reciprocating type are not widely used in water works practice compared to the pumps of the centrifugal rotary type. Reciprocating pumps have the disadvantage of pulsating flow, first high cost, need frequent adjusting, unsuitability for sandy waters, possible damage through sudden stoppage by closed valves or other obstructions. These pumps are complicated with chances of something going wrong frequently. They are heavy and require much more space than centrifugal pumps of equal capacity. All types of reciprocating pumps are noisier than other pumps. But for certain purposes reciprocating pumps are preferable as they are more suitable for high and fluctuating heads and high pressure and low quantity duties.

An air chamber or vessel is placed on the discharge pipe close to the pump, especially in single cylinder pumps, to counteract the shocks and water hammer caused by the pulsations in the flow due to the varying rates of discharge of the pump. A pressure relief valve should be fitted on the delivery side to safeguard against damage to the pump should a valve be closed or a blockage occur in the delivery system whilst the pump is in operation.

When first starting a new pump, or during dry weather, it will be necessary to prime it, *i.e.*, fill it with water, as the valves become hard and dry and do not form air-tight joints. It is also essential to see that the valve in the discharge pipe is open before starting the pump, as otherwise pressure will be built up with possibility of serious damage. Valves are integral parts of reciprocating pumps and are placed in the entrance and exit passage so as to allow water to flow only in one direction.

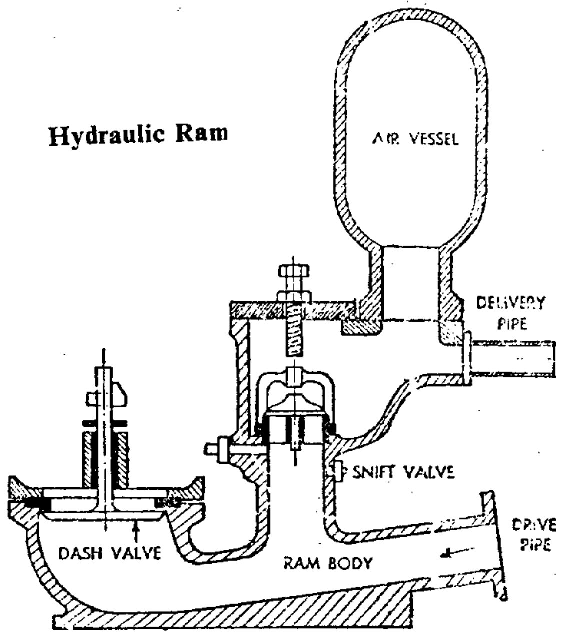
Rotary Pumps

Rotary pumps are also displacement class of pumps and have wide range of pressures and capacities and are particularly suitable for small discharges and moderate heads, and where the volume of the liquid handled is small compared with the pressure required, as in the case of domestic water supplies. Any fluid can be handled provided it is free from grit or abrasive matter which would cause rapid wear. The construction can usually be modified for use with corrosive fluids. They are small in size and simple in operation and will operate with high suction lifts; require no valves ; are self-priming. Rotary pumps are applied to high-vacuum work or pressure up to 7 kg/sq. cm. These pumps should never be started against a closed valve. An air vessel should be fitted on the delivery flange of the pump to cbtain continuous and uniform flow.

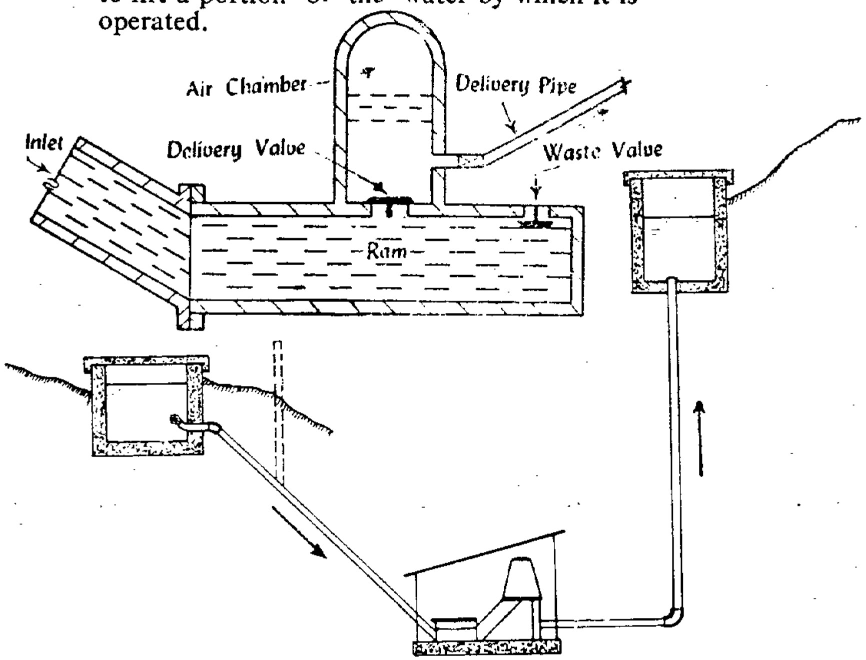
The Pulsometer Steam Pump

Besides ordinary pumping work, semi-liquids and gritty substances

Hydraulic Ram



Single-acting type of hydraulic ram designed to lift a portion of the water by which it is operated.



such as mud, liquid cement slurry, and sewage sludge can be easily pumped. The pump works equally well suspended by a chain and can be worked whilst being lowered and even under water ; can be easily moved from place to place. It is therefore of much use in sinking operations. For general work a suction lift of about 3.5 metres in possible, but may be increased to 6.7 metres in case of emergency. It is especially suitable for temporary works where suction pipe is changed frequently. For sinking works, the Pulsometer patent steam pump has great advantage over ordinary sinking pumps on account of its lightness.

Hydraulic Ram

The hydraulic ram is an impulse pump which utilizes the momentum of falling water. Advantage is taken of a small fall in a running stream to lift water to a great height. The ram is used in small water supply installations where a relatively large amount of water at a moderate head is available in a stream with rapid fall, where the pump can be installed in its bed to pump a small volume to a higher lavel than the supply. Water flows down an inclined pipe (called drive pipe) whose slope should not be less than 1 vertical to 8 norizontal (with 1 to 5 max.), and is generally 1 to 10. It is possible to raise about 1/10th to 1/20th (with 1/6th max.) of the water supplied to the ram, to a height up to about 12 to 20 times the fall obtained in the drive pipe (with 45 metres max: lift). The hydraulic ram is simple, durable, reliable and inexpensive and does not need much attention and works without fuel. The disadvantage being considerable wastage of water and noise of operation. It is rather uncommon to find suitable topographical conditions for the installations of hydraulic rams and are therefore not in much use.

Submersible Pumps (Bore-Hole or Deep Well Turbine Pumps)

The submersible pump is a compact sealed-in unit which is lowered into a bored tube-well casting pipe while the motor is located at ground level and drives a vertical shaft extending down to the pump. This pump is often used for drawing water from drilled wells in which the water level is at a considerable depth below the ground. Deep-well pumps are generally multi-stage centifugal turbing pumps having two or more impellers of relatively small diameter with discharge column extending to the top of the well, and suction piping can be eliminated. These pumps are made for diameters as small as 100 mm, although 150 mm is the recommended minimum, up to 330 mm, and the usual length of the pump is 200 mm to 600 mm. Bore-hole pumps are electrically driven and are quick starting, require no priming, have high efficiencies, produce large volume of water from a small well than any other type, and are very popular in modern well installations. The cost of installation and running of a bore-hole pump is higher than an ordinary pump ; is not suitable for shallow depths of water as the pump requires adequate submergence. see page 15/166

It is desirable that the bowls of bore-hole pumps should be kept at a sufficiently low level so that they remain submerged below water under working conditions. Where full submergence is not possible, the foot-valve should remain submerged, or suction tube may be attached below the foot-valve so that the lower end of this suction tube always remains submerged. Before starting the motor the delivery valve should be opened always about two turns for the air from the rising mains to escape.

It is of utmost importance that the bore of the pump is perfectly vertical and it should never be out of plumb more than the clearance between the inside surface of the well pipe and the outside diameter of the pump bowls. Bore-hole pumps revolve with a high speed and any eccentricity will have great damaging effect. Methods for finding out eccentricity in tube wells has been explained under "Methods of Boring".

Air-lift Pumps

The pump is operated by compressed air and is useful for testing bore-holes and emergency supplies for other sources in which the water-lavel is below the maximum suction lift of 9 metres. Air lift pumps are used to lift water from deep open wells upto 60 metres and sometimes even 150 metres (max.) By mixing the water with air in the discharge, the water is made lighter so that the pressure of the column of air and water in the bottom of eduction pipe (delivery tube) is less than that of the solid water outside in the well, and an upward flow is created. The pump is lowered into the water and compressed air is supplied to it through a rubber tube worked by a small horse-power motor.

The submergence of the rising main (air being supplied there to at the bottom) is about twice the height through which the water is to be lifted.

A stop valve is required on the main supply pipe, a foot valve or non-return valve and a strainer on the suction pipe. A cock is fixed on the suction pipe to allow it to be filled at starting on if it gets empty by chance.

The advantage of an air-lift pump are : It will produce a large volume of water from well of small diameter ; can draw sandy or muddy water and it is possible to pump from a number of wells by using only one compressor. It is very simple arrangement and it is easily operated and maintained. But its discharge cannot be varied according to demands and only limited horizontal pumping is possible.

10. ESSENTIALS FOR DESIGN OF PUMPING STATIONS

Location of the pumping station need careful consideration as it has an important bearing on the pressure to be maintained in the distribution system. A central location of the station will have uniformity of pressure although the sites of elevated storage tanks or reservoirs will also affect the pressure maintained. Large supplies can have a main pumping station as centrally located as circumstances will permit with booster-pump stations where necessary. For small towns, placing the station on one side of the high-use zone and the elevated storage tank on the other gives good pressure.

(*a*) Adjust size of storage tanks to allow uniform rates of pumping for long periods of the day.

(*b*) In order to reduce running costs it is desirable to install pumps of unequal capacities, e.g., one-third and two-third of the total flow desired, which will work according to variations of demand.

(*c*) Provide in duplicate largest pump and motor unit.

(*d*) Provide auxiliary power units, (generally Gasoline or Diesel) to meet emergencies and breakdowns of power.

(*e*) Provide a separate suction intake for each pump if possible. If not, provide tapering header with "Y" branches.

(*f*) Provide flexible coupling to facilities placing and replacing flanged pipe.

(*g*) Provide check valves on discharge side of each pump and gate valves on both sides of each pump.

(*h*) For reciprocating pumps on long suction lines an air chamber near the pump is recommended.

(*i*) It is well to slope the suction lines towards the source of water supply. The slope permits draining the line and avoids air-pockets.

Maintenance of Water Supply Systems

Spare Parts Required : Pipes of each diameter, air valves, hydrants, detachable joints, bends, tees, etc., say half a dozen of each.

For the Pump House : Ropes, shear legs, chains and pulleys for lifting the engine or pump, first-aid kit, fire extinguishers or buckets containing sand.

Maintenance is usually heaviest during the first few years after the laying out. Water mains may settle and develop cracks during the first wet season which is severe test for them.

11. GROUND WATER AND WELLS

Most of the rain-water that falls on the surface of the ground percolates into the pervious soil and moves down by gravity. The accumulation of water under the surface is called *ground water* or *sub-soil water*. The surface of the underground water is called *water table*, *ground water table* or *ground water level* and is the level to which the ground water has saturated the soil. The ground water level fluctuates with rain percolation, temperature, and pumping etc. The water table generally follows the slope of the ground and rises or falls during different seasons depending upon the weather conditions. The water bearing stratum that creates a ground water reservoir and feeds wells or springs is called an *aquifer*. The portion of the soil through which the water moves is called the *zone of saturation*. *Infiltration* or seepage is the percolation of ground water into a pipe or conduit either through cracks, joints or perforations.

Water Divining or "Dowsing". The following indications may lead to the discovery of spring or water near ground level : Certain portions of the earth's surface are more damp, especially in the mornings and evenings, than the surroundings, and from where more dense vapours arise in all seasons of the year. This is due to the presence of underground springs. Where water can be found under ground there the grass is of a brighter colour than in the surrounding fields, especially during the winter months, and earth is also darker and damper on that particular portion than the rest. In summer, the gnats hover in a column and remain always at a certain height above the ground over spot where springs are concealed. Electro-magnetic instruments are sometimes employed for locating springs.

"If you see a palm tree with two crowns, walk a few steps westward and drill a hole. You will strike a huge underground water source at a depth of only 6.8 metres. And if you sight a neem tree with a "knotty" trunk you can be pretty sure of a reservoir in its vicinity at only six metres depth. A termite mound near a "liana" tree is a sure ground water indicator, and so is the presence of a non-poisonous snake or a black frog in the sub-surface soil. One should not look for ground water in the neighbourhood of trees that are "hollow and rough with shattered leaves."

This marvellous science of locating underground water by looking at trees or subsurface features was developed by Varahamihira 1430 years ago.

Water Bearing Strata. Sand and gravels are the most important aquifers. Coarse sand with small gravel and kankar with rounded edges are strong indications of good water contents. The finer the sand the less will its water yield be for the velocity of water cannot be high in fine sand. The supply of water is more reliable in an alluvial soil. Gravel and boulder strata also give good discharge. Clay, although highly porous is so fine grained as to be practically impervious. Clays do not yield water, their permeability being very low but serve as a confining layer to other porous strata. Water can be had from highly fissured rocks but the fissures should be only in the upper layer of rock and the lower layer should be water-tight.

The *porosity* of a formation is the pore space between the particles which make up the formation ; it is the volume of the voids. Particle size has no effect on porosity, but gradation in particle size has a great effect on porosity. Soils with fine separate particle such as clay, top soil and silt, have a very high porsity i.e., they have a big volume of pore space in which water can be stored. The amount of water a material will hold is determined by its porosity. *Permeability* means the ability of the soil to permit passage of water. Materials of the same porosity may have widely different permeability. Permeability is due to small openings between particles. Clays and top soils have low permeability and water passes through them with difficulty.

Measurement of the Velocity of Flow of Ground Water. The rate at which water passes through a water bearing strata depends upon the size of the particles and voids and the head causing the water to flow. The velocity of flow of underground water through porous materials is seldom greater than a few feet per day.

The slope of the water-table and direction of flow can be determined by measuring the elevation of the water in a series of bore-holes. By putting down three test holes at approximately the apices of a right angled

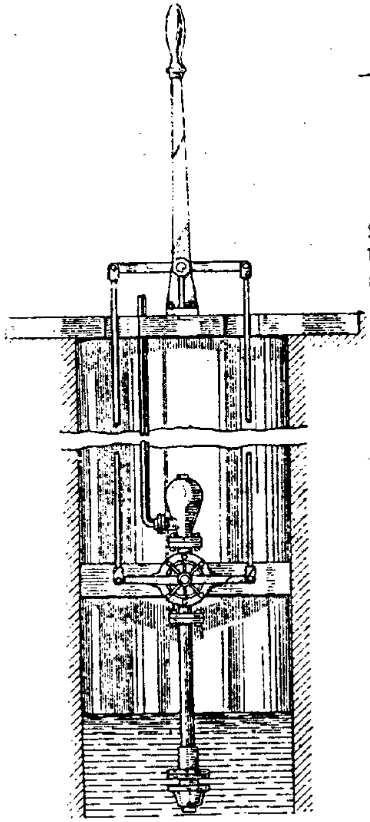
triangle and measuring the elevation of the three water levels, the slope of the hydraulic gradient and the direction of flow can be obtained. With fine material in the stratum the water-table is steep and with coarse material the water-table becomes flatter. In very small size of grains the ground will not yield its water readily. Coarse sands and gravels with steep gradients may have a velocity of 6 to 30 metres per day, fine sands 15 mm per day, whilst in very fine sands with flat gradients and sandstones of fine texture the flow may be only 3 metres per year.

The velocity of ground water may be measured with the help of dyes, salts, or electrical conductivity methods. A solution of salt (usually ammonium chloride) is introduced in the upstream well and the time of arrival in the downsteam well is found by making frequent analysis of the water. Salts can be detected by increase of electrical conductivity in the water. Fluorescent or fluorescein solution is used as the dye. It can be detected in about one in fifty million dilution by the naked eye. Powdered fluorescein has a reddish-brown colour and when dissolved in water the water becomes green. Roughly, the quantity of underground flow can be obtained by multiplying the velocity by the area of the cross-section under consideration.

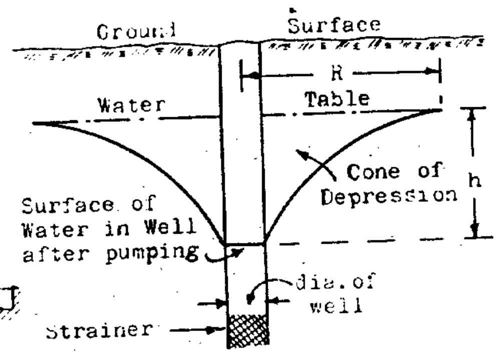
Cone of Depression or Depletion. When water is pumped out from a well the level of the underground water is depressed all-round roughly in the form of an inverted cone (see figure on page 128 the base of the cone coincides with the water-table and the apex with the pumping level. It is commonly called *cone of depression or cone of exhaustion* and the area within which the water-table is appreciably affected is termed the *area circle, or zone of influence*. The distance from the centre line of the well to the edge of the cone is called *radius of influence* or radius of cone of depression (R) of the well. The greater the radius R the greater will be the flow of water into the well. But there is a practical limit for this depending upon the critical velocity that can be allowed through the particular strata for pumping. A reliable method for determing the zone of influence is to observe the standing water level in small diameter observation holes sunk previously for the purpose at known distances.

Depression head is the vertical difference created (by pumping) between the original ground water level (before pumping) and the lowered water level in the well after pumping. Depression head in a well goes on increasing with pumping until the rate of re-cuperation of water equals the rate of pumping. The area of pumping effect for a given quantity pumped is likely to be less in a coarse-textured, well-fissured stratum than in a fine-grained sparsely-fissured rock.

The slope of the curve of the cone of depression (gradient) brought about by pumping depends on the nature of the material and the depth to which water is pumped. In sand the gradient may be on the average from 1 in 50 to 1 in 20, and in chalk as steep as 1 in 10. A gradient of 1 in 10 would mean that the effect of pumping from the well would be perceptible at a distance from the well ten times as great as the difference between the sub-soil water level and the level of the water in the well. Water is more or less completely withdrawn from within the area of the cone of depression,

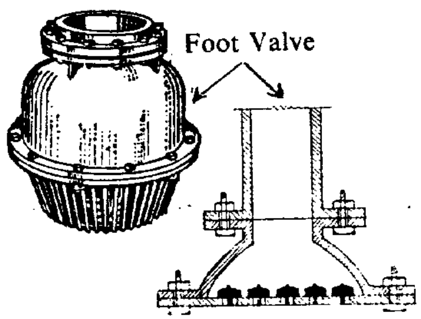


Hand-operated Semi-Rotary Pump fitted to Deep Well



To obtain a uniform continuous water flow, an air vessel should be fitted on the delivery flange of the pump, as shown in the illustration. On long suctions an air vessel fixed to the underside of the pump assists in obtaining smooth action.

Delivery air vessels are always necessary on power operated pumps or on long deliveries.



A tightly closing foot-valve is necessary for suction greater than 2 m. In shallow wells the foot-valve should be at least 15 cm above the bottom, and in deeper wells it should be fitted about 50 cm below the water-level. A sand strainer should be fitted beneath the foot-valve when sandy water is to be pumped.

Foot Valve—Fitted at the lower end of the suction pipe. It is one-way valve and its purpose is to admit water into the suction pipe and to prevent it from flowing back.

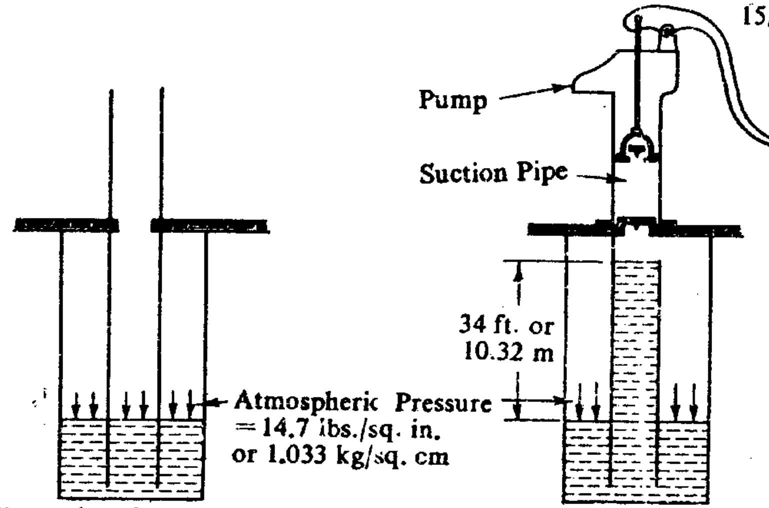
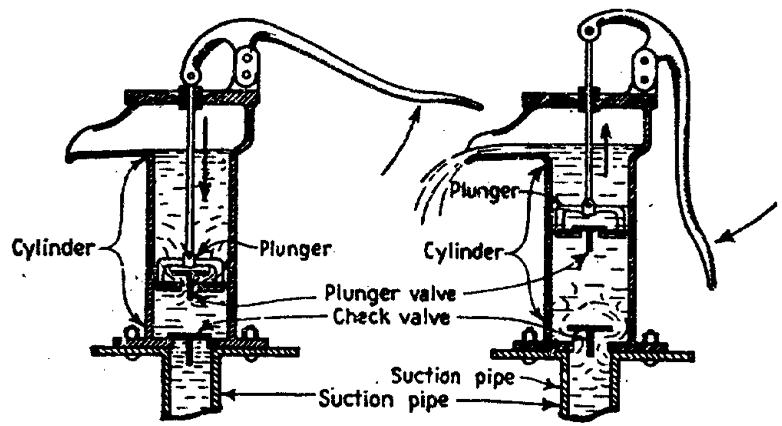


Illustration shows maximum suction lift. When the pressure inside and outside of pipe is equal the water remains at the same level. If, however, a pump be installed as illustrated, water under condition of perfect vacuum may be raised a maximum distance of 10.32 m by suction. Due to the difficulty of establishing of perfect vacuum, however, in actual practice the limit of lift will be somewhere between 6.7 to 7.6 metres.



In operation, suction is created by a piston or plunger working back and forth inside a cylinder. On its downward movement the water below the plunger is forced through a valve in the plunger and into the upper part of the cylinder, the check valve being closed due to water pressure. On the upward movement of the plunger, the plunger valve closes, due to the weight of the water which is at this time flowing through the pump spout by lift action. An additional volume of water will at this time enter the cylinder, the check valve being open by suction.

(which is also called (*cone of exhaustion*). Therefore, it goes to show that any source of pollution existing within this area around the well will affect the water drawn from the well. Any pollution on the upstream side would affect the water, weather inside this area or not ; pollution below the well but outside this area would not sensibly affect the water. Therefore, careful study of the site and soil should be made when selecting a site for a well.

Infiltration head (also called *percolation head, depletion head* or *draw-down*). The head of underground water which makes the water flow or infiltrate through the stratum.

An Infiltration Gallery is a horizontal well which is a sort of tunnel or underground conduit driven into a water bearing stratum which collect the percolating water over practically its entire length.

Wells and Bore-holes

Wells are made for domestic or town water supplies and minor irrigation. With respect to the methods of construction they are classified as open dug wells, bored wells, driven wells and tube wells.

There is no defined demarcation between a "well" and a "bore-hole". A bore-hole may be defined as a shaft of any diameter between 100 mm to 900 mm or more. A drilled well is usually referred to as "bore-hole".

Artesian well is a well penetrated in ground-water confined beneath an impervious stratum under sufficient pressure and from which water rushes up over the surface without pumping. There are not many artesian wells in India.

Intakes are works required for collection of natural surface water. *Jack wells* are intake wells provided at rivers in which water is collected and pumped to the rising mains. Openings provided in the wells for the entry of water are called *penstocks*. Water should be drawn as far away from the inlet as is practicable and about 60 cm below the surface.

Tube-wells are wells of small diameter and consist of tubes or pipes bored into the ground water supplies of one or more water bearing strata. Tube wells are worked mechanically while open wells are worked by human or animal labour.

Discharge from Wells. Open wells generally get their supplies from the first water bearing stratum met after digging at which water stands at spring water level, while in tube wells water can be drawn from the stratum lower down in addition to the top strata and thus large quantities of water can be drawn from a tube-well as compared to an open well.

In the case of shallow wells the water table of shallow ground water which they tap is likely to fluctuate considerably thereby making the yield uncertain. Seasonal variations of water level may be very considerable in some localaties. Drawing water out of shallow wells will lower the water-table and might effect the adjoining structure, but deep wells give large and uniform yield. The ground water level at a site which is heavily drawn by a series of wells, as for a large town supply, usually falls a few

metres permanently shortly after the pumping is regularly established. This permanent drop of ground water level must be taken into account in fixing the level of the pumping engines and suction pipes.

Mutual Interference of Wells. Wells placed in the same stratum near together mutually interfere according to the size and spacing of the wells, the radius of the circle of influence and the lowering of the ground-water table. If wells are placed too close together, pumping water from one well will affect the yield of the others. A second well should be dug away from the circle of influence of the first well, so that the discharge of one well is not affected by draw-off from the other wells. As a rough rule, the area drained by a well may be taken equivalent to that of a circle having a radius equal to the depth of the well. A spacing of 100 to 180 metres between two wells is generally necessary according to the nature of the strata and the depth of the sub-soil water level, and which may be 150 to 300 metres for deep wells.

Maximum discharge can be obtained by placing the wells in line at right angles to the direction of the maximum slope of ground water-table so that almost equal heads in different wells can be secured.

The yield of a well can be tested by "bailer test", "pump test", or by "recuperative test". Reliable information can be obtained in many cases as to the permeability of the strata by the application of certain theoretical formula in conjunction with pumping tests.

Critical velocity is the velocity at which the soil particles (sand) get disturbed and begin to flow into the well. The draw-down at which the critical velocity obtained is known as the *critical head*. Thus *critical yield* of a well is the maximum amount of water that can be drawn at a velocity which does not cause (but exceeding which will cause) grains of sand in the water bearing strata to move. The safe maximum rate at which a well will yield water is called the *capacity of the well.*

The yield of a well is the amount of water which can be steadily withdrawn by pumping when the draw-down has reached an equilibrium and remains constant.

The water level should generally be recorded in the mornings. An indication of the pressure at which water is flowing in a particular strata is given by the level to which water rise in the boring tube. If the level of the water in the boring tube is as high as the spring water level, or higher still, it is an indication of ample supply but if the water stands at a lower level then the surface spring, the pressure (and hence the supply) is poor.

Bailer test is a test conducted of the rate of depression while boring to estimate the probable yield from any water bearing stratum.

Pumping test. When water is pumped out of a well at a steady rate, the water level in the well goes down in the beginning as the pumping progresses ; this is proportional to the rate of pumping and the inflow of the water into the well from the sub-soil. A point is reached when the water does not fall any further with a certain constants rate of pumping. At this stage the rate of pumping equals the rate of inflow into the well

The maximum critical velocity is arrived at which is not exceeded, and the yield at this depressed level is measured which maintains the level constant. This method is rather difficult as it involves steady pumping at a fixed level and an accurate discharge of the pumps. It is essential that the yield of the test-well has stabilized before data is taken for computation.

The Recuperative test. The water surface in the well is depressed by pumping to any desired level below the normal and from this level it is allowed to refill to its normal level. The time of refilling and the amount of water refilled is noted. The rate of filling is more rapid when the head is greater and it become less as the water level rises up.

The observations are to extend, not for uniform period of time but for uniform rises of levels, say for every 8 cm of height till the normal water surface is restored. Conclusion should be drawn from a number of tests extending over a week at least. The yield should be tested at the driest time of the year. The duration of the test depends upon the purpose for which the well or bore-hole has been sunk and may vary from one to 21 days' continuous pumping. A minimum of 14 days' pumping is recommended for work in connection with public water supplies. The yield of a single small diameter bore-hole or well cannot be relied upon to furnish data upon which a reliable estimate can be based for a water supply scheme. The yield of a new well tested soon after completion is usually 10 to 30 per cent greater than the yield at the same rate of pumping a month later. Therefore, the "yield" of a well should be tested about one month after its completion and use. Tests may be conducted as each individual water bearing strata is met with while boring, which can be done by fixing a packing or plug below the level of every individual strata in turn, starting from the top. Tests are usually carried out by submersible type of pumps, though the air-lift type may be used under suitable conditions.

A capacity test should be made on every irrigation well prior to purchase of the pump.

Trial holes. As a preliminary to almost any scheme more than 4.5 million litres per day is required, one or more trial boreholes should be sunk and cores of the underlying strata obtained. If the trial bores are required only to provide details of strata and water levels as an aid to construction, then their diameter need not exceed 150 mm. If, however, information is required as to the probable yield of permanent works, then bores of up to 600 mm in dia. should be sunk. Samples of all changes of strata and at 3 metres intervals throughout the full depth, should be taken and preserved in air-tight bottles if of cohesive soils, otherwise in wooden boxes. Sulphate test should be made on clays, and sands should be mechanically analysed.

Yield from a Well. Maximum yield will be obtained from a well when it is worked continuously without exceeding the critical velocity. If an excessive velocity is used the strata through which water moves will collapse.

A factor of satety of 2 to 4 is usually allowed but it may have to be as high as 10.

In the experiments carried out in the Punjab on the yield of percolation wells, it was found that with ordinary fine sand, the head of depression must not exceed 1.5 metres while in U.P. 5.5 to 6 metres of depression is considered suitable for tube-wells in the average sandy strata. 3.7 metres has been considered to be the safe limit for depression head in alluvial formations of the type of North India. Exact value depends upon the characteristic of the strata in a particular region. No depression head need be considered in gravel or boulders. Critical head must not be exceeded or the sand will come up and be drawn into the pumps. If a strainer tube-well is put down which has very fine spaces between the strainer wires, the sand cannot get through and pumping can be carried on with a very much greater head of depression. Comparing the yield of one 115 mm diameter tube strainer well with that of percolation wells, it is found that one single tube of 115 mm dia. gives the same quantity of water as four ordinary wells each 3 metres in diameter, and at considerably less cost.

The effect of the size of the well on the yield is comparatively small. Comparatively large yields can, on occasions, be obtained from small dia. bore-holes, but unless the water level is within suction lift from the surface they cannot be pumped economically as a really efficient pump cannot be accommodated in small holes. The chief advantages of a well of large diameter are the storage that it affords and the possibility of placing the pumps at a low level and short suction pipes. Large wells are useful where the pumping is variable and especially in cases where the ground water flows through fine material with low velocity.

Yield from a tube well may be taken roughly equal to 50 litres per hour per 1000 sq. cm of the strainer area per 30 cm of depression head. In alluvial soil strainers for tube wells are generally 30 to 45 metres in length located in the water bearing strata. This will give about 2300 to 2700 litres per minute. Open percolation wells with impervious lining of about 3 metres diameter will yield about 250 to 550 litres per minute.

The yield of a well depends upon the transmission constant, the draw-down, diameter, replenishment of the sub-soil, type of strainers used: and also spacing and mutual interference of the wells. The transmission constant depends on the actual pores and not on the percentage pore space, the smaller the pores the lesser the transmission constant. It is thus actual size of the pores which effect the flow.

CONSTRUCTION OF TUBE-WELLS

Drilled Wells. Drilling may be done by percussion or rotary tools or by jetting, and the excavated materials brought up by means of a bailer or hollow drill tool or by some hydraulic process in which water or mud-laden fluid is pumped down the hole and carries up the excavated material as it returns to the surface. Percussion and rotary methods have their own advantages and disadvantages. The percussion ribs are lighter and more portable and need far less water than rotary. On the other hand, the rotary method is normally much quicker, and holes of greater diameter may be made in a shorter time. It is useful for greater depths and for penetrating hard and solid rocks. But rotary rigs are not well adopted for drilling in unconsolidated material.

Drilling methods have great advantage over other methods. Drilled wells tap water from great depths and have large yields and are less affected by draught.

In the case of driven and drilled wells, an annular space at least 4 cm wide, between the well casing and the natural formation is grouted with cement in order to provide a water tight seal to prevent the surface water from reaching the ground water. The depth of the seal should be at least 3 metres below ground surface, or a depth according to the position as regards danger from contamination. The cement grout mixture may consist of 43 kg of cement and 20 litres of clean water to which hydrated lime is added to the extent of 10 per cent of the cement volume.

Core drilling. This method is used for drilling through hard rocks. A core drill bit consists of a ring with small artificial diamonds or chilled steel shots or steel teeth. The ring is attached to a drill rod and rotated and as the cutter advances, a core rises inside the ring which is broken off from time to time and removed. This core is very useful as it gives the representative sample of the materials met with. Water is pumped into the bore-hole to act as a lubricant. In a soft rock, the shot drill is not always satisfactory because the shot often breaks the rock and it then is impossible to secure a good core. A diamond bit will produce a better rock core in soft rock. In hard rock, either type of bit is usually satisfactory.

Water Jets Boring is used for boring through hard and tenacious soluble clays and for deep boring. Drilling is accomplished with the help of a water jet pipe with a nozzle at the end, which is introduced into the casing pipe, and water forced through it under pressure. The water (formed into slurry) returns to the surface through the space between the casing and the jet pipe. This loosens the soil and the casing pipe sinks by its own weight or added weights.

Bored Wells. A bored well is constructed by boring a hole with a hand or machine operated auger and installing casing. This method is suitable for small diameter wells of shallow depths in soft soils devoid of large gravel and stones and which can be easily penetrated. Procedure for boring consists of screwing a hand or power operated (usually hand operated) helical auger (earth borer or earth auger) into the ground and withdrawing it full of soil material which has been caught in the auger, for cleaning. Wells can be bored to considerable depths if the soil is suitable by adding rods to the auger. Small bored wells are made where the water-table is less than 9 metres. These wells are usually 5 to 8 cm in diameter. If the well to be bored is of a bigger diameter, mechanical arrangement is employed and the augers used are different which have sharp tongues (auger bits) and are not of screw type. These augers penetrate short lengths at a time when they are lifted up full of soil and cleaned.

When the soil is so soft that it frequently caves in, casing pipes are lowered as boring proceeds and which are left in position. The wells are bored about 6 to 9 metres into the water bearing strata. Small size pipes with screws or strainers attached are lowered into the hole and pump installed. Where pumps are used for drawing water, casing pipes are 10 to 15 cm in diameter and inner pipes 4 to 6 cm which act as suction pipes. If the stratum is very coarse, the bottom of the inner pipe is left open and fitted with a foot-valve.

In very stable cohesive soils casing pipes may be put up to a short depth to protect the top of the borehole from caving and abrasion and, for protection against infiltration of dirty water.

Rotary Boring. Rotary or churn drills are frequently employed for drilling wells in almost all formations encountered hard or soft. Power driven machines are used with tripod or four legged derrick. Drilling is accomplished by rapidly rotating a pipe fitted with a toothed cutting shoe at the lower end. The two types of drills in common use are diamond drills or shot drills. The drills used in boring vary much in shape, quality and hardness and range from soft steels to diamond drills which will drill through the hardest rock formations. In firm formations either a fish-tail bit or diamond shaped bit is used. Rotary rigs perform the two operations of breaking, loosening, grinding and removal simultaneously. Water is continuously pumped into the well under pressure through holes in the cutting shoe to rise to the surface between the side of the hole and carrying with it the loosened material, all formed into mud or slush. The mud or slush plays an important part in the drilling. The mud keeps the open hole from caving during the drilling process due to its weight and viscosity aided by the plastering action of the rotary drill pipe, and the casing is placed later. In alluvial formations casing pipe has to be put in immediately after the boring is done, otherwise wells may collapse. It is also generally necessary to introduce a colloidal slush at sufficient pressure to make the walls stand, otherwise they collapse when the drill is drawn out. A 200 mm dia. hole may be drilled up to 6 metres in 24 hrs. is soft limestone but in hard formations drilling may take weeks.

Another rotary method uses no water but utilizes compressed air to blow the material to the surface.

Percussion Boring. Drilling is done by the chiselling action of a tool—a heavy drill bit—which is alternatively lifted and dropped repeatedly in a prepared bore hole which pounds the stratum in small fragments and the pulverized material is taken out to the surface by means of a bailer, which is known as "bailing". This method is the most commonly used and can be adopted for all types of strata, but is generally confined to the drilling of soft, fissured or friable rocks and alluvial formations ; not very suitable for loose and unconsolidated soils. The drill is operated by hand or by machines. If any hard strata is met with, the hand precussion drill will not work and mechanical units will have to be used. The hand operated method is convenient up to a depth of about 60 metres. Usual size of percussion boring is 40 to 80 mm hole and about 30 metres of depth. A crab-winch or windlass and a steel rope is used for wells of moderate depths for working the drilling bit inside the boring tube. Tube-wells giving discharges up to 3000 litres per minute have been dug in many places in Northern India in alluvial soils.

A pit is dug at the site about 2 metres in diameter and 4 to 5 metres deep. A casing pipe to the bottom of which cutter shoe is fixed, is lowered into the pit at the site of the well. A tripod or derrick, about 9 metres high, is erected over the pit with a pulley attached at the top so that the drill suspended over the pulley is exactly above the casing pipe. The casing pipe is clamped with cross wooden beams at some convenient point above ground level and a platform made which can be loaded with sand bags or other weights to help forcing down the casing pipe. As the pulverized soil is removed, the casing pipe goes on sinking, and to which more pipes are screwed down. Inserted jointed or flush jointed casing is preferable, as socketed casing is difficult to drive or to jack back. Pipes can also be welded. When drilling in loose unconsolidated formations, the casing must be driven to follow the bit closely, so as to prevent the caving in of the unsupported open hole.

The bailer or sand bucket used with the drilling bit for rocky formations consist of a tube of slightly smaller diameter than that of the casing pipe and 3 to 4.5 metres in length, fitted with a check valve at the bottom. After the soil has been pounded with the jumper bit, the sand pump is dropped in the bore, it fills with the debris and the valve closes as it is raised. Water has to be added during the process for the loose material to form into a "slurry".

Two types of percusston drills are used : For alluival formations driller and bailer combined (called plunger), and for moderately hard rocks, a heavy drilling bit (also called jumper bit) in the form of a flat chisel or cross chisel for breaking and pounding the strata and bailer or sand bucket for removal of the debris from the bore. A plunger or sludger is made of a steel tube about 6 mm thick and of diameter about 50 mm less than the bore of the casing pipe, length about 2 to 3 metres. A cutting shoe of hard steel is welded to the bottom of the plunger pipe. A valve or flap opening upward is fitted inside the plunger pipe just above the bottom so that the material that comes into it is retained in and taken out of the bore. The plunger is worked inside the casing pipe several times. It is lifted up and suddenly released when it falls down by its own weight. The soil is broken up and pulverized by the cutting edge of the plunger and the loose material collects inside the plunger pipe. After about 30 to 40 strokes when the pipe is full of the pounded material, the plunger is pulled out and the material collected inside is removed. The bailer or sand bucket used with the drilling bit for rocky formations consists of a tube of slightly smaller diameter than that of the casing pipe and 3 to 4.5 metres in length fitted with a check valve at the bottom. After the soil has been pounded with the jumper bit, and sand pump is dropped in the bore, it fills with the debris and the valve closes at it is raised. Water has to be added during the process for the loose material to form into a "slurry".

As the pounded material is brought up through boring a chart is

made of the location of water-bearing strata and a complete length of strainers and plain pipes is made out and lowered inside the casing to the correct levels so that the strainers are located opposite the water bearing strata. A piece of "blind pipe" is fixed at the bottom end for any he avy particles of sand settling inside the tube-well during development of the well, which can collect inside this tube and not choke the strainer. The whole finished length of the pipes and stainers is lowered down into the casing pipes and kept suspended from top and the casing pipes jacked up and extracted one by one. As the pipes are extracted the soil surrounding these gets loosened and grip the stainers and pipes on the outside and hold them in position. Before lowering the strainers the casing pipes are raised up to such a level that their bottom end is only slightly below the proposed level of the bottom of the tube-well and the bottom of the well is also filled with some sand and gravel so that the strainer pipe can rest on it when lowered in position

If the material in which the well is drilled is loose and unconsolidated, it may have to be abandoned. To test whether a particular clayey layer will stand firm without collapsing is to put a lump of the clay in a glass of water and if the clay disintegrates quickly, it may not stand by itself.

Rod Boring is similar to rope boring except that rods are used extending from ground level to the boring tool, instead of a rope. This system is useful for very deep borings or borings in very hard strata or where large amounts of water are encountered.

Borings with earth augers, cross chisels, flat chisels, or bull nose auger, etc., can be made for shallow wells of small diameters in unconsolidated material. Rods are used with the augers which are raised to the surface for cleaning when filled. Power may be used for turning the auger. When the auger has reached water-bearing sand, casing for the well is placed by driving in. In sandy material, clay may be dumped into the hole to make the sand sticky, or a plunger used instead of an auger. This method is more expeditious than rope boring. These are called "Bored wells".

Checking Verticality of Bore-holes. In the case of tube-wells, specially if submersible pumps are used, the tube-well must be strictly vertical to avoid hammering. A simple method is to make two similar discs of iron plates about 3 mm thick and of diameter slightly less than the inside bore of the pipe. The discs are jointed with an iron rod or pipe of about 25 mm diameter at a distance of about 3 m and tightened with nuts. A knob is fixed on the top nut to which a thin steel wire is attached and the disc suspended into the tube by the wire passing over a pully suspended from a tripod. When the discs are lowered down into the pipe, the wire is exactly in the centre of the well pipe. When the discs are further lowered down and if the well pipe is not truly verticle, the wire will deviate from the centre, which will be indicated at the top of the pipe. Some holes should be punched in the discs so that they be easily immersed in water. This is called plumbing disc. Shafts can be plumbed by means of plumb lines suspended at the surface in the usual way. A minimum deviation from the vertical is required, with absolute verticality as the ideal. This ideal, however, is rarely obtained, except in the case of shafts. A deviation of 100 mm from the vertical per 30 m boring is generally accepted where submersible pumps are not to be installed.

Setting Right Wells Sunk Out of Plumbs. A simple method is to loosen the soil on the side towards which casing pipe is required to be pulled back to bring it into plumb again. The casing pipe is forced back by jacks which should be applied about a metre below the ground level so as to have a good hold on the side. The soil can be loosened by an additional boring which may be of about half the diameter of the well pipe. When the pipe has been forced into its correct position, the hollow left behind should be filled with gravel and earth well rammed so that the pipe does not spring back, and then jacks removed. As soon as slight devation from verticality is noticed is should be set right.

Extraction of Strainers and Pipes. It is sometimes necessary to lift the pipes or to withdraw them completely from the ground. The following methods can be adopted : In the case of small tube-wells where the pipes have not jammed and the cap put in on the bottom of the blind pipe below the strainer has an "eye", a hook tied with a wire rope can be lowered inside the tube and caught in the eye and pulled up. Where the pipes have jammed, a simple device is to make a tapering conical piece of wood, like a frustum of a cone losely fitting inside the tube, which is attached with an iron rod and lowered into the tube with its wider edge at the lower end, to some point opposite a plain pipe. If gravel or stone chips are poured into the pipe these will collect round the wooden piece and form a wedge, and when the rod is pulled up it will tend to pull the pipe along with it. A steel clamp is fixed over the rod and arrangements made to pull it up. The pipe can also be withdrawn by means of a crow-bar or a wooden log passed below a clamp fixed with the pipe and used as lever, or with the help of jacks. Special tools are also available which will expand when lowered inside the tube and grip the pipe.

Jammed pipes due to sockets : Such pipes should be pulled up and down for short distance till the soil around collar becomes loose. A powerful effort to pull it down or up with only jam it more. Sometimes dynamite is used to loosen the soil around the pipe.

Difficulties in Boring

Sand blowing. Where fine sand under considerable pressure is encountered between two hard strata it will rush through the boring pipe and hinder the progress. With such a condition fill the boring tube with water up to its top and free the boring pipe so that it could sink by its own weight as the sand is cleaned out from within it. The water will keep the sand under pressure and prevent it from rushing up. If necessary another tube can be added at the top and filled with water to increase the pressure. Some chemicals are also available which are helpful in boring and prevent the sand from blowing.

Descriptions of Boring Pipes and Shoes etc., for Tube-Wells

Blind pipes : (i) A length of plain pipe in which there are no slots or holes and which is fixed against an impervious stratum, (ii) The small

length of plain pipe used at the bottom end of a strainer (or slotted pipe) and has a cap or bail-plug fixed to it. A blind pipe is generally 1.2 to 1.5 m in length. The bottom of this blind pipe should be a little above the bottom of the bore otherwise it will give way under the weight of the whole pipe length.

Fishing tool is the name given to a large variety of tools used for the recovery of tools and pipes lost in a hole.

Housing pipe : The larger diameter plain pipe in which the bore hole pump bowls are housed (for compound wells).

Frozen pipe : Boring pipe which has been struck in the bore hole and cannot be moved.

Bail plug : A cap put at the bottom of a plain pipe to close the end. It generally has an "eye" on the inside and a lifting hook lowered with a wire rope from the top can be caught into it and the tubes withdrawn.

Boring-tube Joints : The types of joints generally used are—Flush joints ; Socketed joints ; and Swelled and Cressed joints. Pipes with flush joints have to be thicker than other joints, but sinking and withdrawing is easier.

Bell-socket : Is a connecting piece for two different diameter pipes used in a compound well. The socket is screwed to both the pipes at their junction.

As regards threading of casing pipes, 8 threads to 25 mm are stronger than 10 threads but it requires thicker pipes. Therefore, it is preferable to have 10 threads when the thickness of the pipe is less than 8 mm and 8 threads for thicker pipes.

Sludger or Sand Pump : Is a boring tool used with percussion drills and is made of a steel pipe about 6 mm thick. The diameter is about 50 mm less than the bore of the casing pipe and length varies from about 1.8 to 3 m. A cutting shoe of hard steel is welded to the bottom of this pipe and is sightly tapered out at the bottom. A flap or ball valve (non-return) is fitted inside the sludger pipe just above the cutting shoe, to retain loose material. The sludger is used for bringing up loose mud or rock from inside the hole.

Strainers or Well Screens are perforated or slotted pipes. Various proprietary makes of strainers are available in different diameters and lengths. The size of the openings (or narrow ends of the slots) depend upon the grain size of the stratum through which water is to be drawn. The yield from a tube-well is very much dependent on the relative size of the strainer slots to the grain size of the strata, and the diameter and length of the strainer as a whole. The opening must be small enough to prevent the entrance of any large quantity of sand. The net total area of the strainer openings should be such that the discharge velocity from the well is neither too high nor too low. A high velocity will lift the sand with water and clog the strainer, and if the velocity is low, the fine sand will settle down and is likely to gradually choke the well. Where this cannot be helped with the sand of the strata, shrouding is done. A strainer of 13 to 16 mesh per sq.

cm may give traces of sand for a few days but will not get choked early. It is considered that the superficial area of unperforated surface in the perforated tube should be more than twice the area of perforations to prevent eddies and back flow. A strainer should be at least 1.5 m long. The length may be about 150 times that of the diameter of the pipe for maximum draw-in.

A strainer made like this will deliver roughly 5 to 7 litres per minute per 1000 sq. cm of strainer area with infiltration head of about 3.7 metres. The sand of the particular stratum met with is passed through a sieve and the sieve which retains about 10 per cent of dry sand (this will retain about 20 to 30 per cent of the material when wet)--(some engineers prescribe 20 per cent and 50 per cent respectively) is considered to be the correct size of strainer for that stratum. (If the stratum consists of sand and gravel combined, only sand is taken for sieving). If the sand is very uniform in size, a finer screen is used. Where salts are present in the water which corrode and choke the screens by deposition of carbonates, larger slit area, or low velocity of inflow should be provided.

Strainer pipes are generally made of wrought iron, brass or other non-ferrous metals, about 3 mm thick or more according to the diameter of the strainer, usually wound with thin brass or copper wire-gauze. Tube-well pipes are usually jointed together through sockets. The slots in strainer pipes are either vertical or horizontal and may be staggered. The section of the slots is trapezoidal, tapering with the wider end inside and narrow end outside and this wider opening inside prevents the sand from chocking the perforations. The size of openings of a strainer should be less than the overage diameter of the soil particles and small enough to prevent the entrance of any large quantity of sand. For irrigation tube-wells, perforations are large and far apart. Thin wire-gauge wound strainers are not suitable for permanent irrigation wells, and in formations where falls may be expected. Where, however, a gauge is wrapped round the perforated tube, the gauge should not be in direct contact with the perforated tube, there would be a likelihood of the perforations being chocked up and also of the area of the perforations being reduced; but should be fixed at a distance. Mesh type strainers are considered very suitable for irrigation wells ; these strainers consist of vertical slots made by punching a metal plate. A strainer which has slit like opening is less likely to choke than one having woven wire mesh or small holes. In the case of irrigation wells the perforations in the inner tube are kept large enough and sufficiently far apart. The tube types of screen are more robust than the wire-wound type and should be used in formations where falls may be expected.

In long lengths of strainers diameter may be varied with blind pipes of smaller diameters put in-between for the different strata. Blind pipes or plain casing pipes seal off undesirable water-bearidg strata. But where the strainers have to be extracted for some reasons, a pipe with different diameter will offer great difficulty. Well pipes are replaced with strainers in

the acquifers to be tapped and which let in water into the tube-well without allowing the sand of the strata to come in.

Well screens should be made of corrosion-resistant metal because wells fail several times due to corrosion of screens, and should also preferably of one metal to avoid any electrolytic action. A strainer should be sturdy to stand rough handling.

For a discharge of 30 litres/per sec., the size of the pipe will be about 150 mm to give a velocity of about 1.5 m/sec. and if the discharge is 45 litres/per sec., the velocity will be about 2.4 m/sec. A 250 mm diameter strainer of 40 m total length is sufficient to give a discharge of 45 litres/sec. with a max: depression head of 3.7 m and a velocity of about 0.91 m in the tube. It is not necessary to increase the size of the blind pipe above 150 mm if the strainers are of a bigger size. There will be no difference in the discharge as long as the depression is the same.

Drive-point Strainer is usually a wrought iron tube perforated with small holes with a layer of fine copper or brass wire gauze wrapped round it and which is protected by a thin perforated metal sheet (generally of brass) fastened round it. This tube has a conical steel drive-point riveted or welded at the bottom and of size a little larger than the diameter of the tube at its upper end so as to form a bigger hole than the straining tube, and screw at the other to which more pipes can be screwed on as the tube is driven. It is generally 40 to 75 mm in diameter and 120 to 180 cm in length. This strainer is used for house-hold water supply. Water-way is reduced by about 75% in this strainer.

Life of a strainer depends very much on the types of salts present in the water and the metal of which the strainer is made. Mild steel and cast iron are attacked by sodium salts, copper is attached by sodium carbonate and sodium chloride ; brass is not readily attacked by the salts usually present in soils.

Shrouding or Gravel-Packing of Tube-Wells.—Where the water bearing stratum consists of uniform fine sand that will come up with the water even with the minimum velocity at which the water can be pumped, or that will need extremely fine slots in screen to prevent sand from entering into the well, and such fine slots will let only a small quantity of water or that will clog the screen and prevent economical development of the well, coarse sand and gravel is interposed between the soil. A common method is to drill the well 150 to 300 mm larger in diameter than the intended screen diameter and the larger outer casing is lowered to the bottom of the well. The strainer and the casing are placed inside the outer casing. The well is developed by pumping at a high rate and simultaneously withdrawing the outer casing 30 to 60 cm at a time, and gradually filling the annular space with gravel. For the first 60 cm at the bottom generally coarse sand is added. The outer casing is withdrawn up to the top of the strainer where it is finally sealed off.

Shrouding must be done with gravel of the correct size so that the finer particles of sand are not drawn into the interspaces. The packing gravel or sand should not be very coarse but should be about 2.5 to 3 times the size of the sand of the strata. The shrouding should be graded to a uniform size and not a mixture. A common size will be passing through a screen of 4 meshes to one lineal cm (with max: size up to 6 mm) and retained on a 16 mesh to one cm screen. Smooth gravel obtained from river beds should be used and not crushed gravel. The slot size of the strainer should also be correctly designed. Slotted tube is sometimes used instead of the strainer. The size of the slots is generally 25 mm×6 mm with a spacing of 15 mm to 10 mm between slots. Gravel is poured around the slotted tube.

The capacity of a properly constructed gravel-packed well may be increased to several times the capacity of a plain screened well in the same location, but it is not always very effective and if not carefully done may even decrease the discharge. The larger the diameter of the shrouding the better the result.

Development of New Wells.—As a new tube-well is worked it draws at first a considerable quantity of fine sand with the water which gradually clears up with pumping, but sometimes it is in such large quantities that it has to be removed by bailing or over-pumping. This is the simplest and most common method, and this system while not always fully effective has the advantage of clearing fine paticles from the well and its immediate surroundings, and may be used only on small wells driven into fairly well graded formations. Over-pumping can be detrimental and should not be done when the formation consists of uniform fine sand, silt or clay as it may result caving-in of the ground surrounding the well. Where the velocity is high it will carry some fine sand with it ahd some will also stick on the outside of the straining material gradually diminishing the flow. Reversed flow should be resorted to under such conditions using compressed air or back-washing as explained under "chocked strainers." This may have to be repeated several times. The draw-off from a new well should be increased gradually so that sufficient time is given to the cavity to form, otherwise all the big and small particles will be dislodged and will choke the strainer. *Excessive or over-pumping* is to draw more water than the percolation at the re-charge area of the water-bearing stratum.

Effect of Increase in Well Diameter (hole) on Well Yield (approx.)

Well Diameter in cm						
10	15	20	30	45	60	90
Per cent Increase						
0	5	10	15	23	28	38
	0	5	10	18	23	33
		0	5	13	18	28
			0	8	13	23
				0	5	15
					0	10

In very coarse gravel formation, yield increases only slightly with an increase in diameter. The yield of a well increases very rapidly with the coarseness of the formation.

Relationship between Well Yield and Recommended Size of Pump Bowls and Diameter of Housing Pipe in Tube-wells :

(Vertical turbine pumps and submersible pumps)

Anticipated yield of well litres/sec.	Nominal size of pump bowl cm	Optimum size of housing pipe cm	Min: size of housing pipe cm
6	10	15	12.5
5 to 11	12.5	20	15
10 to 25	15	25	20
22 to 40	20	30	25
37 to 56	25	35	30
53 to 82	30	40	35
75 to 112	35	50	40
100 to 190	40	60	50

Remedial Measures for Choked Screens

Clogging of screens is generally due to the following two causes : (*a*) Excessive pumping with velocity of inflow more than the optimum velocity of the strata which disturbs the materials surrounding the strainer drawing in sand particles ; (*b*) Chemical action on the strainer metal due to the presence of salts (most commonly lime) in water, which will deteriorate strainers by corrosion.

"Back-washing" or "back-blowing" under pressure will dislodge fine sand or clay particles clogging the strainer holes. Water is forced into the well with a pump or from a high level storage tank through a water jet tube over the strainer openings of compressed air blown into the strainer. Raising and lowering a plunger will induce reverse flow forcing water outward through the screen. If a turbine pump is operated without a foot valve and suddenly stopped the water in the pump column will cause the back flow. Trouble due to incrustations of lime can be removed by introducing into the well weak hydrochloric acid and allowing the acid to remain for several hours, and this will dissolve the incrustant salts. Heavy pumping will bring up the deposits. Where the stratum is clayey or is high in organic matter, the above process should be repeated with sulphuric acid. Where a screen is obstructed by heavy growth of iron bacteria, a 50 ppm chlorine solution will clear it. Where a strainer has collapsed or caved in, a smaller casing can be placed inside if the well is of large diameter otherwise replacement is necessary.

The life of a strainer depends upon the type of salts present in the water and the effects of their chemical action on the metal of which the strainer is made. Mild steel and cast iron are attacked by sodium salts ; under ordinary conditions steel may last for 15 to 20 years. Copper is attacked by sodium carbonate and sodium chloride ; brass is not readily attacked by the salts usually present in the soils. Where the screens cave in or collape or corrode and leak, allowing water to escape into the ground, replacement is necessary. If the well is of a large diameter, a smaller casing may be placed inside the old one.

Household Tube-wells or Hand Pumps

Outside India such hand pumps are often called *Jack pumps* and the tube-wells as *Abyssinian tube-wells*. The working parts in these Jack pumps are few and simple—a bucket and a plunger.

In its simple form it consists of a wrought iron strainer tube of about 1.2 to 2.4 m length and 40 to 60 mm diameter, perforated with small holes. A layer of fine copper or brass wire-gauze is wrapped round the perforated tube to act as a straining material and over this a layer of thin perforated metal sheet (generally of brass) is fastened to act as a protection to the wire gauze. The water-way area is considerably reduced (up to about 75 per cent), through the straining tube in this manner. The bottom end of the straining tube is provided with a steel driving point which is conical in shape and of size a little larger than the diameter of the tube at its upper end so as to form a bigger hole than the straining tube. The other end of the tube is connected with a wrought iron pipe. The tube thus formed is driven vertically into the ground, for small depths, generally to the uppermost or comparatively high water-bearing stratum. For bigger depths small borings are made and the strainer tube lowered down as described before.

For wells under 9 m a hand pump with a 10 mm barel and a stroke of about 25 cm is convenient for one man to work, and will deliver about 23 litres per minute.

In the case of a well for drinking the top of stainer should be 15 m min: below ground to avoid surface pollution unless unusually impervious earth (3 m compact clay) occurs at the surface.

The trouble with these pumps is generally refusal to pump without being "primed" with water. This is due to worn leather cup on the bucket or a faulty bottom valve. If the pump is dry through long standing it will have to be primed before it will work, and in bad cases the bucket will have drawn out and put to soak in a pail of water until the leather cup softens.

Various types of house-hold pumps are available, mechanical and hand operated and are used where small quantities of water are to be raised from shallow wells or tanks. Such pumps will generally lift water from a depth of about 6 m forcing it up to heights of as much as 15 m. Where the draw-down level is less than 5 m the cylinder is usually placed above ground, and when the static water level is more than 5 m the cylinder is attached to drop pipe and placed in the well.

In operation the valve in the piston is closed on the upstroke and open on the downstroke, thus pushing water from the cylinder and out of the spout while at the same time water is drawn into the cylinder through the second valve which is open. These pumps generally deliver

water intermittently. An air-chamber is sometimes incorporated in the hand pump to give a smooth flow. The riser pipe is usually about double the diameter of the suction pipe.

Where the cylinder is in the well it is called deep well pump. Priming is necessary for pumps with cylinders above ground and no priming is required where the cylinder is below static water level in the well. When a closed cylinder is placed in the well it is more difficult to maintain. Force pumps are generally made for raising water from 15 to 23 metres and do not require priming. Inside the pump barrel is the bucket controlled by a piston fixed with steel rods, generally submerged in water.

The capacity of a hand-operated pump is limited by the size of the pump and the power exerted by the person operating it. Consequently the lower the level from which the water is drawn the less the amount delivered. Some typical figures are as follows :

(*i*) Shallow pump, one man operated, lift 6 m, 35 litres per minute.

(*ii*) Deep-well pump, one man operated, lift 30 m, 5.5 litres per minute.

(*iii*) Deep well pump with rotary head, two men operated, lift 30 m 5.5 litres per minute.

Other type of hand pumps are semi-rotary pumps which work for small suction lifts. Their capacity depends upon the number of strokes of the handle per minute.

Open Dug Wells

Dug wells can be made in alluvial soils beside lakes and ponds where the porosity of the soil permits free drainage of water into the excavation, and where the ground water-table is close to the surface. Dug wells have the advantage of much large storage capacity than drilled or bored wells. But these wells may fail in times of drought when the water-table recedes, because of their shallow penetration into the zone of saturation. Because of the large opening and large perimeter these wells are subject to pollution by surface-wash, by wind-blown material and by other foreign matter. Dug and bored wells that extend only a short distance below the water-table are also likely to admit contaminated water. A good degree of security from contamination can be obtained by making a water-tight casing that extends a considerable distance below the normal water-table. Sources of possible contamination can be minimised by constructing the well on a higher land free from contamination.

Recommended minimum distance between a well and the sources of contamination :

(*a*) Pit privies ; septic tanks ; sewers and sub-surface pits	... 15 metres
(*b*) Seepage pits : sub-surface sewage disposal fields and barn-yards	... 30 metres
(*c*) Cesspools	... 45 metres

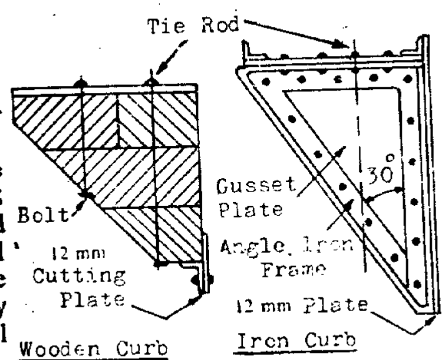
A lining of permanent materials is always necessary except when wells are sunk into rock or consolidated hard soil. The lining serves several purposes : It is a protection during construction and afterward against caving and collapse, and acts as a foundation and support for the well top and for a platform to be built, or any pumps that may be installed. The lining should be made water-tight to exclude contaminated surface water and rodents.

Open wells with steenings can be made economically up to a depth of about 30 m and Kachha wells up to about 6 m. For an open well with impervious lining in alluvial soil, the maximum safe discharge is taken to be 250 litres/minute. An irrigation well should be sunk about 6 to 7.5 m below the spring level so that in years of drought there is a margin of about 3 m leaving about 3 to 4.5 m infiltration head. The supply of water is more reliable in alluvial soils though the construction of an open well in such a soil is difficult and of comparatively high cost.

In the case of impervious lined wells, when the bottom of the well is in sandy soil it is usual to provide a layer of coarse sand at the bottom, a layer of fine gravel in the middle and a layer of bajri about 10 to 12 mm size at the top. These layers are about 30 cm thick each and serve as filters so that a dangerous cavity is not formed during the year of drought.

In the case of pervious lined wells (with dry bricks or inter-twined brushwood) in sandy soils, if brick-ballast of small size (about 20 to 25 mm) is put behind the lining it will prevent sand from the sides getting into the well.

Well Curbs may be of wood, iron or reinforced concrete and of the same width as the steening. A wooden curb is made of hardwood such as, kikar, shisham, sal, babool, tamarind, mango or jamba. It is of two thicknesses of wood for wells 1.8 m in dia. and under, and of three thicknesses for large wells, strongly dove-tailed and dowelled together and secured by iron bolts to avoid risk of the curb breaking up during sinking. When rings cannot be made of one piece acros the width, the concentric rings should break joints ; the upper and lower courses to be alternatively one-third and two-thirds of the whole width. An iron curb is made of 6 to 8 triangular frames of angle iron covered by 10 mm to 12 mm thick plate all round. Iron curbs filled with concrete should be preferred. It is better to lay curb on a layer of clay which is well below water level in dry season.



Vertical tie rods of about 20 mm dia. are fixed in the curb at about 1 to 2 m intervals and brought up through the centre of the steening masonry for reinforcement. The length of these rods is 2 to 3 times their horizontal spacing. These tie rods are anchored (with nuts) at the

top (of tie rods) with a circular flat iron 6 to 10 mm thick and 75 to 150 mm wide ; another plate may be used in between (the curb and the top of rods) if necessary. This will enable the masonry to sink without cracking.

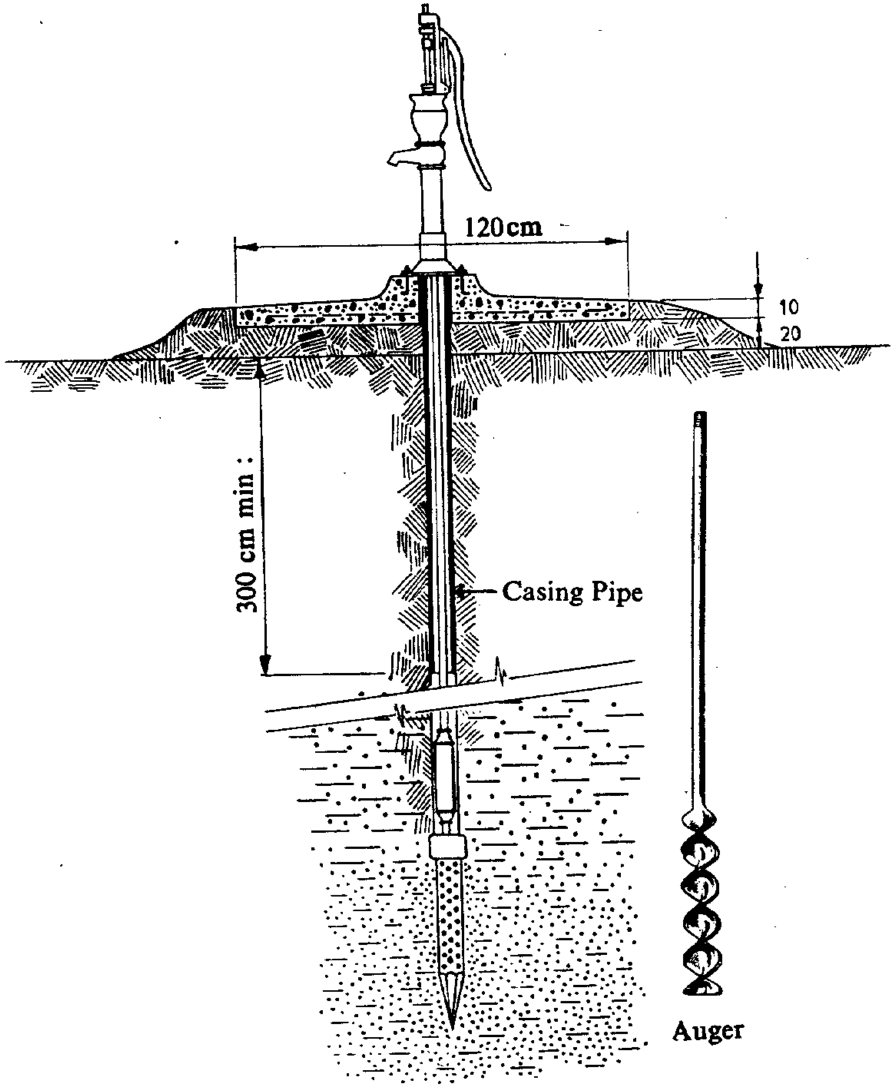
A well curb or 'shoe' is employed where the well casing is sunk as the excavation progresses, and is usually practicable in alluvial formations and fine sands. In areas of coarse sand and gravel, instead of the curb, stones are laid after the well has been completely excavated. In some places where wells are sunk through hard soil, it is not the custom to use curbs, but to build the steening of the well on a ledge formed in the soil through which the well is sunk after the excavation of the well is complete. In such cases, the steening should be started at such levels, of the foundations where there will be no danger of disintegration subsequently taking place below the level of the steening. Above the commencement of the steening the finished diameter of the well is 60 cm greater than below it. In rocky areas the method of excavation adopted is to explode the hard rocks by charges which generally penetrate throgh such formations.

Where a well is to be built on a curb, a circular pit is dug at the site about 1.5 metres bigger in diameter than the outer diameter of the well, and up to the spring level or bottom of the upper stiff strata. The pit should be dug with an inside slope of a least 1.5 : 1 in good soil or 1 : 1 in light soil cut in steps up to 15 cm above the water table. The curb is laid and steening built up to about 2.5 to 3 metres above ground level. In under-sinking wells great care should bo taken to keep the steening truly vertical ; for this purpose three or four plumb lines should be kept suspended round the instde of tne steening, and the well should not be out of plumb by more than 1 in 50 (max.).

The diameter of a well will vary with the quantity of water available; larger diameters are required (up to a certain limit as has been explained earlier) where the supply is scanty than where it is plentiful. A minimum convenient and standard size 1.3 metres diameter is for one man and 1.52 metres for two men to work in a well. Efficiency of two men working together is considered more than twice that of a single man.

A platform is built on the steening masonry projecting half inside and outside leaving about 1.5 m diameter space inside for removing the excavated stuff. The platform is loaded with gunny bags filled with sand. If there is a hard layer of soil on one side and the well does not sink uniformly but starts to tilt, the load on the platform is adjusted accordingly. Care should be taken that the dredger does not make too deep a sump below the cutting edge of the well curb, and the sump should not be deeper than the inside diameter of the well. Care should be taken to take out the earth only from the centre of the well to avoid unequal sinking. As the excavation proceeds and the loaded masonry sinks down it is built up at the top. Sinking of the well should be done after allowing setting time of at least 14 days to the well masonry built above the curb.

Driven Well with Drop-Pipe & Cylinder



In areas with relatively coarse sand, driven wells can be an excellent and cheap means of obtaining water. They can be driven rapidly and put into operation quickly. Water-tight casing should extend down to a minimum of 3 metres below the ground surface.

In sandy soil sinking is expedited by bailing out the water from inside the well. Care should however, be taken to stop bailing immediately a "blow" is threatened. (Sinking of wells has also been described under "Bridges").

The discharge from an open well can be increased by sinking a tube-well in its bed. The bottom of the well is sealed with cement concrete. Open wells can be provided with pumps of the force type *i.e.*, with a cylinder submerged in the water.

Boulder well is a cavity well through aquifer cantaining boulder, gravel and sand.

Well Linings or Steening. In alluvial soils open dug wells with steening can be made economically up to a depth of about 30 metres and wells without lining (and curbs) are practicable only up to depths of about 5.5 to 6 metres. The steening is usually made of brickwork or masonry. Large cement concrete rings 60 to 90 cm high are also used. Where dry masonry is used, in between the dry masonry some bonds of pucca masonry are also provided for strength. Bricks with mortar are sometimes used for a height of about 3 metres from the ground surface with dry joints below.

To prevent infiltration of dirty surface water into the well, the steening should be made water-tight and also, when filling up the excavation, to back the lining with some impermeable material, such as puddle, for 2.5 to 3 metres from the surface and about one metre thick. Where the whole of the lining is made impervious, the water can percolate into the well from the bottom only. Weep holes 150×75 mm for the admission of water are left at intervals in the steening and which may be omitted in shallow wells.

Thickness of Steening of Wells

Depth of Well from ground level in Metres	Suggested Thickness in Centimetres for Diameter of Well in Metres						
	1.5 m	3 m	4.5 m	6 m	9 m	12 m	
3 m	40	40	40	50	50	50	
6 m	40	50	50	50	60	70	Where high
9 m	50	50	50	60	70	90	skin friction
12 m	50	60	60	70	80	100	is anticipated
18 m	60	60	70	80	90	110	thickness of
24 m	60	70	80	90	—	—	the steening
30 m	70	80	90	110	—	—	should be
36 m	70	90	100	—	—	—	increased

For rubble masonry steening for smaller depths, the thickness may be 7.5 to 15 cm more than the respective figures given above for brick-work steening.

Steel or RCC tubing can be used as lining up to 1.8 metres in loose and sandy soils.

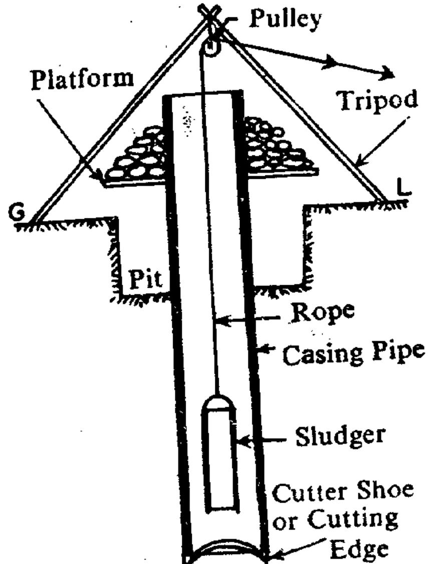
Kachha wells without lining which are of temporary nature can be made only where the spring level is not below 3 to 4.5 metres from the ground level and the soil can stand vertically (or with a slight slope).

Cavity Wells. Are tube-wells, which, being without strainers, draw their supplies from a cavity developed in the water - bearing aquifer at the end of the pipe. A cavity well can be formed in a suitable water bearing stratum which is just below a good hard clay, conglomerate or a stone layer capable of supporting itself. This layer is pierced through and a cavity developed by pumping a sand pump is used for the purpose. When constructing a cavity well, the sand pump should be worked about 1.5 metres ahead of the cutting shoe of the boring pipes. This cavity can be developed further by pumping either with an air compressor or a pump. The well should preferably be over-developed to some extent. The process of pumping is repeated till the water becomes clear of sand particles. There is no strainer in a cavity.

Lining of Temporary Wells can be made as follows : (*a*) A circular cylinder is made of wooden frames and studs with corrugated iron sheets nailed outside it with angle iron cutting edge fixed at the bottom. This is something like a 'caisson' or steel cylinder described under "Bridges". The cylinder is placed in the excavation. Two rails are placed across the top and the ends weighted with sand bags. The earth is then excavated round the cutting edge and the cylinder sinks by its own weight and that of the sand bags. When the cylinder is flush with the ground another cylinder is bolted on top and the process continued to the required depth. This is suitable for wells up to 2.5 metres diameter and 4.5 metres depth. If cross-braced it may be used for greater diameters. (*b*) Timber planks can be sunk as for shafts. For shallow wells 120 × 120 cm size will do, but for wells over 15 metres depth 160 × 160 cm should be adopted. Above this size cross-bracing is required.

Driven Wells. Small wells giving supplies of 25 to 50 litres per minute can be made in shallow water-table area where the sub-soil water is about 8 to 9 metres in alluvial tracts of unconsolidated material. It is also a very useful and cheap method for obtaining water from ponds and rivers for small supplies in which case the well should be driven into the bank close to the water. The well is constructed by driving into the ground a pipe at the lower end of which is fixed a "drive-point strainer" (explained under 'strainers'). Driving is done either by wooden mallets or a droping weight arranged as a monkey sliding freely upon the pipes and raised by two ropes passing over sheaves at the top of a pulley bar which fits into the top of the well. More pipes are screwed on as driving continues. In clayey soils sometimes casing pipes are also driven to safeguard against the strainer being clogged or damaged. Driving is continued until the strainer reaches a water bearing stratum. Driven wells can be made up to 80 to 100 mm dia., the most common size for domestic tube-wells being 2 to 40 mm and can be driven up to about 15 metres. The drive point strainer is capable of penetrating soil almost up to the hardness of

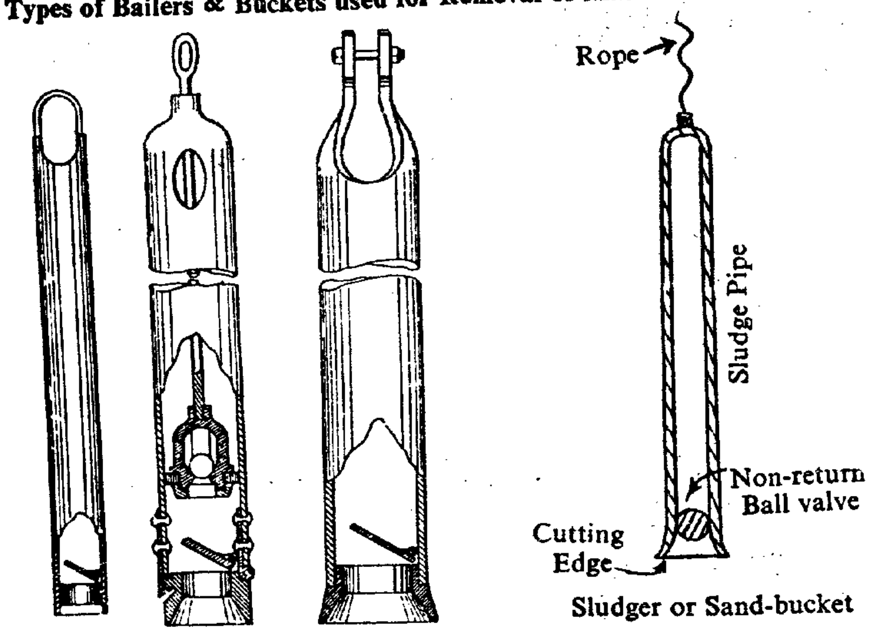
Percussion Method
or
Standard Boring Method



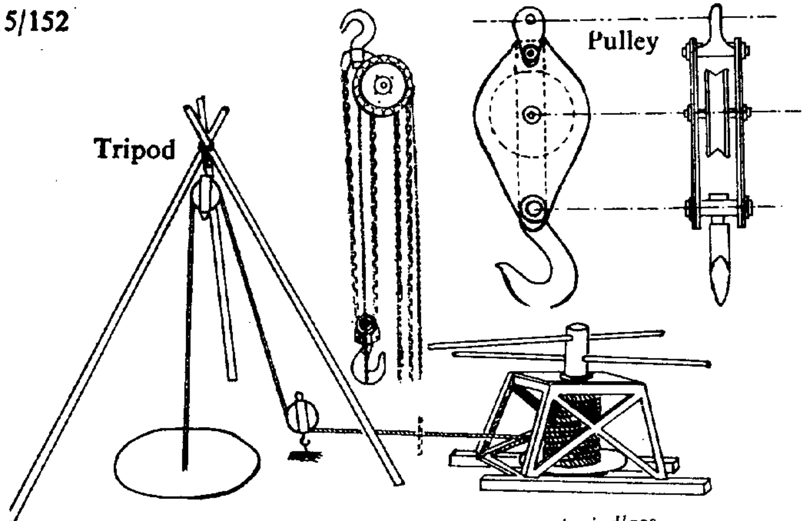
A pit 3 m to 6 m deep and 3 m in diameter is dug over the site and a tripod with pulley is placed over it firmly, with a sludger or sand bucket suspended from it.

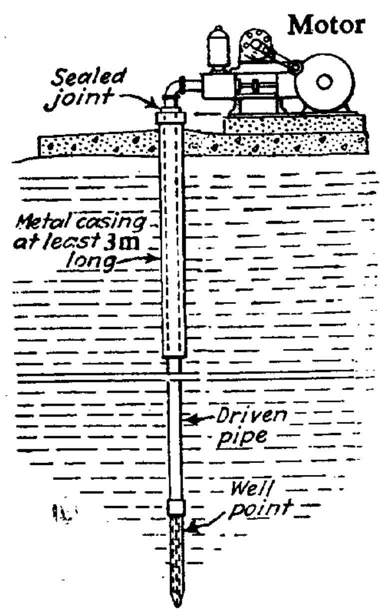
A boring or casing tube with a cutter shoe is now placed in bore hole. Wooden sleepers are placed around the casing tube clamping it firmly in position, thereby making a platform which is loaded down by placing sandbags over them to exert downward pressure on the casing tube which is filled with water partly. The plunger is raised and dropped into the bore hole suddenly several times until it gets filled up with disintegrated material. It takes about 30 strokes to fill up the plunger or bucket completely. When it is taken out and emptied. The casing tube under the load slowly goes down and down as material is removed.

Types of Bailers & Buckets used for Removal of Material from Holes

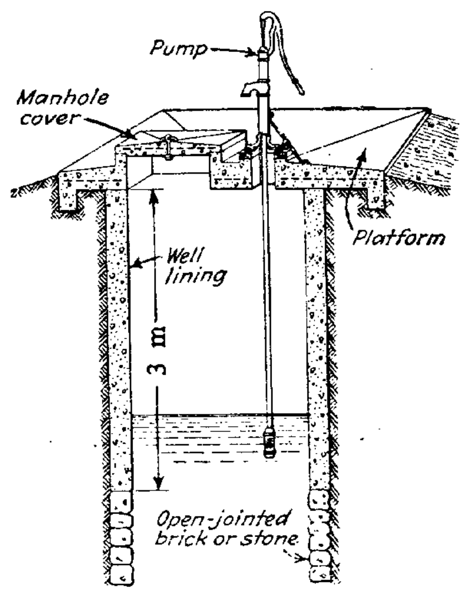


Sludger or Sand-bucket

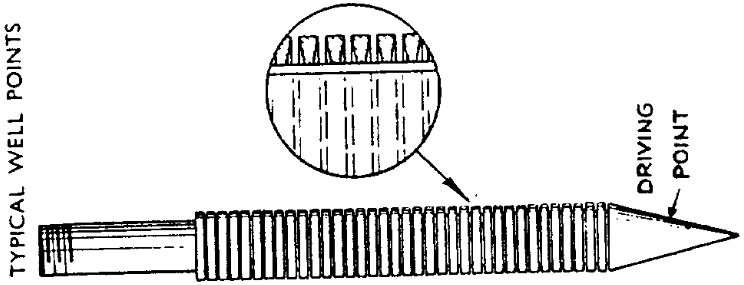




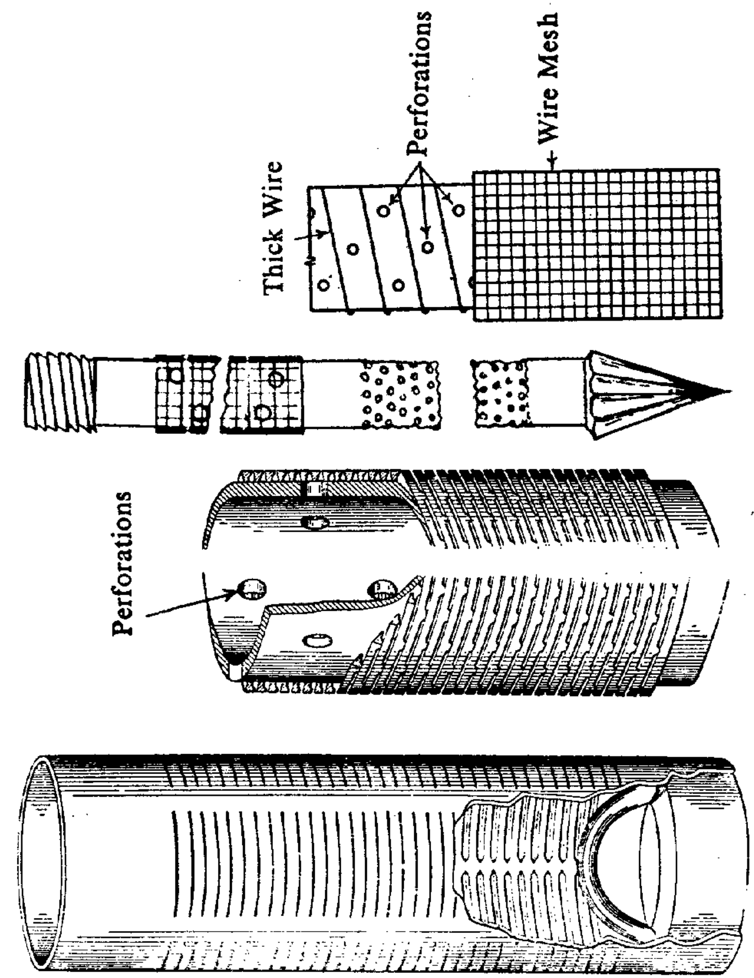
A Driven Pump with a Power Pump



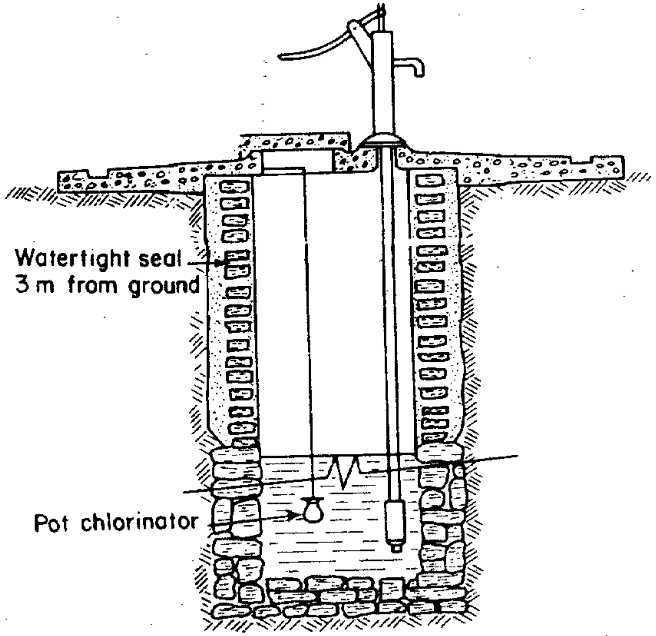
Dug Well



TYPICAL WELL POINTS



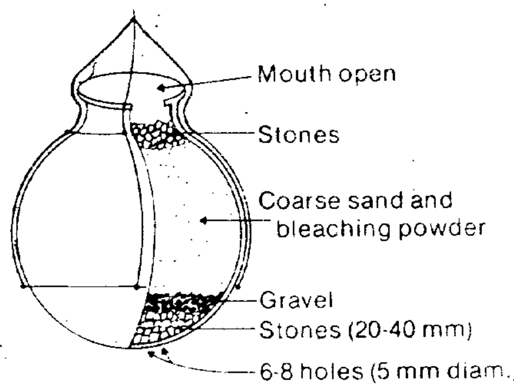
Metal Well Strainers



Dug Well with Hand Pump

Chlorination of Wells

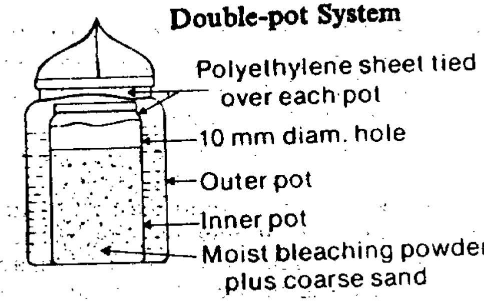
Termed "chloride cartridge", the device comprises an earthen pot of seven to eight litre capacity with a number of 0.5 cm diameter at the bottom. A dry mixture of bleaching powder and coarse sand in proportion of 1:2 by weight is placed in the cartridge over the gravel. This cartridge covered at the neck is immersed in the well. Due to agitation, while drawing water from the well, the bleaching powder gradually oozes out into the water, thus providing an effective method of disinfection.



Simple Chlorination Pot

Single-pot System

Double-pot System



Relationship between the depth. dia. of the well, and the consumption of bleaching powder

Water depth (m)	Bleaching powder (g) for wells with diameters: 0.5 m	0.8 m	1.0 m	1.5 m	2.0 m
1	1.6	4.1	6.4	14.4	25.6
2	3.2	8.2	12.8	28.4	51.2
3	4.8	12.3	19.2	43.2	76.2
4	6.4	16.4	25.6	57.6	102.4
5	8.0	20.5	32.0	72.0	128.0
6	9.6	24.6	38.4	86.4	153.6
7	11.2	28.7	44.8	100.8	179.2
8	12.8	32.8	51.2	115.2	204.8
9	14.4	36.9	57.6	129.6	230.4
10	16.0	41.0	64.0	144.0	265.0

The required chlorine is 2 mg of bleaching powder containing 25% free available chlorine for each litre of treated water

Bleaching powder consumption of different water sources in grams/cu.m. Bleaching powder with 25% chlorine content

Source of water	Chlorine requirement (mg/Litre)	Bleaching powder consumption grams/cu.m
Rainwater	0.5–1.0	2–4
Deep-well water	0.5–1.0	2–4
Shallow-well water	1.0–1.5	4–6
Spring water	1.5–2.0	6–8
Turbid river water	2.0–2.5	8–10
Pond water (in a good environment)	2.0–2.5	8–10
Pond water (in a poor environment)	2.5–3.0	10–12
Lightly polluted stored water	2.0–2.5	8–10
Heavily polluted stored water	2.5–3.0	10–12

chalk. Care must be taken that strainer is not driven through a water-bearing stratum down into some lower impermeable stratum. By lowering a small plumb line down inside the tubing, the well can be sounded from time to time. Such wells are sometimes called *Norton tube-wells.*

In areas with relatively coarse sand, driven wells can be an excellent and very cheap means of obtaining water. They can be driven rapidly and put into operation quickly. They cannot be put in hard, solid stone or hard-pan strata, or in heavy beds of clay, but can be successfully driven through compact soils. In individual installations where the ground water is within 7.5 m of the ground surface the pump cylinder is usually attached directly to the top of the rising pipe. The total depth of such wells seldom exceeds 10 to 15 metres.

Bored Wells. A bored well is constructed by boring a hole with a hand or machine operated auger and installing casing. This method is suitable for small diameter wells of shallow depth in soft soils devoid of large gravel and stones and which can be easily penetrated. Procedure for boring consists of screwing a hand or power operated (usually hand operated) helical auger (earth borer or earth auger) into the ground and withdrawing it full of soil material which has been caught in the auger, for cleaning. Wells can be bored to considerable depth if the soil is suitable, by adding rods to the auger. Small bored wells are made where the water table is less than 9 m. These wells are usually 5 to 8 cm in diameter. If the well to be bored is of a bigger diameter, mechanical arrangement is employed and the augers used are different which have sharp tongues (auger bits) and are not screw type. These augers penetate short lengths at a time when they are lifted up full of soil and cleaned.

When the soil is so soft that it frequently caves in, casing pipes are lowered as boring proceeds and which are left in position. The wells are bored about 6 to 9 m into the water bearing strata. Small size pipes with strainers attached are lowered into the hole and pump installed. Water pumps are used for drawing water, casing pipes are 10 to 15 cm in diameter and inner pipes 4 to 6 cm which act as suction pipes. If the stratum is very coarse, the bottom of the inner pipe is left open and fitted with a foot-valve.

In very stable cohesive soils casing pipes may be put up to a short depth to protect the top of the borehole from caving and abrasion, and for protection against infiltration of dirty water.

12. PREPARATION OF PROJECT ESTIMATES FOR WATER SUPPLY SCHEMES

A project estimate should give the following details in the Report :—

(*i*) Reasons necessitating an improved or additional supply.

(*ii*) Nature, quality and quantity of existing supply and its sources.

(*iii*) Possible sources of additional supply and arrangements for its filtration and purification, etc.

(*iv*) Area and the number of people and approximate number of animals to be catered for.

(*v*) Estimated daily allowance per head and how calculated.

(*vi*) Provision for future developments.

(*vii*) Wheather the supply will be metered ; constant or intermittent supply proposed.

(*viii*) If pumping is contemplated, the annual cost of working the pumps should be estimated.

(*ix*) The mode of calculating dimensions of pipes etc., and the formulae used should be mentioned

Estimate should include for the cost of quarters for the staff of the pumping house.

Details will have to be worked out for the total cost of the scheme, its annual maintenance, depreciation of the plants and running expenses, etc., and the water rate proposed to be levied.

The following costs may be taken roughly :—

Pumping stations	...	18 per cent of total cost
Reservoirs	...	6 ,, ,,
Filter plants	...	10 ,, ,,
Distribution system	...	50 to 70 ,,
Supply	...	9 ,, ,,
Building works	...	2 ,, ,,
Metres on consumer side	...	4 to 5 ,,

The cost of the pipes and valves usually amounts to 80 per cent of the cost of the distribution system.

Total cost roughly, of a water supply scheme comes to about (including interest and depreciation) Rs. 400/- for big schemes to Rs. 1600 for small schemes, per million litres of water supplied.

Maintenance of distribution system including valves, fire hydrants and services will be about Rs. 2400/- to Rs. 4800/- per kilometre of mains annually.

Water treatment cost will be about Rs. 180/- to Rs. 300/- per million litres.

(Estimates are based on 1985 costs and are very approximate).

The following annual depreciations may be taken :—

Mains, valves, meters	...	1.5 to 3.5 (av: 2.5)	per cent
Pumping plants	...	5 ,,	,,
Filter plants	...	3.5 ,,	,,
Meters (consumers)	...	3.5 ,,	,,
Buildings	...	2 ,,	,,

Such schemes are generally made on commercial basis and profit or loss has to be justified.

An Index map of each zone should be prepared showing the line of mains and distributary piping and sites of filter beds, settling tanks, service reservoirs, pumping house, etc.

The map may be prepared on the following scales :—

Index map of zones	...	1 cm to 40 or 50 metres
Alignment	...	1 cm to 25 metres
Index maps of the whole town	...	1.5 cm to 1 km
Longitudinal Sections :		
Horizontal	...	1 cm to 25 metres
Vertical	...	1 cm to 1 metre

A contoured plan of the town on a scale of 12 cm to 1 km showing water mains, branches, valves, service reservoirs, pumping stations, boosters, roads and streets etc.

Detailed drawings of different units of treatment works on a scale of 1 cm to 1 metre.

In small towns the diameter of the transmission main may not exceed 300 mm while in large cities several transmission mains with diameters up to 1000 mm and over may be laid.

In small cities, the diameters of distribution mains may be 200 to 300 mm, while in large cities the diameters vary from 300 to 400 mm. Only in very large cities the diameters of the distribution mains may be from 600 to 1000 mm. The distribution lines consist of pipes with diameters of from 100 to 200 mm, and in large cities up to 300 mm and over.

Transmission mains are laid from the high-lift pumping stations to the city. Usually only one main is laid, and subsequently, as the water consumption increases, a second main is laid parallel to the first. Connecting lines are installed between the mains so that separate sections of one or another of the pipe-lines can be cut out in case of pipe failure.

The distribution water supply system is laid in the form of a closed ring, which makes it possible to deliver water by by-pass lines when any particular section of a pipeline is damaged.

Constructing a complete water purification scheme consists of : construction of inlet channels, alumina or other chemical dosing arrangement with chemical house, storage and sedimentation tanks, pumping chamber, inlet well. Flow measurement arrangements with mixing well, mechanical clarifeoculators with complete sludge disposal arrangements. Rapid sand gravity filters, overhead service reservoir. Connections of various units of the scheme etc.

Design of Distribution System

Suggested Internal Diameters of Pipes in Millimetres for Various Lengths of Discharge-lines

Discharge in Litres per minute	Length of Pipe-line in Metres									
	15	30	75	150	250	300	600	900	1200	1500
25	25	25	25	25	32	32	32	40	40	40
35	25	32	32	32	40	40	40	50	50	50
45	25	32	32	32	40	40	40	50	50	50
70	32	32	40	40	50	50	50	65	65	80
90	32	40	50	50	65	65	65	80	80	100
135	40	50	50	65	65	80	80	80	100	100
180	50	50	65	65	80	80	80	100	100	125
225	50	50	65	80	80	100	100	100	125	125
270	50	65	65	80	80	100	100	100	125	125
320	65	65	65	80	100	100	125	125	125	125
360	65	65	65	80	100	100	125	125	125	125
450	65	80	80	100	100	100	125	125	125	150
550	80	80	80	100	100	125	125	125	150	150
700	80	100	100	125	125	125	125	150	150	150
900	100	100	100	125	125	125	125	150	150	150
1100	100	100	100	125	125	125	150	150	150	200
1300	100	125	125	125	125	150	150	150	200	200
1800	125	125	150	150	150	200	200	200	250	250
2500	150	150	150	200	200	200	200	250	250	250
3000	150	150	150	200	200	200	250	300	300	300
3500	200	200	200	200	250	250	300	300	350	350
4500	200	200	200	250	250	250	300	300	350	350
5500	200	250	250	250	300	300	350	350	380	400
7000	250	250	250	300	300	350	350	380	400	400
8000	250	250	300	300	350	350	380	400	450	450
9,000	250	300	300	300	350	350	400	450	450	450
10,000	300	300	300	350	350	380	400	450	450	450
11,000	300	300	300	350	350	380	400	450	450	500
13,500	300	350	350	350	380	400	450	450	500	500
16,000	300	350	350	350	380	400	450	500	600	600
18,000	350	380	380	400	400	400	500	500	600	600

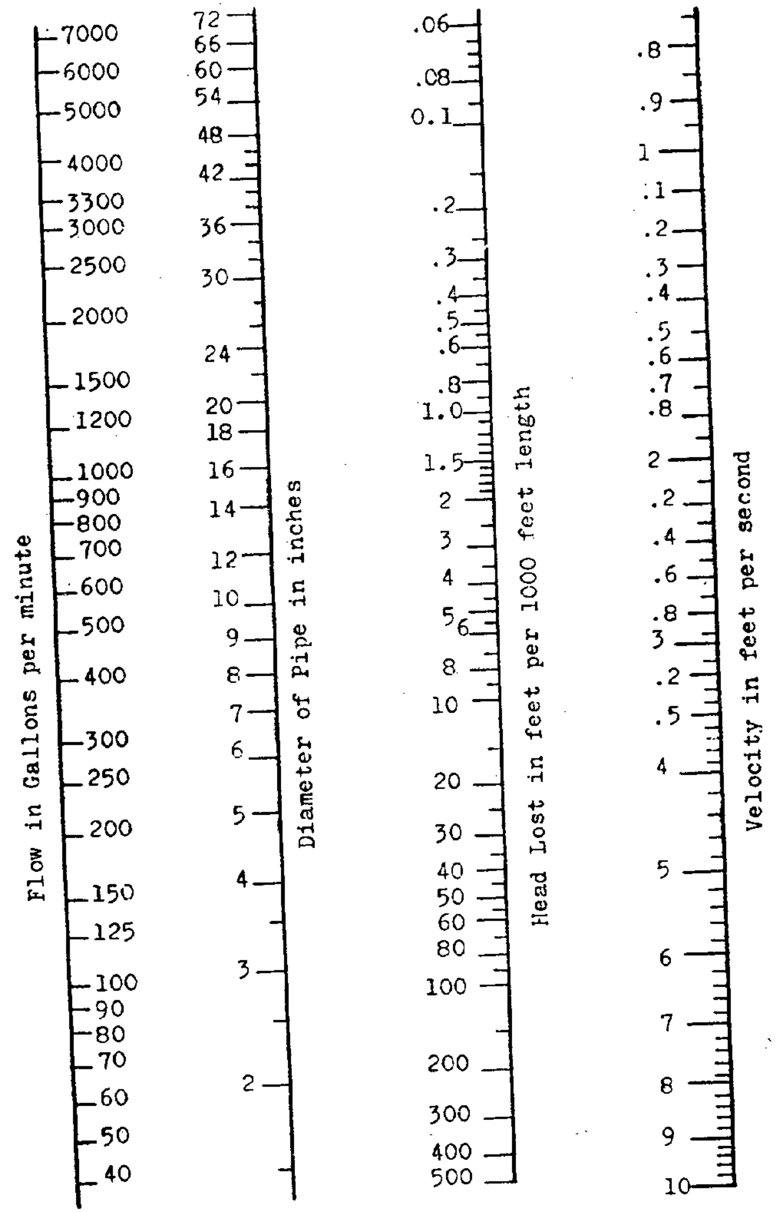
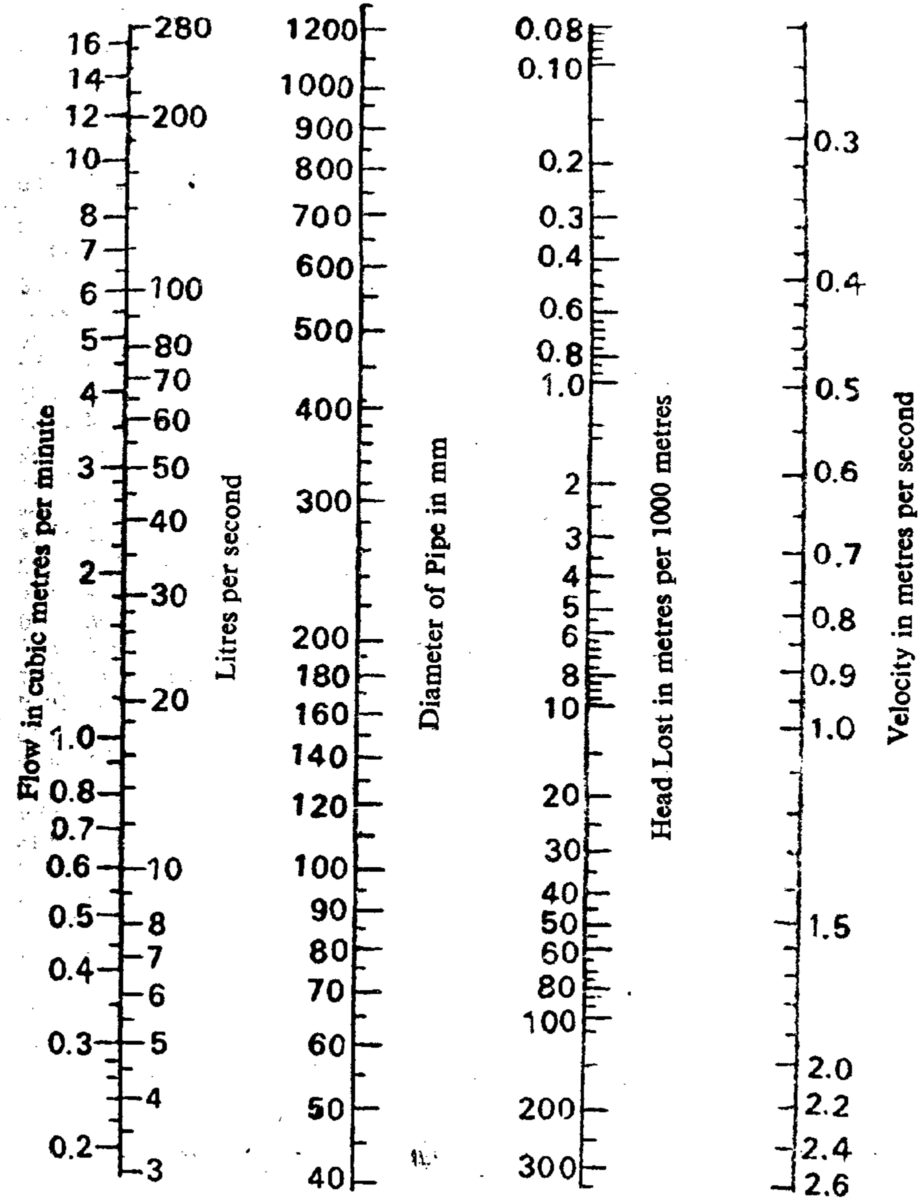


CHART FOR FLOW IN WATER PIPES
(Based on Hazen and Williams Formula with C = 100)



NOMOGRAM FOR FLOW IN WATER PIPES
(Based on Hazen & Williams Formula with C=100)

See page 15/90

Table for Hydraulic Design of Water Supply Pipes

Galvanized Iron, Coated Cast Iron & Concrete Pipes in "Normal" Condition having same Surface Roughness

(Hydraulic Research, Wallingford Oxfordshire, England)

Slope		Pipe Diametres in mm											
		50	75	80	100	125	150	175	200	225	250	275	300
1/250	V	0.35	0.46	0.48	0.56	0.64	0.74	0.80	0.88	0.95	1 01	1.08	1.14
	D	0.68	2.02	2.40	4.37	7.91	12.85	19.34	27.54	37.61	49.68	63.89	80.37
1/200	V	0.39	0.52	0.54	0.63	0.72	0.82	0.90	0.98	1.06	1.14	1.21	1.28
	D	0.77	2.28	2.71	4 91	8.90	14.44	21.73	30.94	42.20	55.80	71.70	90.21
1/154	V	0.45	0.59	0.62	0.72	0.83	0.94	1.04	1.13	1.22	1.30	1.38	1.46
	D	0.88	2.62	3.11	5.64	10.21	16.60	24.90	35.50	48.40	63.9	82.14	103.3
1/125	V	0.50	0.66	0.69	0.80	0.93	1.04	1.15	1.26	1.35	1.45	1.54	1.63
	D	0.99	2.92	3.45	6.29	11.40	18.50	27.70	39.40	53.90	71,13	91.42	114.9
1/100	V	0.57	0.74	0.78	0.90	1.04	1.17	1.29	1.41	1.52	1.63	1.73	1.82
	D	1.11	3.29	3.91	7.07	12.80	20.73	31.15	44.32	60 46	79.80	102.5	128.9
1/77	V	0.65	0.85	0.90	1.03	1.19	1.34	1.48	1.61	1.74	1.86	1.97	2.09
	D	1.28	3.77	4.48	8.11	14.60	23.7	35.70	50.70	69.20	91.30	117.3	147.4
1/62	V	0.73	0.95	0.99	1.15	1.33	1.50	1.65	1.80	1.94	2.07	2.20	2.32
	D	1.42	4.21	4.99	9.03	16.30	26.4	39.70	56.41	77.00	101.4	130.5	164.0
1/50	V	0.82	1.07	1.12	1.29	1.49	1.68	1.85	2.01	2.17	2.32	2.46	2.60
	D	1.60	4.72	5.60	10.4	18.31	22.64	44.52	63.30	86.29	113.8	146.2	183.7
1/38	V	0.94	1.23	1.28	1.48	1.71	1.92	2.11	2.30	2.48	2.65	2.81	2.97
	D	1.84	5.41	6.43	11.6	20.96	33.92	50.92	72.38	98,66	130.1	167.1	210.0
1/31	V	1.04	1.36	1.42	1.65	1.90	2.13	2.35	2.56	2.76	2.95	3.13	3.30
	D	2.05	6.03	7.16	12.9	23.32	37.73	56.63	80.47	109.7	144.6	185.8	233.4
1/25	V	1.17	1.53	1.60	1.85	2.13	2.39	2.64	2.87	3.09	3.30	3.50	3.70
	D	2.30	6.77	8.03	19.5	26.14	42.28	63.46	90.16	122.9	162.0	208.0	261.4
1/20	V	1.32	1.72	1.79	2.07	2.39	2.68	2.96	3.21	3.46	3.700	3.92	4.14
	D	2,58	7.59	9.00	16.2	29.30	47.38	71.09	101.0	137.6	181.4	233.0	292.7
1/15	V	1.51	1.97	2.05	2.37	2.73	3.06	3.78	3,67	3.95	4.22	4.48	4.73
	D	2.96	8.68	10.3	18.6	33.50	54.15	81.24	115.4	157.2	207.3	266.1	334.2
1/12	V	1.73	2.25	2.35	2.71	3.13	3.51	3.87	4.21	4.53	4.84	5.13	5.41
	D	3.40	9.97	11.8	21.3	38.40	62.06	93.08	132.2	180.1	237.4	304.8	382.8
1/10	V	1.88	2.45	2.55	2.95	3.40	3.81	4 20	4.57	4.92	5.25	5.57	5.88
	D	3.69	10.8	12.8	23.1	41.70	67.40	101.1	143.5	195.5	257.7	330.8	415.5

V = Velocity in Metres/Second D = Discharge in Litres/Second

Conversion Factors — Rates of Flow

1 cu. metre/sec.
= 35.314 cu. ft./sec.

1 cu. metre/hr.
= 0.278 litres/sec.
= 3.668 Imp. gall./mt.
= 4.403 US gall./mt.

1 litre/sec.
= 0.0353 cu. ft./sec.
= 2.119 cu. ft./mt.
= 86400 litres/day
= 86.400 kilolitres/day
= 13.2 Imp. gall./mt.
= 15.87 US galls./mt.
= 3 6 cu metres/hr.

Slope		Pipe Diametres in mm											
		350	375	400	450	500	525	600	675	700	750	800	825
1/500	V	0.87	0.91	0.95	1.03	1.10	1.13	1.23	1.32	1.35	1.41	1.47	1 50
	D	0.08	0.10	0.12	0.16	0.21	0.24	0.35	0.47	0.52	0.62	0.74	0.80
1/417	V	0.96	1.00	1.05	1.13	1.20	1.24	1,35	1.45	1.49	1.55	1.61	1.65
	D	0.09	0.11	0.13	0.18	0.24	0 27	0.38	0 52	0.57	0.89	0.81	0.88
1/294	V	1.15	1.20	1.25	1.35	1.44	1.49	1.62	1.74	1.78	1.86	1.93	1.97
	D	0.11	0.13	0.16	0.21	0.28	0 32	0.46	0.62	0.68	0.82	0.97	1.05
1/250	V	1.25	1.31	1.36	1.47	1.57	1.62	1.76	1.89	1.93	2.02	2.10	2.14
	D	0.12	0.14	0.17	0.23	0.31	0.35	0.50	0 68	0.74	0.89	1.05	1.14
1/200	V	1.41	1.47	1.53	1.65	1.76	1.81	1.97	2.12	2.17	2 26	2.35	2.40
	D	0.13	0.16	0.19	0.26	0.34	0.39	0 56	0.76	0.83	1.00	1.18	1.28
1/154	V	1.61	1.68	1.75	1.88	2.01	2.07	2.25	2.42	2.48	2.58	2.69	2.74
	D	0.15	0.19	0.22	0.30	0.39	0.45	0 64	0.87	0.95	1.14	1.35	1 46
1/125	V	1.79	1.87	1.95	2.10	2.24	2.31	2.50	2 69	2.75	2.87	3.00	3.04
	D	0.17	0 21	0.24	0.33	0.44	0.50	0.71	0.96	1.06	1.27	1.50	1.63
1/100	V	2.01	2.10	2.18	2.35	2.51	2.58	2.81	3.02	3.08	3.22	3.35	3.41
	D	0.19	0.27	0.37	0.49	0.56	0.79	1.08	1.08	1.19	1.42	1.68	1.82
1/67	V	2.47	2.58	2.68	2.89	3.08	3.18	3.45	3.71	3.79	3.95	4.11	4.19
	D	0.24	0.28	0.34	0.46	0.60	0.69	0.97	1.33	1.46	1.75	2.10	2.24
1/50	V	2.86	2.99	3.11	3.34	3.57	3.68	3 99	4.29	4.38	4.57	4.76	4.84
	D	0.27	0.33	0 39	0.53	0.70	0.80	1.13	1.53	1.69	2.02	2.39	2.59
1/38	V	3.27	3.41	3.55	3.82	4 07	4.20	4.56	4.90	5.00	5.22	5.43	5.53
	D	0.31	0 38	0.45	0.61	0.80	0.91	1.29	1.75	1.93	2.31	2 73	2.96
1/31	V	3.63	3.79	3.95	4.24	4.53	4.66	5.06	5.44	5.56	5.80	6.03	6.14
	D	0.35	0.42	0.50	0.67	0.89	1.01	1.43	1.95	2.14	2.56	3.03	3.28

V = Velocity in Metres/Second D = Discharge in Litres/Second

1 litre/mt.
= 1440 litres/day
= 1.440 kilolitres/day
= 13 2 Imp. gall./hr.

1 kilolitre/day
= 0.695 litres/mt.
= 41.65 litres/hr.
= 0.01157 litres/sec.

1 Imp. gall./mt.
= 0.076 litre/sec.
= 272.8 litres/hr.

1 Imp. gall./hr.
= 0.076 litre/mt.

1 US gall./mt.
= 0.063 litre/sec.

1 cu. ft./sec. (cusecs)
= 0.0283 cu. metre/sec
= 1.69 cu metres/mt.
= 101.9 cu. metres/hr.
= 28 32 litres/sec.
= 373.7 Imp. galls /mt.
= 448.8 US galls./mt.

1 million litres/day
= 11.57 litres/sec.
= 694.445 litres/mt.
= 41666 667 litres/hr.
= 0.22 million gall/days

Slope		Pipe Diametres in mm: 900	975	1000	1050	1100	1200	1350	1500	1650	1800	1950	2100
1/5000	V	0.48	0.50	0.51	0.52	0.54	0.57	0.61	0.66	0.70	0.74	0.77	0.81
	D	0.30	0.37	0.40	0.45	0.51	0.65	0.88	1.16	1.49	1.87	2.31	2.81
1/3333	V	0.59	0.62	0.63	0.65	0.67	0.71	0.76	0.81	0.86	0.91	0.96	1.00
	D	0.37	0.46	0.49	0.56	0.64	0.80	1.09	1.44	1.84	2.32	2.85	3.47
1/2500	V	0.69	0.72	0.73	0.76	0.78	0.82	0.88	0.94	1.00	1.06	1.11	1.16
	D	0.44	0.54	0.58	0.65	0.74	0.93	1.27	1.67	2.14	2.69	3.31	4.02
1/2000	V	0.77	0.81	0.82	0.85	0.87	0.92	0.99	1.06	1.12	1.19	1.25	1.30
	D	0.49	0.60	0.65	0.73	0.83	1.04	1.42	1.87	2.40	3.02	3.72	4.51
1/1333	V	0.95	1.00	1.02	1.05	1.08	1.14	1.22	1.31	1.39	1.46	1.53	1.61
	D	0.61	0.75	0.80	0.91	1.03	1.29	1.75	2.31	2.96	3.72	4.58	5.56
1/1000	V	1.11	1.16	1.18	1.22	1.25	1.32	1.42	1.52	1.61	1.69	1.78	1.86
	D	0.70	0.87	0.95	1.05	1.19	1.49	2.03	2.68	3.44	4.31	5.31	6.45
1/667	V	1.36	1.43	1.46	1.50	1.54	1.63	1.75	1.87	1.98	2.09	2.91	2.29
	D	0.87	1.07	1.14	1.30	1.47	1.84	2.50	3.30	4.23	5.31	6.54	7.93
1/500	V	1.58	1.66	1.69	1.74	1.79	1.89	2.03	2.16	2.29	2.42	2.54	2.65
	D	1.01	1.24	1.32	1.51	1.70	2.13	2.90	3.82	4.90	6.15	7.57	9.18
1/333	V	1.95	2.04	2.08	2.14	2.20	2.32	2.49	2.66	2.89	2.97	3.12	3.26
	D	1.24	1.53	1.63	1.85	2.09	2.63	3.57	4.70	6.02	7.59	9.31	11.29
1/250	V	2.26	2.37	2.41	2.48	2.55	2.69	2.89	3.08	3.26	3.44	3.61	3.77
	D	1.43	1.77	1.89	2.15	2.42	3.04	4.13	5.44	6.97	8.75	10.77	13.06
1/200	V	2.53	2.65	2.70	2.78	2.86	3.01	3.24	3.45	3.65	3.85	4.04	4.22
	D	1.61	1.98	2.11	2.40	2.71	3.41	4.63	6.09	7.81	9.80	12.06	14.62
1/133	V	3.11	3.26	3.31	3.41	3.51	3.70	3.97	4.23	4.48	4.73	4.96	5.18
	D	2.00	2.44	2.60	2.95	3.34	4.19	5.69	7.48	9.59	12.03	14.81	17.95
1/100	V	3.60	3.78	3.83	3.95	4.06	4.28	4.60	4.90	5.19	5.47	5.73	5.99
	D	2.29	2.82	3.01	3.42	3.86	4.84	6.51	8.66	11.09	13.91	17.12	20.76
1/67	V	4.42	4.64	4.71	4.85	4.99	5.26	5.64	6.01	6.37	6.71	7.03	7.35
	D	2.81	3.46	3.70	4.20	4.74	5.94	8.08	10.62	13.61	17.07	21.01	25.47

V=Velocity in Metres/Second D=Discharge in Litres/Second

1 cu. ft. of water
= 28.32 litres or kg.
= 0.0283 cu. metres
= 6.24 Imp. galls.
= 7.48 US galls.

1 gall. (Imp. U K.)
= 4.546 litres
= 10 lbs.
= 0.16 cu. ft.
= 8 pints or 160 fluid ounces
= 1.20 US galls.

(Also see page 14/2)

1 litre
= 1 kilogram
= 2.2 lbs.
= 0.22 Imp. gall.
= 0.264 U.S. gall.
= 0.0353 cu. ft.

1 cu. metre of water
= 1000 litres or kg.
= 35.3 cu. ft.
= 220 Imp. galls.
= 264.8 US galls.

1 gall (US)
= 3.785 litres or kg.
= 0.833 Imp. gall.
= 8.33 lbs.
= 0.134 cu. ft.

Asbestos-Cement in "Normal" Condition or PVC Pipes with Chemically Cemented Joints

Slope		Pipe Diametres in mm: 50	75	80	100	125	150	175	200	225	250	275	300
1/250	V	0.37	0.49	0.52	0.60	0.70	0.79	0.87	0.95	1.03	1.10	1.17	1.24
	D	0.73	2.18	2.59	4.72	8.58	13.95	21.03	29.99	41.00	54.22	69.78	87.85
1/200	V	0.42	0.56	0.58	0.68	0.79	0.89	0.99	1.08	1.16	1.25	1.32	1.40
	D	0.83	2.47	2.94	5.34	9.69	15.76	23.74	33.84	46.25	61.14	78.67	99.02
1/154	V	0.49	0.65	0.67	0.78	0.91	1.03	1.14	1.24	1.34	1.43	1.52	1.61
	D	0.96	2.85	3.39	6.16	11.18	18.17	27.36	38.99	53.26	70.39	90.55	114.0
1/125	V	0.55	0.72	0.76	0.88	1.02	1.15	1.27	1.39	1.50	1.60	1.70	1.80
	D	1.08	3.20	3.80	6.91	12.52	20.33	30.60	43.60	59.54	78.66	101.2	127.3
1/100	V	0.62	0.82	0.85	0.99	1.15	1.30	1.43	1.56	1.69	1.80	1.92	2.03
	D	1.22	3.62	4.30	7.80	14.13	22.93	34.51	49.14	67.09	88.61	113.9	143.3
1/77	V	0.72	0.94	0.99	1.14	1.33	1.49	1.65	1.80	1.94	2.07	2.20	2.33
	D	1.41	4.17	4.96	8.99	16.28	26.40	39.71	56.53	77.15	101.9	131.0	164.8
1/62	V	0.80	1.06	1.10	1.28	1.48	1.67	1.84	2.01	2.17	2.32	2.46	2.60
	D	1.58	4.67	5.55	10.0	18.20	29.51	44.37	63.14	86.15	113.7	146.1	183.7
1/50	V	0.91	1.19	1.25	1.44	1.67	1.88	2.08	2.26	2.44	2.61	2.77	2.92
	D	1.78	5.28	6.27	11.3	20.51	33.24	49.97	71.08	96.95	127.9	164.4	206.8
1/38	V	1.05	1.38	1.44	1.66	1.92	2.16	2.39	2.60	2.80	2.99	3.18	3.36
	D	2.06	6.08	7.22	13.0	23.60	38.23	57.43	81.66	111.3	146.9	188.7	237.2
1/31	V	1.17	1.54	1.61	1.86	2.15	2.41	2.66	2.90	3.12	3.34	3.54	3.74
	D	2.31	6.80	8.07	14.0	26.36	42.68	64.09	91.11	124.2	163.8	210.4	264.4
1/25	V	1.33	1.73	1.81	2.09	2.42	2.72	3.00	3.26	3.51	3.75	3.98	4.20
	D	2.60	7.67	9.10	16.4	29.67	48.02	72.09	102.4	139.6	184.1	236.4	297.0
1/20	V	1.49	1.95	2.04	2.36	2.72	3.06	3.37	3.66	3.95	4.21	4.47	4.72
	D	2.94	8.64	10.2	18.0	33.39	54.01	81.05	115.1	156.9	206.9	265.6	333.6
1/15	V	1.72	2.25	2.34	2.71	3.12	3.51	3.87	4.20	4.52	4.83	5.12	5.41
	D	3.38	9.93	11.8	21.3	38.34	61.99	92.99	132.0	179.9	237.1	304.4	382.3
1/12	V	1.99	2.59	2.70	3.12	3.60	4.04	4.45	4.83	5.20	5.55	5.89	6.21
	D	3.90	11.4	13.6	24.5	44.14	71.33	106.9	151.8	206.8	272.5	349.8	439.2
1/10	V	2.17	2.82	2.95	3.40	3.92	4.39	4.84	5.26	5.66	6.04	6.40	6.76
	D	4.26	12.5	14.8	26.7	48.06	77.64	116.4	165.2	225.0	296.4	380.4	477.6

V=Velocity in Metres/Second D=Discharge in Litres/Second

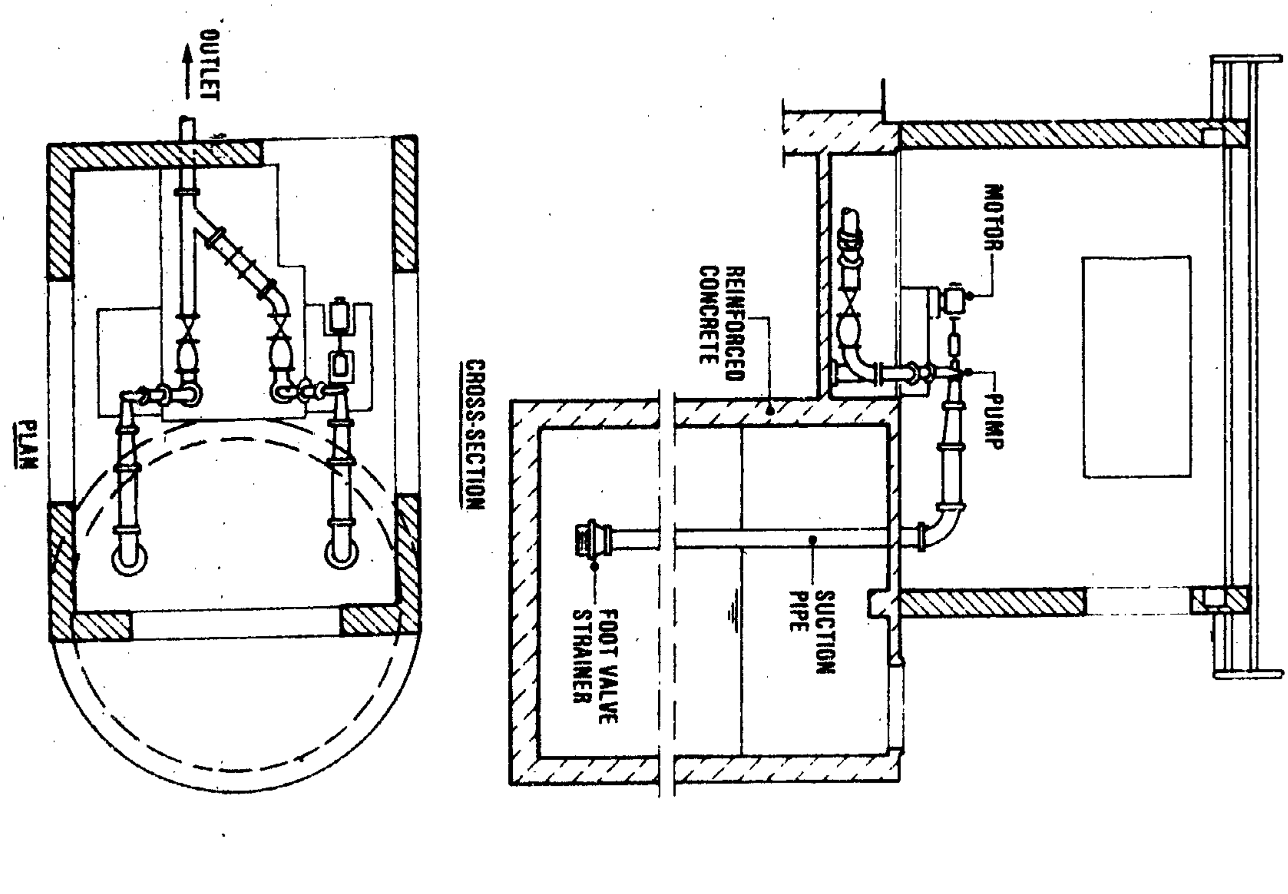
Water Horse Power (H.P.)—(Also see page 15/113)

$$= \frac{\text{Discharge in litres per sec. xH in metres}}{75}$$

$$\text{Kilowatt input to motor} = \frac{\text{B.H.P. x 0.73}}{\text{Motor efficiency}}$$

Equivalent Pipe Sizes

Diameter in mm	Diameter in mm: 75	100	125	150	200	250	300	350	400	450	500	550	600	750	900	1200
1200									15·59	11·61	8·92	7·03	5·65	3·24	2·05	1·0
1100								17·50	12.54	9·34	7·17	5·67	4·55	2·61	1·65	—
1000							20·23	13·47	7·85	7·31	5·64	4·44	3·57	2·05	1·26	—
900							15·58	8·41	7·59	5·65	4·34	3·42	2·73	1·58	1·0	—
825					34·55	19·78	12·54	8·52	6·11	4·55	3·49	2·74	2.21	1·27	—	—
750					27·00	15·54	9·84	6·54	4·80	3·57	2·74	2·16	1·74	1·0	—	—
675				42·95	16·61	9·96	7·59	5·16	3·70	2·75	2·11	1·67	1·34	—	—	—
600			50·50	32·00	15·58	8·92	5·65	3·84	2·75	2·05	1·57	1·24	1·0	—	—	—
550		70·96	40 60	25·73	12·53	7·17	4·55	3·09	2·16	1·65	1·26	1·0	—	—	—	—
500		55·96	32·05	20·29	9·88	5·66	3·58	2·43	1·71	1·30	1·0	—	—	—	—	—
450		42·71	24·63	15·58	7·25	4·34	2·75	1·87	1·34	1·0	—	—	—	—	—	—
400	65·77	32·01	18·31	11·60	5·65	3·23	2·05	1·39	1·0	—	—	—	—	—	—	—
350	47·14	22·74	13·15	8·32	4·05	2·32	1·47	1·0								
300	32·05	15·60	8·93	5·65	2·75	1 57	1·0									
250	20·21	9·88	5·66	3·38	1·74	1·0										
200	11·63	5·66	3·24	2·05	1·0											
150	5·66	2·75	1·58	1·0												
125	3·58	1·75	1·0													
100	2·05	1·0														
75	1·0															



WATER SUPPLY

Pumping Station with Horizontal Pumps (self-priming)

FLOW OF WATER IN ENCRUSTED CAST IRON PIPES

TABLE GIVING DISCHARGE IN GALLONS PER MINUTE

Bore of Pipe in inches

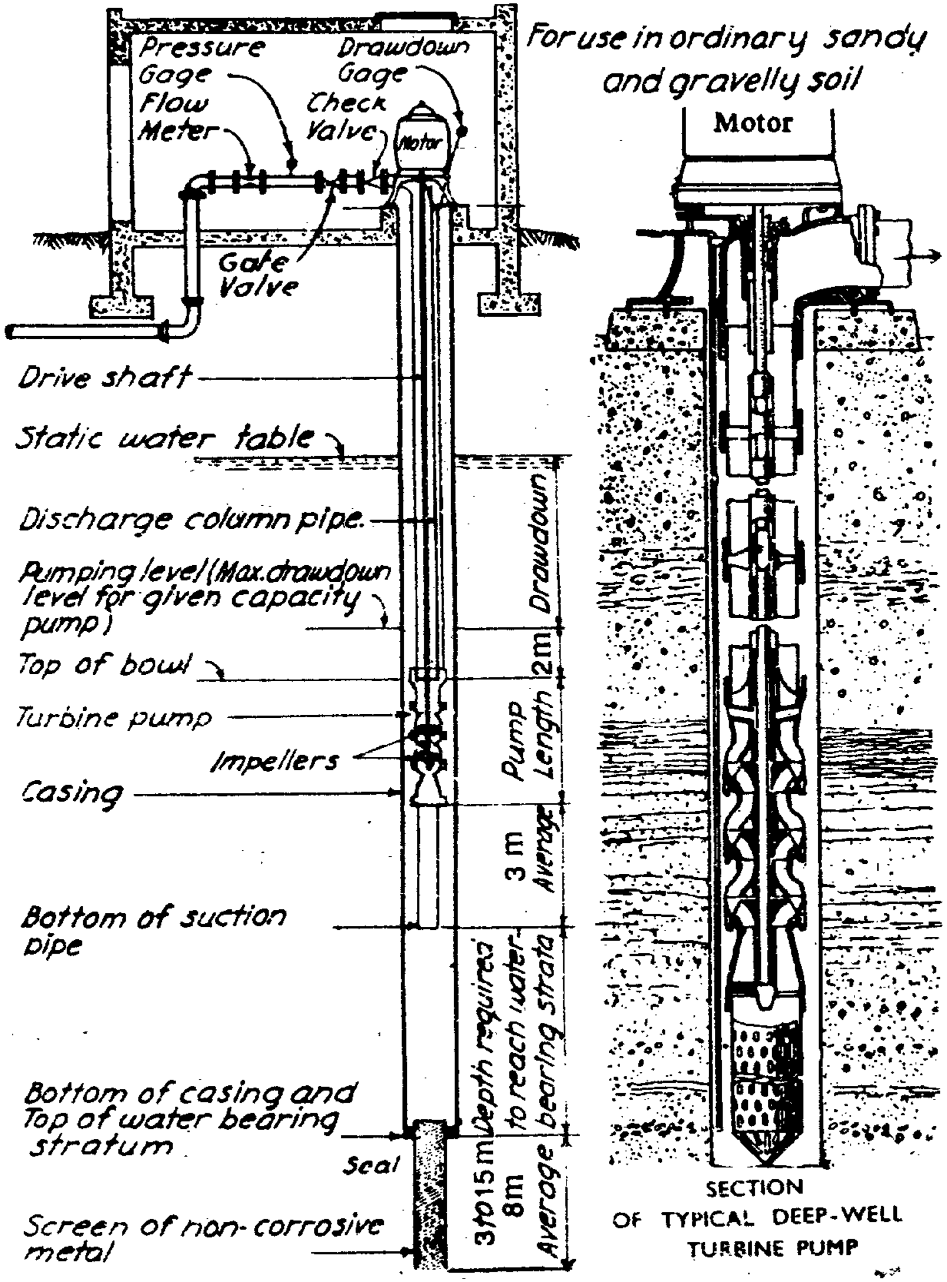
Hydraulic Gradient—1 in……	3	4	5	6	7	8	9	10	12	14	15	16	18	20
10	90	196	365	580	910	1290	1800	2370	4000	5840	7150	8620	11600	15800
15	74	158	293	480	740	1035	1460	1925	3200	4750	5700	6900	9500	12800
20	63	137	245	418	635	910	1250	1780	2745	3980	4950	5900	8200	11000
30	52	113	207	335	518	759	1110	1350	2220	3320	4050	4840	6600	8900
40	45	96	180	285	458	640	890	1170	2025	2780	3440	4100	5600	7600
50	39	88	160	264	408	565	780	1040	1720	2510	3030	3670	5000	6800
60	37	79	148	239	347	516	715	940	1550	2280	2740	3340	4500	6150
70	34	73	138	220	325	470	655	870	1440	2100	2500	3040	4200	5600
80	31	68	128	205	308	440	620	810	1335	1960	2380	2850	3950	5200
90	29	64	118	190	290	420	580	750	1240	1850	2240	2660	3660	4900
100	28	60	108	180	260	395	530	710	1170	1770	2100	2500	3500	4700
120	25	53	100	164	245	354	480	645	1060	1600	1950	2320	3200	4300
150	23	48	89	147	215	315	440	575	950	1420	1725	2150	2800	3800
200	19	42	77	131	190	265	375	500	820	1228	1495	1800	2450	3300
250	18	38	68	112	170	240	335	445	730	1100	1325	1600	2190	2920
300	16	34	64	103	153	220	305	412	668	1005	1210	1420	1950	2620
350	—	30	59	95	142	208	280	368	605	908	1090	1300	1800	2400
400	15	28	53	87	130	188	260	342	560	850	1000	1200	1680	2170
450	14	27	49	81	122	178	245	322	525	805	960	1130	1580	2100
500	—	26	47	78	115	170	230	305	500	755	910	1040	1500	2030
550	13	—	46	75	108	160	218	290	475	715	860	1010	1420	1900
600	12	25	44	70	105	153	209	275	455	682	825	980	1360	1840
650	—	—	42	68	100	144	200	260	435	652	790	950	1300	1740
700	—	—	41	65	97	138	190	250	420	615	762	910	1240	1680
800	11	24	38	60	90	129	178	235	400	586	705	850	1160	1570
900	10	21	35	58	85	121	170	222	367	550	660	800	1100	1485
1000	8	18	33	53	80	113	160	210	345	515	625	750	1020	1390
1200	—	16	30	49	72	103	146	192	315	475	563	680	920	1270
1500	7	14	26	44	65	93	129	171	278	423	500	600	840	1100
2000	6	12	23	37	56	79	110	146	236	362	435	520	720	970
3000	—	9	18	30	45	65	90	118	197	293	352	420	580	780
4000		—	15	26	39	54	78	102	170	249	296	360	490	665
5000			14	23	35	49	69	91	152	220	265	320	435	590
6000			13	20	32	45	63	84	136	193	244	276	400	530
7000			—	19	29	40	56	80	123	186	221	265	368	490
8000				18	28	38	52	70	114	174	208	247	344	465
9000				—	25	35	50	66	108	165	195	232	322	430
10000					23	34	48	63	104	157	188	225	307	420

Hydraulic Gradient—1 in……	21	22	24	27	28	30	33	36	40	44	48
10	18400	21000	26300	36700	40000	—	—	—			
15	14800	17000	21400	29500	32600	40000	—	—			
20	12800	15600	18400	25400	27800	34000	44300	—			
30	10200	11600	13900	20600	22500	27400	36000	—			
40	8750	10000	12800	17600	19400	23400	30600	39300	—	—	
50	7800	8850	11200	15800	17500	21000	26400	35100	—	—	
60	7100	8100	10200	14200	15600	18900	24700	31500	42800	—	
70	6500	7400	9400	13000	14200	17400	22500	28500	39450	—	
80	6150	6950	8650	12100	13300	16500	21400	27000	36800	—	
90	5650	6500	8250	11200	12500	15400	19150	25500	34500	43500	—
100	5300	6150	7800	10600	11700	14600	18800	24000	32250	42000	60000
120	4950	5600	7100	9750	10700	13200	17300	21880	29450	38200	52000
150	4350	5000	6400	8700	9500	11650	15400	19500	26100	33800	43000
200	3750	4270	5350	7500	8400	10000	13200	17000	22550	29000	37000
250	3370	3940	4900	6750	7350	9000	11800	15000	20250	25800	33700
300	3040	3450	4350	6000	6510	8100	10500	13500	18150	23200	30000
350	2740	3150	4000	5490	6000	7350	9500	12300	16650	21400	27000
400	2550	2920	3750	5100	5610	6900	9000	11500	15600	20000	25500
450	2420	2740	3530	4850	5250	6500	8500	10900	14650	18800	24000
500	2320	2600	3340	4570	5000	6200	8100	10200	14250	17800	22500
550	2170	2470	3150	4350	4720	5850	7700	9750	13100	17200	21700
600	2100	2400	3000	4200	4570	5550	7200	9300	12500	16400	20800
650	2000	2320	2850	3980	4300	5300	7050	9000	12000	15650	20000
700	1940	2170	2750	3900	4200	5100	6680	8500	11450	14900	19000
800	1800	2050	2560	3640	3900	4750	6250	8100	10500	14000	17900
900	1700	1940	2420	3370	3670	4500	5800	7500	10000	13100	17000
1000	1610	1810	2280	3150	3450	4230	5450	7100	9500	12200	15800
1200	1460	1650	2090	2850	3150	3850	5000	6450	8625	11000	14300
1500	1280	1480	1860	2550	2750	3450	4500	5700	7700	10000	12800
2000	1100	1270	1620	2200	2400	2900	3850	4950	6600	8150	11000
3000	900	1020	1280	1790	1950	2400	3150	4000	5250	6900	8850
4000	770	880	1090	1530	1660	2000	2600	3400	4600	5850	7500
5000	680	780	975	1350	1485	1820	2380	3040	4000	5180	6750
6000	620	705	890	1225	1340	1660	2140	2740	3680	4750	6150
7000	565	645	825	1120	1240	1530	1900	2510	3440	4350	5600
8000	530	604	765	1050	1100	1425	1880	2380	3230	4130	5000
9000	500	565	720	990	1070	1340	1760	2240	3000	3870	4500
10000	465	540	684	955	1040	1275	1680	2130	2850	3680	4000

Velocity 1, 2, 3, 4, 5, 8, 10 Ft. per Second

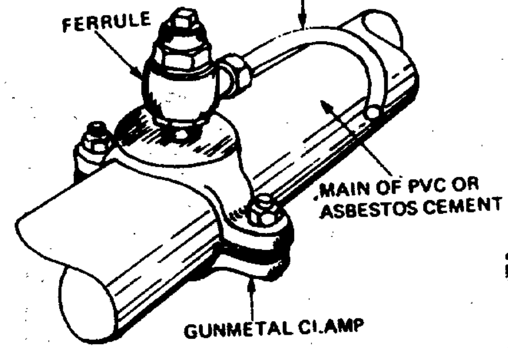
The table is based on a roughness co-efficient of 90 in Hazen and Williams' formula.
New clean cast iron pipes will carry about 33% more discharge with the same bore and gradient.

Single Cased Well

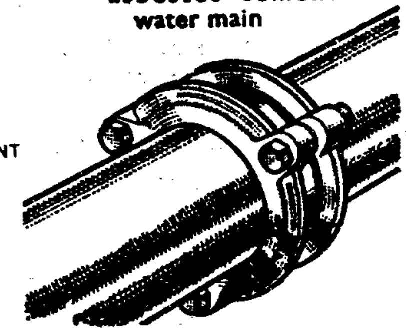


SECTION OF TYPICAL DEEP-WELL TURBINE PUMP

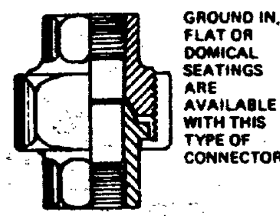
Annexes



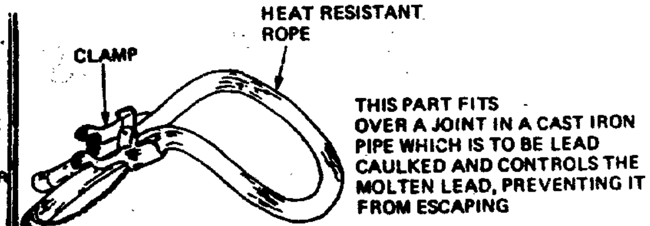
Bolt on Ferrule Connection for Asbestos Mains



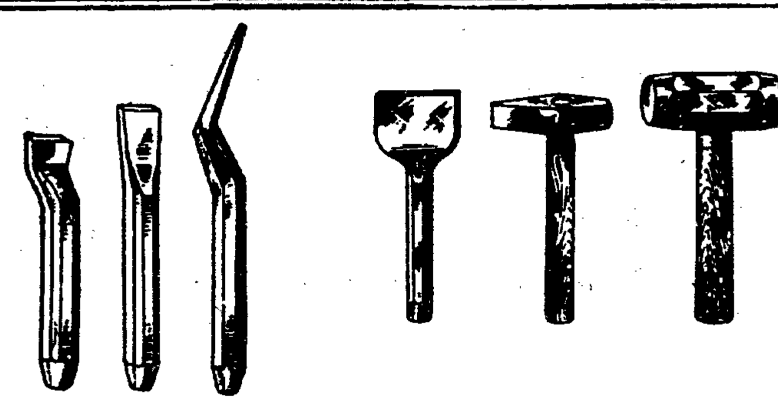
Cast-iron detachable joint for asbestos cement water main



Union Connector for Pipes

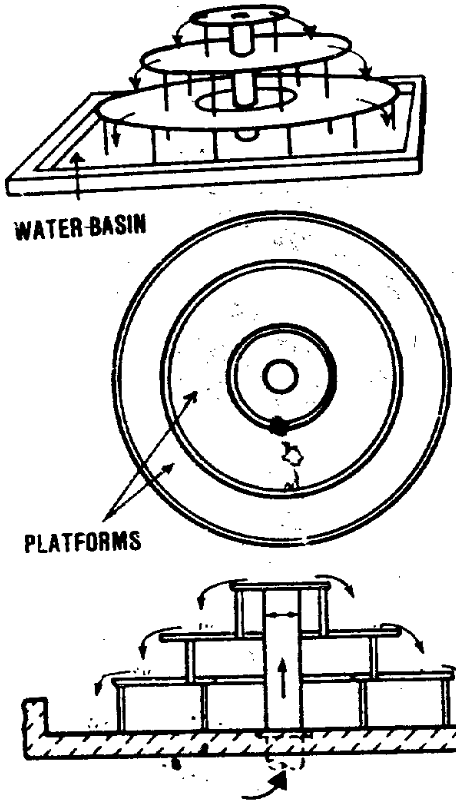


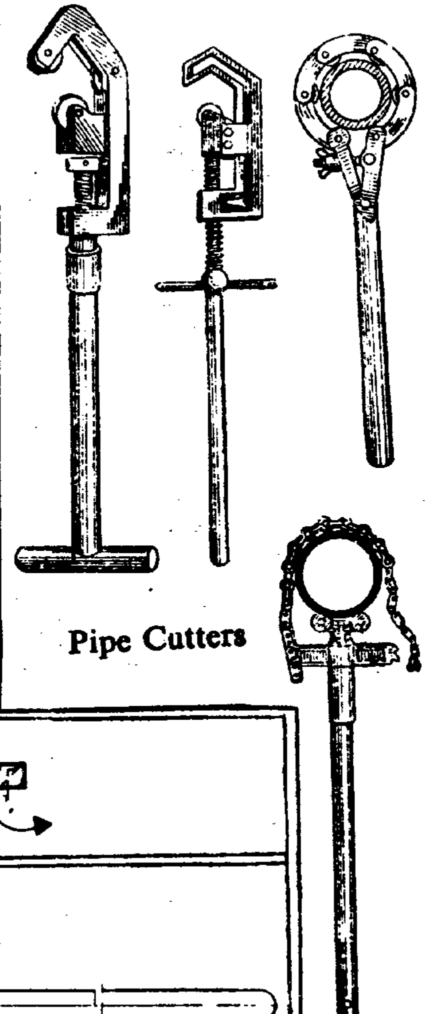
Jointing Ring (Asbestos)



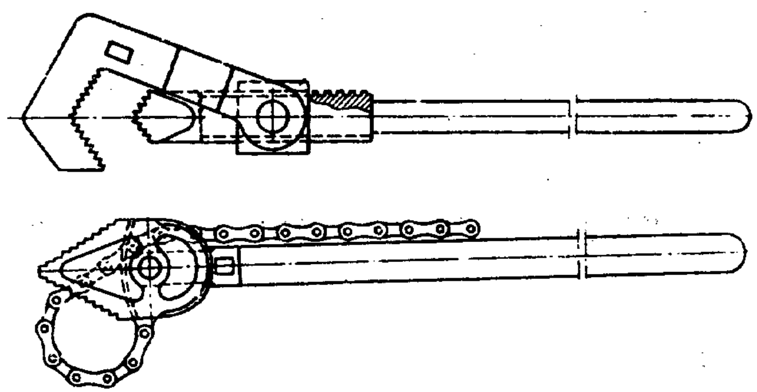
Caulking tools

Multiple-Platform Aerator



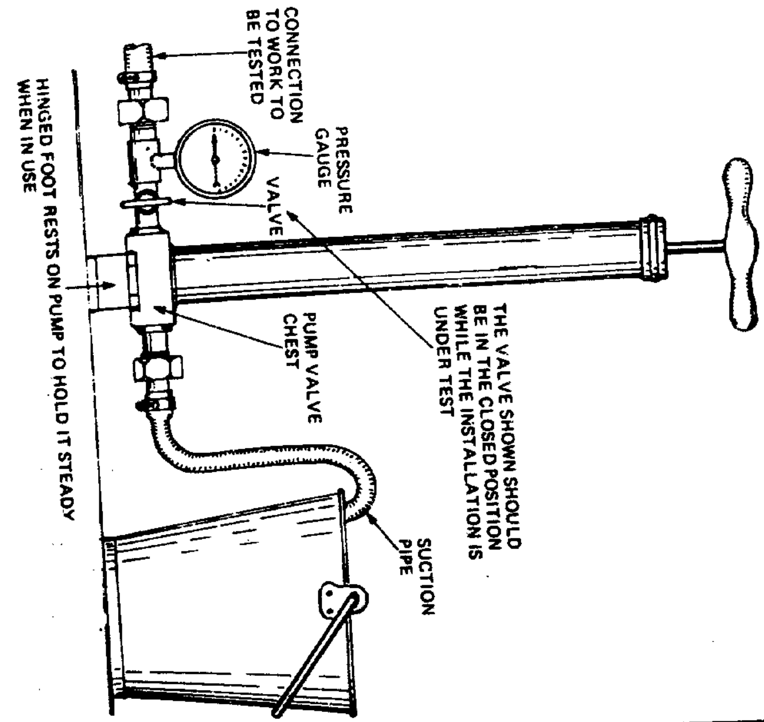


Pipe Cutters

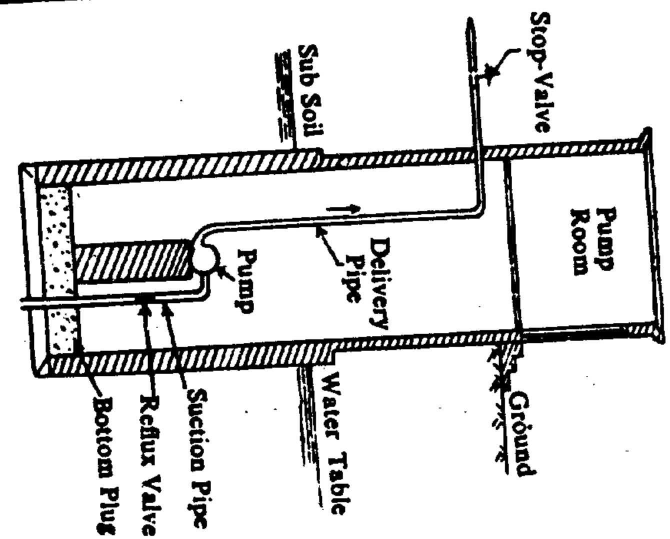


Pipe Wrenches

Typical force pump used to pressure test water services



Annexes



Flow of Water in New House Service Pipes

Discharge in c. ft. per minute

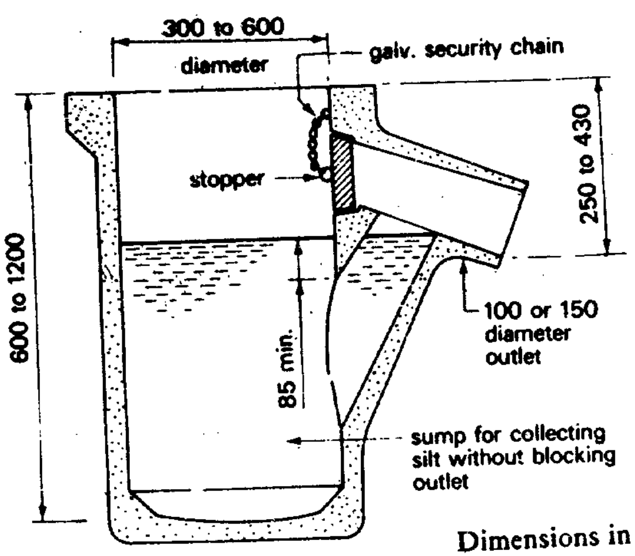
(One cu ft. = 28.32 litres)

Pressure in Main lbs./sq. in.	Nominal internal dia. of pipe in inches							
	1	1/2	3/4	1-1/2	2	3	4	6
Through 35 ft. (10.7 m) of service pipe, no back pressure								
30	1.10	3.01	6.13	16.6	33.3	88.2	174	445
40	1.27	3.48	7.08	19.1	38.5	102	201	513
50	1.42	3.89	7.92	21.4	43.1	114	224	574
60	1.56	4.26	8.67	23.4	47.2	125	246	669
75	1.74	4.77	9.70	26.2	52.7	139	275	703
100	2.01	5.50	11.2	30.3	60.9	161	317	812
130	2.29	6.28	12.8	34.5	69.4	184	362	926
Through 100 ft. (30 m) of service pipe, no back pressure								
30	0.66	1.84	3.78	10.4	21.3	58.2	118	317
40	0.77	2.12	4.36	12.0	24.6	67.2	136	366
50	0.86	2.37	4.88	13.4	27.5	75.1	153	410
60	0.94	2.60	5.34	14.7	30.1	82.3	167	449
75	1.05	2.91	5.97	16.5	33.7	92.0	187	502
100	1.22	3.36	6.90	19.0	38.9	106	216	579
130	1.39	3.83	7.86	21.7	44.3	121	246	660
Through 100 ft. (30 m) of service pipe and 15 ft. (4.6 m) vertical rise								
30	0.55	1.52	3.11	8.57	17.6	47.9	96.2	261
40	0.66	1.81	3.72	10.2	21.0	57.2	116	311
50	0.75	2.06	4.24	11.7	23.9	65.2	132	354
60	0.83	2.29	4.70	12.0	26.5	72.3	147	393
75	0.94	2.59	5.32	14.6	30.0	81.8	166	445
100	1.10	3.02	6.21	17.1	35.0	95.6	194	520
130	1.26	3.48	7.14	19.7	40.2	110	223	597
Through 100 ft. (30 m) of service pipe and 30 ft. (9.1 m) vertical rise								
30	0.44	1.22	2.50	6.80	14.1	38.6	78.5	212
40	0.55	1.53	3.15	8.48	17.8	48.7	99.0	267
50	0.65	1.79	3.69	10.2	20.8	57.0	116	342
60	0.73	2.02	4.15	11.5	23.4	64.2	131	352
75	0.84	2.32	4.77	13.2	27.0	73.8	150	404
100	1.00	2.75	5.65	15.6	31.9	87.4	178	479
130	1.15	3.19	6.55	18.1	37.9	101	206	555

Pressure in lbs./sq. in. x 0.7 — head of water in metres

SECTION 16
DRAINAGE & SEWERAGE

Page



Dimensions in mm

Street Gulley

1. SURFACE DRAINAGE

This subject has been discussed in detail in Section 19—"Bridges and Culverts".

Another useful formula for calculating rainfall run-off drainage design is :

$$Q = \frac{CAR\sqrt[4]{S/A}}{455}$$

Where : Q=run-off in cubic metres per second ;
C=co-efficient/impermeability factor of the surface—taken as 0.50 in rural areas ; 0.90 in suburban areas and 0.75 average ;
A=area in hectares ;
R=max. rainfall intensity over the entire area—usually taken as 25 to 75 mm per hour ;
S=slope of the ground surface in metres per thousand.

Some engineers recommend the following run-off for the design of surface drains in town areas :—

Av. annual rainfall—cm	up to 50	50 to 100	100 to 200	200 to 300	above 300
Run-off per hour—mm	3	6	12	20	25

In town and developed areas more of the rainwater finds its way into the drains than in rural districts. The run-off decreases with increase in drainage area. (See Table below)

For calculating the sizes of drains or sewers, the following figures of rainfall may be taken for town areas (where nearly all the streets are covered with impervious pavings) in regions of average rainfall of 750 to 1000 mm. The town should be divided into separate sectors or areas for the particular branch and main sewers :

Drainage area in hectares	Rainfall per hour in mm	Drainage area in hectares	Rainfall per hour in mm	Drainage area in hectares	Rainfall per hour in mm
8	25	40	12	160	8
10	23	50	11	180	8
12	21	60	11	200	7
14	19	70	10	220	6
16	17	80	10	240	6
18	15	100	10	280	6
20	13	120	10	320	5
30	12	140	9	400	5

2. DESIGN OF TOWN DRAINS AND SEWERS

Systems of Drainage. There are two principal systems known as the *combined* and *separate* systems. In the former system, one set of drains or sewers is provided for the removal of both the soil-sewage and the rain water. In the separate system, two sets of drains are provided, one for the soil-sewage and the other for rain water (or one underground sewer for sewage and one surface drain for rainwater). Both the systems have their advantages and disadvantages and in most cases a *partially separate* system is considered most suitable.

In the case of a totally separate system two branch drains or sewers in each street and two house connections for each building are necessary and it not only becomes 1½ to 2 times more costly than the combined system but also more complicated. Separate system is generally suitable for districts where the average rainfall per year exceeds 750 mm and a combined system where the rainfall is small. In a combined sewer, which will be large and deep, it is not possible to obtain a self-cleansing velocity during the dry weather when the flow is very small. Where a suitable outfall at low level is not available and the sewage has to be pumped, the separate system has definite advantages as the storm water has not then to be treated. In India Bombay, Calcutta and Madras have separate systems while most of the other towns have drainage on partially combined system.

In the partially combined (or partially separate) system, the greater part of the rainwater is passed to the surface water sewers or open drains, but the run-off from roofs, paved yards and streets is discharged into the soil sewers.

Surface (or Open) Drains in Small Towns are generally designed for combined sullage and (some) storm water flows. For the quantity of sullage, it is generally assumed that 60 to 90 per cent of the water-supply will find its way to the drains which must be large enough to carry the same in 8 hours, as the rate of flow is not uniform throughout the day. Therefore, the size of the drains should be 3 times the average flow per hour (called "dry whether flow"). Where there is no water-supply and the inhabitants depend upon wells and hand-pumps, water will be used much less and a lot of it will be spilled outside and will not be taken by the drains. In such cases, an allowance of 25 to 45 litres per head per day will seem sufficient according to the situation and the habits of the inhabitants, and availability of water.

Soil Sewage is the sewage from water closets, slop sinks and urinals, etc., plus sullage-water.

Sullage is waste water from bath-rooms, lavatory basins, kitchen sinks, courtyards and roof washings, etc., and which does not contain human or animal excreta. Term also used for all types of foul waters.

As regards the rainwater, it is very expensive to provide for the maximum flood water which may occur perhaps only once every fifty years; therefore, only a reasonable figure is taken as the heavy floods are only of short duration. Where, however, the high cost is justified, provision may be made for the exceptional rainstorms, but where the cost is a material factor and the damage caused by an exceptional storm is not likely to be

great, provision may be made only for a storm which occurs once every five years or less. In Britain, branch sewers are usually designed for one storm (av.) a year and trunk sewers up to one storm in five years. Sullage water is usually led through intramural drains and storm water flow through the main drains. (See also further under "Capacity of Sewers.")

Shape of Street Drains. For small flows, semi-circular drains or some modifications of the same are generally adopted but for large flows, especially drains taking up both sullage and storm water, peg-top sections are preferable. The cunnette portion of the drain is assumed to carry the sullage flow and the full section storm water. Narrow and deep drains have increased velocities and are suitable for flat slopes.

Sides of open drains may be with 1/2 to 1 slope and lined with half bricks and clay puddling where necessary, or make vertical walls (say one or half-brick thick) according to the nature of the soil.

Size of Sewers for Different Systems

Volume of Sewage. This is based on the consumption of water. It is generally assumed that half the daily consumption of water occurs within 6 hours. This gives an average peak consumption of:

Daily consumption $\div 2 \times 6$ (litres per head per hour).

The amount of sewage flow per day per person generally varies from 80 to 150 U.S. gallons in America, from 25 to 45 Imperial gallons in England and from 100 to 200 litres in Western European countries. American sewages are generally three times greater in volume than English sewages but are of half the strength (as they are much more dilute due to greater consumption of water per capita) of the latter.

Size of Sewers. If maximum rate of flow is taken twice the average peak consumption, the capacity of a sewer will be four times the average flow of 24 hours (dry weather flow). This will allow not only for the fluctuations of the hourly rate of flow but also for a small quantity of rain-water run-off or infiltration as the peak domestic flow is not usually more than three times the average dry weather flow. An average sewage flow of 135 litres per capita per day (including bath and sullage water) will do for most of the towns.

Where it is proposed to discharge part of the rainwater (from roofs, paved areas, etc.) into the soil sewer, the size of the sewer may be increased to six times the average dry weather flow, and also where the future extensions of the system cannot be accurately estimated. Storm water flow above this is diverted to a separate storm water drain. It is not considered good practice to make sewers larger than six times the average dry weather flow as too large a sewer capacity means low velocity of flow resulting in deposits.

According to IS : 1742—For large groups of houses, schools, public institutions, hospitals, factories etc., maximum rate of flow of foul water for the design of sewers shall be taken three times the average dry-weather flow, flowing half-full with a minimum self-cleansing velocity of 0.75 m/sec. A good average rule is to allow for a flow of liquid wastes from buildings at the rate of 0.03 cu.m/min. per 100 persons based on a water consumption of 135 litres per head per day.

Some engineers recommend as follows :

(A) All laterals, branches and outfall sewers should be designed for a peak flow at the rate of 3 times the dry weather flow for population below one lakh ; and the outfalls may be designed for 2.5 times the dry weather flow if the population contributing to the flow in the outfall sewer is more than ten lakhs.

Sewers of 350 mm dia. and below should be designed to run half-full at the peak flow, and sewers of 450 mm dia. and above for two-third full. Sewers of size larger than 900 mm dia. may be designed to run three-fourth full.

(B) Laterals to be designed for 4 to 6 times the average dry weather flow, branch sewers for 3 times, main sewers for 2.5 times, and trunk and outfall sewers for twice the average dry weather flow. Average daily discharge is multiplied by these figures to give the design discharge for the particular sewers and which are normally designed to run 2/3rd full. It is considered that some space need be left in a sewer for gases.

The amount of dry weather sewage is generally negligible as compared to the storm water. The following assumptions are generally made for the quantity of storm water :—

Where rainfall is heavy and frequently exceeds 25 mm per hour, drains should be designed to carry a flow of 25 mm per hour from the area under consideration in towns and 12 mm per hour in country side. In medium rainfall districts, an allowance of 6 mm in urban and 4 mm per hour in rural areas may be taken. In dry districts a run-off of 6 mm an hour may be taken for main drains and 4 mm to 3 mm for branch drains.

Where volume of sewage is based on the water supply, it should be considered that a portion of the water supply is lost by evaporation, watering of lawns and gardens, and washing of roads, etc.

Sewage production is taken from 75 per cent to 90 per cent of the water supplied to the community.

Scope for future developments (for about 30 years) of the area and change of habits of inhabitants during that period should be considered. (See under "Preparation of Drainage Schemes.") Design proceeds from the most remote point of the system downwards, for the main drains as well as the branches.

Storm-water Overflow is a weir formed by the side of a sewer to drop the extra flow to a storm water channel as soon as a certain level of flow in the sewer is exceeded. There are a number of simple devices for such an arrangement.

Surface water from streets should pass through catch-pits before reaching the combined or storm sewer, so that some grit be arrested.

Importance of Velocity of Flow in Drains and Sewers

The sullage water must flow in the sewers at all times with velocity sufficient to keep the solid matter moving. The velocity should neither be too slow nor too great. With too slow velocity, the solid matter will settle down at the bottom of the sewer, and a too great velocity is likely to erode [illegible] velocity of 0.61 m/sec. and

a maximum velocity of 2.4 m/sec. is generally taken for design of sewers. In case of combined sewers, the velocity of flow should not be less than 0.75 m/sec.

Self-Cleansing Velocities. It is essential that all sullage drains have self-cleansing velocities as far as possible so that there are no accumulations in the sewers and the sewage does not become septic. In India sewage has been found to get septic after six hours whereas, it takes over about 10 to 12 hours in cold countries. In cold countries, a velocity of 0.61 m/sec. for large sewers and 0.76 m/sec. for medium and small size sewers, has been found satisfactory. In India, higher velocities are necessary for the climate and the habits of using ashes, fibrous materials and grit for cleansing of pots and pans, and should be at least 0.76 to 0.91 m/sec. for open drains and 0.76 to 1.1 m/sec. in sewers, to prevent deposition of grit and other solid matter. Greater velocities are required for combined sewers than for sewers carrying only soil sewage.

Sewer pipelines should follow the natural slope of the ground surface so that they will always lie at the same minimum depth. The minimum slope for sewers should be not less than 1 in 145 for 150 mm diameter pipes, and not less than 1 in 200 to 250 for 200 mm diameter pipes.

Slopes should be as steep as possible in the upper lengths while in lower reaches they may be flatter. In the intramural drains slopes should never be flatter than 1 in 250, and in outfalls, 1 in 500 to 800. Intercepting drains should have slopes of between 1 in 300 in the upper lengths and 1 in 500 towards the ends. The slopes will be adjusted according to the designed velocity.

Formulae for calculation of velocities are given in Section 14—"Hydraulics".

In designing a sewer or a drain, it is necessary to adopt such diameter and gradient as will ensure the attainment of the desired velocity at least during the periods of peak flow. It is desirable to investigate the design on the basis of minimum flow conditions. Sewers are generally considered to flow only one-quarter full for calculation of the velocity, which is only about 0.7 of the velocity when the sewer is flowing full. Functions of flow in a circular pipe for various depths are given in the Section on "Hydraulics," and at page 14/24. Smaller sewers require greater inclination than larger ones, (bigger-sized sewers develop comparatively higher velocities and can, therefore, be laid at flatter slopes) and stone-ware glazed pipes lesser inclination than brick sewers.

The velocity of flow shall not be more than 1.83 to 2.13 m/sec. for brick drains, 2.4 m/sec. for concrete drains, 3.0 m/sec. for cemented drains, and 4.6 m/sec. for vitrified pipes or drains. The velocity in force mains must be at least 0.3 m/sec., when the lowest duty pump is working.

The carrying capacity of a sewer should be adequate but not excessive. Subject to velocity considerations, it is false economy to save a little by providing a smaller diameter at the expense of possibly having to duplicate it at some future date. In the case of branch sewers and those serving small areas a relatively larger margin of capacity is desirable than in the case of trunk sewers. No pipe street sewer should be of a smaller diameter than 150 mm (prefer 200 mm) even though calculations might show that a pipe of much smaller capacity would do all the work required.

Table of Velocity and Discharge in Cement Concrete Drain Pipes Flowing Full

Gradient 1 in		Dia. of Pipe in mm 100	150	230	300	Gradient 1 in		Dia. of Pipe in mm 150	230	300
10	V	1.81	2.36	—	—	90	V	0.78	1.01	1.23
	D	0.92	2.82	—	—		D	5.19	2.72	6.05
15	V	1.47	1.91	—	—	100	V	0.75	0.95	1.19
	D	0.74	2.31	—	—		D	0.84	2.52	5.75
20	V	1.29	1.68	2.18	—	120	V	0.68	0.88	1.09
	D	0.64	1.97	6.01	—		D	0.78	2.36	5.26
30	V	1.06	1.36	1.78	2.15	150	V	0.61	0.78	0.95
	D	0.51	1.59	4.88	10.9		D	0.69	1.89	4.69
40	V	0.95	1.16	1.53	1.88	175	V	0.54	0.75	0.90
	D	0.44	1.35	4.25	9.40		D	0.61	1.89	4.20
50	V	0.85	1.05	1.36	1.66	200	V	0.51	0.68	0.85
	D	0.40	1.21	3.76	8.39		D	0.59	1.80	4.06
60	V	0.74	0.95	1.26	1.53	250	V	0.48	0.61	0.75
	D	0.35	1.11	3.42	7.59		D	0.52	1.62	3.50
70	V	0.68	0.88	1.16	1.40	300	V	—	0.59	0.68
	D	0.33	1.02	3.10	6.99		D	—	1.45	3.25
80	V	0.62	0.81	1.09	1.33	350	V	—	0.51	0.62
	D	0.30	0.95	2.91	6.48		D	—	1.32	2.97

V—is Velocity in Metres per Second.

D—is Discharge in Cubic Metres per Minute.

A pipe flowing quarter-full has a velocity of 0.72 and discharge 0.14 of when running full. When flowing half-full, it has a velocity of 1.00 and discharge 0.50. Maximum velocity of 1.13 occurs when the depth of flow is 0.82, and maximum discharge of 1.068 when the depth of flow is 0.94.

Sewers are normally designed as running 2/3rd full leaving top 1/3rd for gases.

Sewers (cement concrete) with small average flow should have the following minimum gradients to give a velocity of about 0.85 m/sec. when flowing quarter-full.

Pipe Dia. in mm	100	150	200	230	300	380	450	530
Gradient 1 in—	40	70	75	100	150	200	250	300
Discharge in cu. m/min.	0.13	0.31	0.65	0.76	1.23	2.01	3.29	4.60

The following gradients will give self-cleansing (minimum desired) velocities of about 0.75/sec. in cement concrete pipes when flowing half-full.

Pipe Dia. in mm	100	150	200	230	250	300	380	450	530
Gradient 1 in—	57	100	145	175	195	250	500	650	800
Discharge in cu. m/min.	0.18	0.42	0.73	0.93	1.10	1.70	2.60	3.30	5.30

Limiting gradients to give velocities of about 1.4 m/sec. (in pipes of cement concrete—not very smooth surface) when flowing half-full :

Pipe Dia. in mm	150	230	300	450	530	600	750	900
Gradient 1 in—	40	60	90	150	170	200	250	300
Discharge in cu. m/min.	0.80	2.00	3.60	8.40	13.0	19.0	27.0	40.0

These gradients give a velocity of 2.4 m/sec. when flowing half-full :

Pipe Dia. in mm	100	150	200	230	250	300
Gradient 1 in—	5.6	9.7	14	17	19	24.5
Discharge in cu. m/min.	0.59	1.32	2.40	2.98	3.90	5.30

Large unlined drains should not have a steeper slope than 1 in 1200 otherwise scouring will occur. Under culverts, drains should be given steeper gradients say, about double.

In very flat areas where there is difficulty in obtaining the minimum grades, it is bad practice to enlarge the size of the sewer to obtain higher velocities. In fact, the pipes have lower velocities when depth of flow is reduced. Even a slight reduction in velocity due to either increase in section, increase in roughness of surface or resistance at bends, reduces the transporting power of a liquid considerably.

1.4 m/sec. is generally called the *limiting velocity* for the sewers. It is the velocity beyond which if the sewage flows, a scouring action is exerted on the walls of the sewer and which is likely to damage the inside smoothness. Where the velocity of flow must be greater than 2.4 m/sec., cast iron pipes should be used.

If, in order to conform to the natural slope of the ground, a sewer would normally have to be laid at such a gradient as to produce a velocity in excess of the above figures, the surplus available fall should be absorbed by the introduction of backdrop manholes or drop walls and water cushions at appropriate intervals, and the sewer thus carried down the slope in a series of steps. The force of the fall can also be broken by staggered horizontal plates, a flight of steps, or by means of a well or sump at the bottom from which the sewage overflows to the low-level sewer.

Side drains must enter main drains tangentially or through curves at junctions ; to secure a really self-cleansing junction the incoming side drain must be ramped down so that it joins tangentially in section as well as tangentially in plan. This costs a little more but a junction so constructed is always self-cleansing. Another important point at the intersections of sewer pipes, or wherever there is a change of diameter, is to see that the tops (or crowns) of the different diameter pipes are kept level, one with the other, as far as possible.

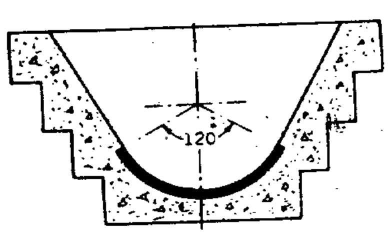
TABLE SHOWING VELOCITY AND DISCHARGE IN TYPE DRAINS

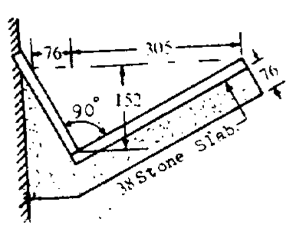
A sq. cm	R cm	Type of Drain		Slope in— 25	50	75	100	150	200	300	400	500	600	700	800	900	1000	Type of Drain
91.5	3.8	I	V	1.89	1.32	1.08	0.95	0.77	0.66	0.54	0.47	0.42	0.38	0.36	0.33	0.31	—	I
			Q	1.02	0.72	0.59	0.50	0.42	0.35	0.29	0.25	0.23	0.20	0.20	0.18	0.17	—	
251	6.4	II	V	2.67	1.88	1.54	1.34	1.09	0.94	0.77	0.67	0.59	0.55	0.51	0.47	0.44	0.42	II
			Q	4.00	2.82	2.30	2.00	1.63	1.41	1.16	1.01	0.89	0.82	0.76	0.71	0.66	0.62	
465	8.2	III	V	3.16	2.26	1.81	1.58	1.29	1.12	0.91	0.78	0.71	0.64	0.60	0.56	0.52	0.50	III
			Q	8.77	6.18	5.02	4.38	3.58	3.09	2.54	2.18	2.00	1.78	1.66	1.55	1.44	1.38	
706	9.8	IV	V	3.56	2.49	2.03	1.77	1.44	1.25	1.02	0.88	0.79	0.72	0.67	0.63	0.58	0.56	IV
			Q	15.0	10.5	8.57	7.44	6.08	5.28	4.30	3.73	3.32	3.04	2.81	2.64	2.42	2.33	
1031	11.3	V	V	3.59	2.53	2.06	1.80	1.47	1.27	1.04	0.90	0.80	0.73	0.68	0.64	0.59	0.57	V
			Q	21.9	15.5	12.6	11.0	8.95	7.74	6.33	5.64	4.91	4.47	4.15	3.91	3.63	3.46	
1350	12.8	VI	V	3.90	2.75	2.25	1.95	1.61	1.38	1.13	0.98	0.87	0.80	0.74	0.70	0.65	0.62	VI
			Q	31.2	22.0	17.9	15.6	12.8	11.0	9.00	7.80	6.97	6.35	5.90	5.56	5.19	4.92	

A is area of flow in sq. cm ; R is hydraulic radius in cm ; V is velocity in metres per sec. ; Q is discharge in cu. metres per min. Drains Types I to IV have smooth cement concrete surface, and Types V and VI Brickwork cement plastered 6 mm thick 1 : 1, smooth finished surface. The cement concrete may be 1 : 2 : 4 or 1 : 2½ : 5.

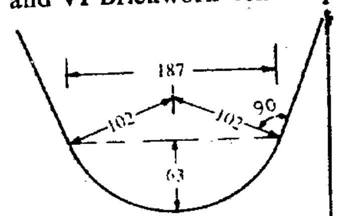
Table of Quantities for Type Drains in cu. metres per 100 metres length of Drain

Type of Drain	I	II	III	IV
1. Excavation	3.72	14.6	19.9	25.3
2. Lime concrete or lean cement concrete (1 : 6 : 12) in foundations	1.30	5.58	7.63	9.11
3. Cement concrete (1 : 2 : 4) finished smooth	2.98	5.02	5.77	6.60

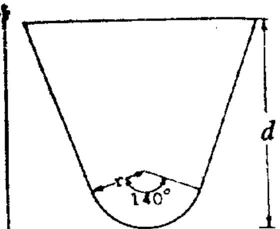




The above two figures show useful cross-sections.



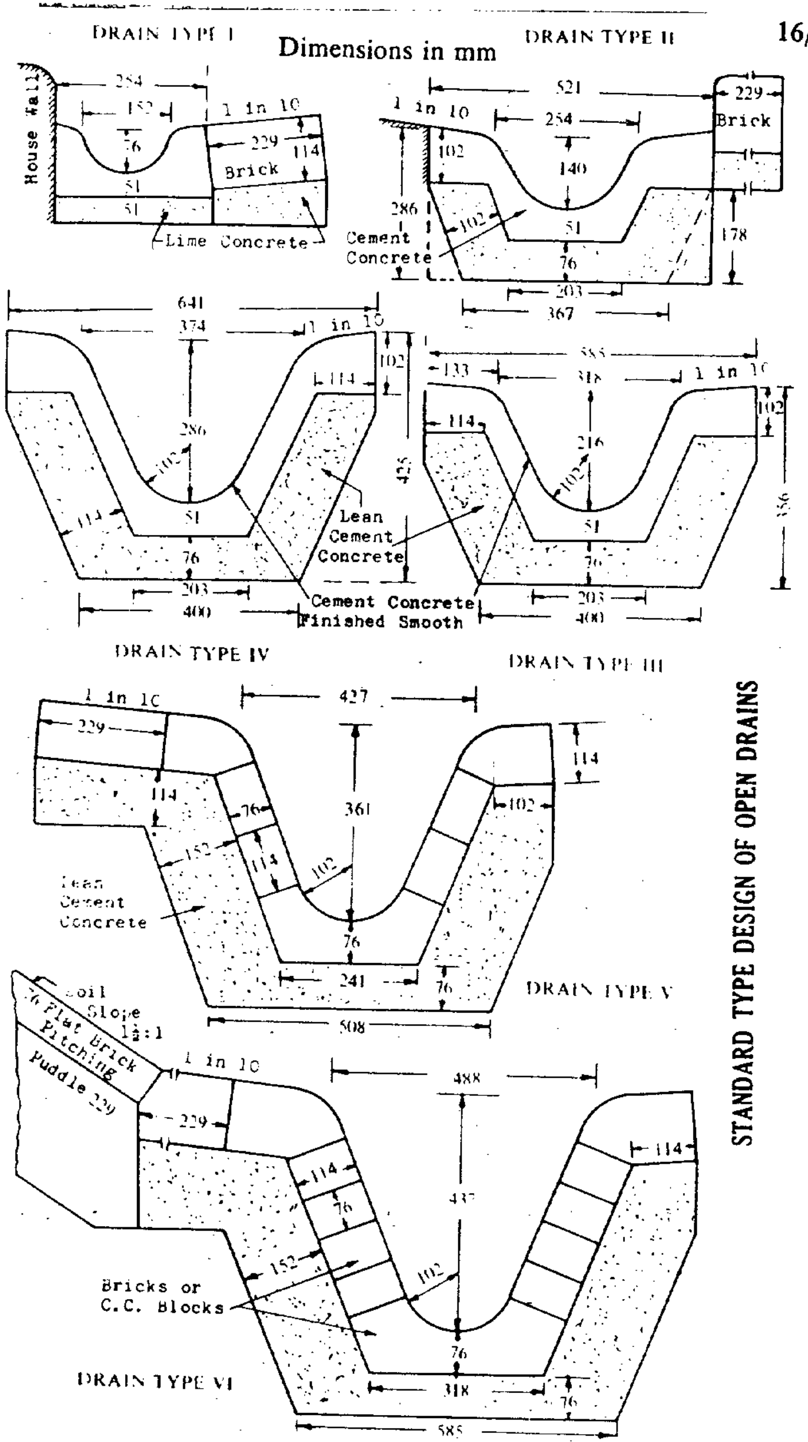
Bottom for Type Drains



Values in centimetres of Hydraulic Radius (R) and Area (A)—in sq. centimetres, of Peg top Drains with 140° central angle.

Proportion of depth *d* on invert to Radius *r* of base		Radius *r* of segmental portion in millimetres 76 mm	89 mm	102 mm	114 mm	127 mm	153 mm
$d=2r$	R	5.88	6.86	7.80	8.81	9.81	1177
	A	209	322	421	531	660	948
$d=3r$	R	7.50	8.72	9.97	11.22	12.44	14.97
	A	423	576	752	957	1273	1700
$d=4r$	R	8.93	10.42	11.95	13.41	14.94	17.92
	A	652	892	1254	1468	1821	2118

RCC Drains. Where it is proposed to make drains of RCC the same should be not less than 50 mm thick and should be reinforced with 3 longitudinal bars 6 mm diameter and 2 cross bars of the same size in 60 cm lengths of the drains. The drains should be cast in lengths not



STANDARD TYPE DESIGN OF OPEN DRAINS

more than 60 cm and the moulds removed after 48 hours. They shall then be kept well watered for a fortnight and after this, watering may be discontinued and the drains left to cure and harden for at least another fortnight more before laying. The ground shall be cut to the exact shape and slope at which drains are to be laid and the trench well watered and rammed.

Flushing of Drains and Sewers. Where it is not possible to obtain self-cleansing velocities due to flatness of the gradient especially at the top ends of branch sewers which receive very little flow, it is essential that some form of flushing device be incorporated in the system. This can be done by making grooves at intervals of 45 to 60 m in the main drains in which wooden planks are inserted and water allowed to head up and which will rush on with great velocity when the planks are removed. Alternatively, an over-head water tank is built from which connections are made through pipes and flushing hydrants to rush water to the main drains.

Velocities in sewers should not, however, be less than 0.46 to 0.55 m per sec. where flushing arrangements are provided.

Flushing can be very conveniently accomplished by hand through the use of a fire hydrant and hose. Flushing should be done at least once a day.

(See also "Flushing Tanks" under Manholes in the pages following).

FORM OF SEWERS

Circular section. The great majority of sewers are built of circular cross-section. A circular section has the least perimeter for a given area and gives the highest velocity when flowing full or half-full, and is most suitable when the discharge is more or less constant which is in a separate system of sewers (where only foul water flows). But it is not very suitable for a combined system where the dry weather flow is very small and the velocity very low for which an egg-shaped sewer may be more suitable. It is the strongest form for resisting external and internal pressures. Circular sewers permit the use of standard pipes.

A circular section is generally satisfactory up to 1.2 m dia. but in the large sizes it is not always the best shape for structural strength and economy.

Egg-shaped section is more efficient than a circular section for variable flows where there is much difference between the maximum and the normal flow. This shape has slightly higher velocities for low flows over circular sewers of equal capacity and thus there is less tendency for the solids to settle at the bottom.

The discharging capacities of both the egg-shaped and circular sewers, for the same cross-sectional area, when running full, are about the same. Max. discharge occurs when the depth of the flow is 0.94, and min. when it is 1/3rd full.

Egg-shaped sewers are about 50 per cent more expensive than circular barrels of equal capacity, are difficult to construct and somewhat unstable. Brick sewers (circular and egg shaped) are now going into disuse.

Rectangular sections are suitable for storm sewers of moderate or large sizes. Hydraulic qualities are fair when flowing nearly full but the

FLOW OF SEWAGE IN NEW CLEAN ASPHALTED CAST IRON PIPES

Table Giving Discharge in Cubic Metres Per Minute Flowing Full

Hydraulic Gradient or Slope 1 in—	Bore of Pipe in Millimetres: 150	200	250	300	350	400	450	500	600	700	750	850	900	1000	1100	1200
10	3.41	7.55	14.09	23.87	35.46											
15	2.80	6.07	11.43	19.09	28.87											
20	2.39	5.27	9.91	16.32	24.78	35.91										
30	1.93	4.27	7.96	13.18	20.05	29.32										
40	1.64	3.64	6.77	11.37	16.82	25.00	34.10									
50	1.46	3.20	6.05	10.09	15.23	22.28	30.46	41.37								
60	1.34	2.96	5.46	9.18	13.87	20.23	27.73	37.28	61.37							
70	1.23	2.68	5.07	8.46	12.73	18.41	25.46	34.10	56.83							
80	1.15	2.52	4.73	7.84	11.87	17.26	23.87	31.82	53.42							
90	1.09	2.39	4.45	7.32	11.18	16.14	22.28	29.78	50.00	76.24	93.19					
100	1.02	2.25	4.16	6.96	10.59	15.32	21.14	28.41	47.28	70.92	88.65	113.6				
120	0.24	2.05	3.86	6.36	9.77	14.09	19.55	26.14	43.19	64.55	80.46	105.0	132.3	178.2		
150	0.84	1.82	3.41	5.68	8.59	12.59	17.05	23.18	38.64	57.73	70.92	93.9	118.2	158.2	204.6	261.4
200	0.72	1.55	2.91	4.91	7.32	10.86	14.77	19.96	32.50	49.78	60.92	80.01	103.2	136.1	175.0	225.0
250	0.63	1.39	2.61	4.36	6.59	9.77	13.18	17.73	29.55	44.55	54.55	71.37	91.37	122.7	156.4	204.6
300		1.26	2.39	3.93	5.96	6.68	11.82	15.91	26.37	39.78	49.10	63.64	81.83	109.6	140.9	181.8
350		1.02	2.16	3.59	5.46	7.96	10.91	14.55	24.32	36.37	44.55	58.42	74.55	100.9	129.6	163.7
400			2.00	3.32	5.09	7.32	10.18	13.18	22.73	34.10	42.05	54.55	69.55	95.01	120.9	153.6
450			1.90	3.11	4.86	6.91	9.55	12.73	21.37	31.82	39.55	61.82	65.01	89.10	113.7	145.5
500			1.80	2.98	4.55	6.55	9.09	12.27	20.23	30.23	37.50	49.10	61.83	86.37	107.7	136.8
550				2.82	4.32	6.18	8.64	11.50	19.09	28.64	35.46	46.14	59.10	79.56	103.2	130.9
600				2.70	4.14	5.96	8.18	11.14	18.18	27.73	33.64	43.87	56.37	75.92	99.10	126.4
650				2.57	3.93	5.77	7.80	10.55	17.30	26.14	32.28	42.73	54.32	72.74	95.01	120.5
700					3.80	5.50	7.50	10.18	16.64	25.46	30.92	40.46	51.82	69.33	90.47	115.5
800					3.55	5.14	7.00	9.50	15.50	23.64	28.87	37.96	48.64	64.10	84.56	108.2
900						4.84	6.59	9.00	14.64	22.28	27.28	35.00	45.69	60.92	79.10	102.7
1000						4.55	6.21	8.41	13.82	20.91	25.68	33.41	42.96	57.51	74.10	95.92
1200							5.61	7.68	12.57	19.09	23.41	30.23	39.19	52.28	67.28	86.83
1500								6.75	11.27	16.73	20.87	27.05	34.55	46.60	59.78	77.28
2000									9.77	14.55	17.73	23.41	30.00	40.00	51.82	66.60
3000										11.82	14.50	19.00	24.32	31.82	41.82	53.64
4000												15.91	20.68	27.73	35.46	45.46
5000															31.37	40.91

Velocity in Metres per Second: 3.05, 2.43, 1.52, 1.22, 0.91, 0.61

Relative discharging capacities of pipes of different diameters are given in "Water Supply"

Functions of Flow (Velocity and Discharge) in a Circular Pipe Running Partly Full are given at Page 14/24.

(a) Empirical formulae for flow in pipes give varying results. As such, all figures should be taken as more or less approximate.

(b) For different co-efficients of friction (for calculating of velocity) for various kinds of pipes, see Section 14 (Hydraulics).

(c) Where Cement Concrete Pipes are to be used multiply the figures by 1.12.

carrying capacity is suddenly decreased by about 30 per cent when the flow touches the top of the section which is caused by the sudden increase in the wetted perimeter without a corresponding increase in area. Rectangular sections, therefore, should be so designed that the section will never flow quite full. This section is not so economical in material as the circular section. Rectangular shape is easy to construct and design. The invert is dished or sloped to make V shape in the centre to provide for small flows and assist in cleaning.

Horse-shoe section has a semi-circular arc on top with sides either vertical or slightly inclined inward. The invert may be flat, circular or a parabolic arc ; slightly curved bottom is usually adopted. The height of the section is slightly less than an equivalent circular section for the same width. This form is sometimes favoured for concrete sewers as it is more economical to construct though a little inferior to a circular form as regards hydraulic properties.

Other forms of sewers are **Semi-ellipitical U-shape** or, some modifications of the above mentioned cross sections to suit the site and the conditions.

Choice of Materials for Making Sewers

Earthenware or Stoneware Salt Glazed Pipes. Drain pipes are generally of stoneware or fireclay, of varying qualities ; stoneware being much better. Fireclay pipes though less brittle than stoneware pipes are not considered as strong or durable as the latter ; they are also more aborptive. The most vitreous are the best.

They are cheap, easy to lay and very durable if properly laid ; not affected by the sewage acids. Need very careful handling during transit and laying as they are very brittle and easily broken. Earthenware pipes cannot withstand heavy earth or direct susface loads, or settlements due to unstable ground below, and are liable to fracture. Usual limiting size is only up to 450 mm diameter, with max. up to 600 mm, and are manufactured in 60 cm lengths, with spigot and socket ends. All drain pipes should be "salt-glazed".

Tests for Stoneware Pipes. The breaking weight of the stoneware pipes should not be less than 770 kg applied by means of a flat board of hardwood of the same length as the pipe, laid along the top of the pipe throughout its length, exclusive of the socket. The pipe, when subjected to this test should be supported on a similar flat board underneath the socket overhanging, and a layer of felt being laid between the pipe and the boards. Stoneware pipes should not absorb more than 2 per cent of their weight of water when immersed in it for 48 hours.

Cement Concrete Pipes. These pipes are now coming into general use as they have many advantages over other materials. Concrete pipes are either made in moulds or by the *hume* (spun) process, or centrifugal system in which the ingredients are passed into rapidly revolving cylinders which make uniform pipes of great density and strength. Pipes of diameter over 600 mm are reinforced, the reinforcement is either a mesh or solid steel plates and made in varying lengths. (See also under "Water Supply") Concrete pipes should not be used to carry acid effluents nor be laid in those sub-soils in which concrete is liable to be attacked (see Sections 8 and 17). Concrete pipes have various types of joints according to the sizes : spigot-and-socket joints ; ogee or rebated joints with separate collars. Concrete pipes with spigot-and-socket joints present an alternative to glazedware for sewers over 150 dia. These pipes with ogee or rebated joints may be used for surface-water drains in all diameters.

Where concrete sewers are to be cast in situ they can be made in two or three stages. First the invert is poured, then the side walls and finally the arch.

Asbestos-Cement Pipes. Equivalent hydraulically to cement concrete pipes with approximately equal life if laid carefully but not of equal strength for handling. Not very popular except for rainwater pipes. Usually manufactured in 1.8 m and 3 m lengths with all fittings as for cast iron pipes.

Cast Iron Pipes are used under heavy pressures of earthfills, in unstable grounds with possibilities of settlements, or under buildings, or where it has to be carried on piers or trestles above ground, and for high velocities in steep slopes and such like places. Cast iron pipes are also lined with cement mortar. (More details will be found in the following pages and also in 'Water Supply.')

Inverted Siphons are constructed to carry the sewage through depressions under obstacles. A siphon usually consists of two or more pipes in parallel. Small chambers are made at the inlet and outlet ends to facilitate cleaning and repairs. Sufficient head should be allowed in the design to supply the head losses in the siphon. A minimum velocity of 0.91 m per sec. should be provided to minimize the possibility of clogging. Siphons are usually source of trouble unless designed with utmost care.

Setting out Sewer Lines and Excavations (See also under "Water Supply")

Levels are taken along the centre line of the proposed sewer, say every 15 m, or closer if the surface is irregular. A longitudinal cross-section is prepared with an assumed datum line (which may be the invert level of the lowest point of the sewer), showing the proposed sewer set to suitable gradients, giving clearly the invert and surface levels at all manholes and points where the gradient changes.

The centre line of the trench is first staked out on the ground driving in 50 mm pegs about 30 m or less, apart. The width of the trench to be excavated is marked on both sides of this centre line and excavation lines cut out with a spade. It is important to excavate the trench to the correct width and depth at all points, any extra depths cut out at the bottom of the pipe line should be made good with concrete not weaker than 1 : 10. It is preferable to excavate to about 75 mm of the finished formation level, this final 75 mm being trimmed and removed as a separate operation immediately prior to the laying of the pipes or their foundations.

In obtaining the formation of the bottom of the trenches and the levels of the inverts of the pipes, the usual method of sight rails and boning rods is employed. The practice of "transferring" levels by means of a straight-edge and spirit level should be discouraged. Sight-rails (also called sighting rails or batter-boards) are wooden boards of size about 150 mm × 50 mm and of sufficient length to extend over more than the full width of the trench and are nailed across the line of the trench at ends by upright posts at 8 m intervals, so that they remain truly horizontal. These posts are set sufficiently wide apart so that excavation can be carried out between them without their being disturbed ; may be about 0.6 m more than half the width of the trench on either side from the centre line, and can be fixed in the ground on either side of the trench or planted centrally in stoneware drain pipes resting firmly on sockets. Earth is filled in the pipes and well rammed to securely fix the posts. The top of the sight rail is fixed about 1.2 m above the ground, which is a convenient distance for sighting. A level has to be used for setting. The centre line of the sewer is marked on the sight-rail by nailing an upright cleat on it.

The line at the trench bottom can be marked or checked by a plumb bob hanged from a cord extended from cleat to cleat on the sight-rails. The line sighted along the top edge of the rails represents the true fall of the sewer, and this gradient is transferred below the ground level by means of a boning rod (or traveller) of a fixed length which is boned in between the rails for each pipe with the help of the cord. The boning rod consists of a long wooden piece of size 75 mm × 40 mm cut to the length required (which is equal to the distance between the invert of the pipe and the top edge of the sight rail cross-piece). One boning rod is provided for each length of excavation. A cross-piece of size 450 mm × 90 mm is fixed with nails at the top of this rod so as to form shape like a Tee-square. At the foot of this rod an iron shoe is fixed to rest on the inverts of the pipes. It is important that boning rod is cut to the exact length required. Sometimes boning rods are made of adjustable lengths.

Drain pipes are always laid with the socket at the higher end, and consequently it is necessary to begin at the lower end of a drain and to work upwards probably over the point of connection to an existing sewer. A sight rail will be required over this point. More sight rails will be required at manholes, change of gradient and at intermediate positions if the distance for sighting is too far, which may not be more than 15 m apart. The excavation should be boned in at least once in every 2 m the foot of the boning rod being set on a block of wood of the exact thickness of the material of the pipe. Each pipe should be separately and accurately boned between sight rails.

When excavating, the sides of the trench should be supported by timbering where the depth exceeds 1.2 m, unless the soil is very stiff. For ordinary cases with reasonably firm earth, vertical poling boards of size say, 230 mm × 50 mm can be fixed and which are supported by 230 mm × 75 mm waling boards placed horizonally and at right angles, above the poling boards where the soil is loose. The distance apart of poling boards and the waling boards depends upon the looseness of the soil to be supported. These boards are securely held in place by 150 mm × 150 mm horizontal struts fixed across the trench about 2 m apart. Waling boards can be done away with, and struts fixed directly on poling boards where the soil will stand. The bottom set of poling boards should be driven at least

230 mm below the bed of the trench level. Steel (or wooden) interlocking sheet piles are used where water or running sand is encountered.

The trench should be sufficiently wide to allow space for timbering where required and also for the work-men to work and walk sadle-wise along the pipe line. A space of 150 mm to 230 mm on either side of the body of the sewer is considered sufficient. Extra excavation is required under the sockets to allow hands to pass for making joints. Minimum width of a pipe trench should be 530 mm even for the smallest pipe, to facilitate working.

Where rock occurs in a trench a cushion of sand 25 mm thick should be provided on which to lay the pipes. In some grounds where the finished surface of the formation becomes soft after levelling, a firm bottom may be obtained by spreading and compacting a 75 mm layer of gravel or broken stone over the trench bottom, which should be further excavated to receive this.

Bedding Methods. In order to make a strong foundation, especially for earthenware pipes, the bottom must be shaped to fit the pipe barrel and hollowed out to receive the socket and make joints, so that the barrels of the pipes rest throughout their entire length on the solid ground and the bearing of a pipe is eventually taken by the body of the pipes and not by the sockets. This method can be adopted in firm ground but in a soft soil the trench bottom must necessarily be flat and a cradle (concrete filled around the pipe) of lean concrete (1 : 8 or 1 : 6) is necessary. In the case of large sewers flat concrete bottom has to be provided under a circular pipe so as to distribute the weight (weights of the sewer masonry, water, earth above, etc.) over a larger area. The bedding (concrete) should extend at least 150 mm beyond and at both sides of the projection on the barrel of the pipe. The thickness of the concrete below the pipe should not be less than 100 mm for pipes under 150 mm in dia. and 150 mm for pipes 150 mm and over in diameter.

Ordinary Bedding. Earth foundation shaped to fit the lower part of the pipe exterior with reasonable closeness for a depth of at least 1/10th of the external diameter of the pipe.

Better Class Beddings :

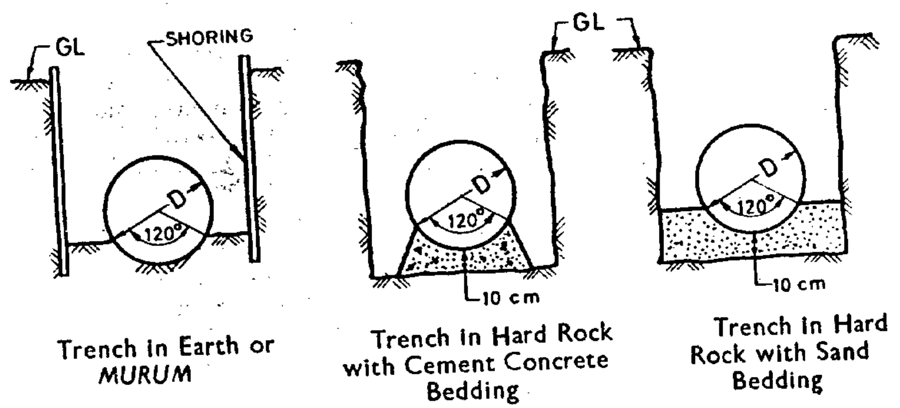
(*i*) The pipe is bedded in an earth foundation shaped to fit the lower part of the pipe exterior for a width of at least half of the external diameter of the pipe and the remainder of the pipe is surrounded to a height of at least 150 mm above its top by granular materials.

(*ii*) Selected granular material is tamped under and around the pipe to a height equal to 3/4th of the diameter.

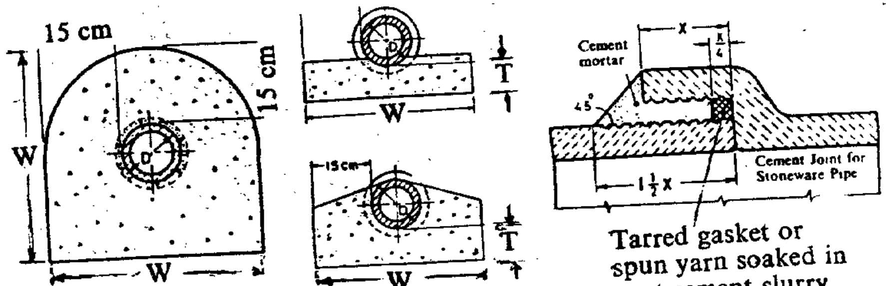
(*iii*) Pipe is bedded in a 150 mm cradle of lean concrete.

(*iv*) Ground shaped to the bottom quadrant of the pipe and concrete filled in on the sides.

First Class Bedding. The pipe is carefully bedded on fine granular materials in an earth foundation carefully shaped to fit the lower part of the pipe exterior for a width of at least 7/10th of the external diameter of



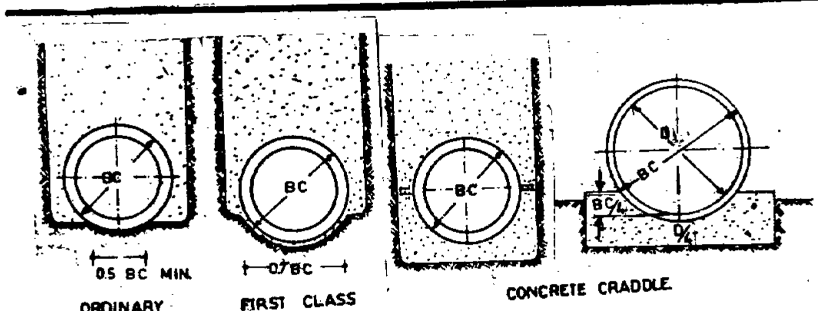
TRENCHING FOR STEEL PIPES IS : 5822



$T = 10$ cm for pipes under 150 mm dia.
15 cm for pipes of 150 mm dia. and over.

$W = D + 30$ cm

BEDDING & ENCASING STONEWARE PIPES



BEDDING OF PIPES

the pipe and the remainder of the pipe is entirely surrounded with concrete and up to a height of at least 300 mm above its top.

Pipes are generally bedded throughout the length between the joint holes. A pipe resting on flat ground develops only 80 per cent of its strength and a pipe laid in concrete develops strength up to 200 per cent.

Laying and Jointing Pipes. Each pipe should be carefully examined for soundness before laying, it should be rung with a light hammer and those that do not ring true and clear, rejected.

Normal requirements as regards concrete protection of stoneware and concrete sewer pipes are laid down by the Ministry of Health (England) as follows :

(*i*) Pipes and tubes in heading or with 6 m or more of cover in trenches to be surrounded with at least 150 mm of concrete.

(*ii*) Subject to (*i*) all pipes and tubes with over 4.3 m of cover or 450 mm or more diameter to be bedded on and haunched with at least 150 mm of concrete to at least the horizontal diameter of the pipe or tube. Any splaying of the concrete to be above that level.

(*iii*) Subject to (*iv*) all pipes and tubes under 450 mm diameter and with less than 4.3 m of cover may be laid without concrete, if the joints are of socket or collar type.

(*iv*) All pipes and tubes with less than 1.5 m of cover under roads, or 0.91 m not under roads, to be surrounded with at least 150 mm of concrete.

Jointing Stoneware Pipes. The spigot of each pipe should be placed in the socket of the one previously laid. The spigot ends should be in the direction of the flow (spigot at the lower end and the socket at the higher end). Socket ends are useful for adjustment of small angles in the alignment during laying. The pipes should not be jointed until the earth has been partly refilled over the portion of the pipe between the joint holes. Before laying the second pipe, the socket of the first pipe laid is thinly painted all round on the inside with cement mortar (1 cement to 2 clean sharp sand). A ring of rope yarn (closely twisted hemp or jute, called "gasket") dipped in neat cement grout (thick paste) or tar or bitumen, is inserted in the socket of the pipe and driven home with a wooden caulking tool and wooden mallet. The rope should fully encircle the spigot with a slight overlap and should not occupy more than one-fourth of the total depth of the socket. Where the spigot end of the pipe is made for receiving the gasket (the exterior of the spigot end and interior of the socket are provided with grooves and left unglazed) it should be wrapped round with two or three turns of tarred spun yarn, as near the end as possible, before inserting into the socket. This helps to keep an even space all round the spigot in the socket.

The joint is then completely filled with cement mortar (1 : 1), which should have very little water, and levelled to form a splayed fillet at an angle of 45 degrees with the outside pipe. Special care skould be taken that any excess of cement mortar, etc., left inside the pipe joint is neatly cleaned off immediately each joint is made. A semi-circular wooden scraper or a rubber disc can be made to which a long handle is fixed.

The amount of cement needed is roughly 300 grams and spun yarn or hemp 8 grams for each 25 mm diameter of the pipe. One man can make about 16 joints in a 200 mm pipe in one day.

The refilling of the trenches or concreting of haunching or surround, where specified, should not be undertaken until the joints of the pipes are thoroughly set and have been inspected, tested and approved. Refilling should be done in 230 mm layers thoroughly rammed. Excess of watering should be avoided. The finest material should be selected for the first 30 cm of the filling which should be free from stones or any other hard material. Large clods of earth should never be thrown in as the shock may injure the drain.

In jointing stoneware pipes certain defects are liable to occur : (*i*) The pipes may not be concentric. This should be guarded against by wedging up the spigot end of the pipe that is being laid by a chip of wood so as to bring the pipes concentric.

(*ii*) The socket may not be properly filled with cement especially on the underside, where it is not easy to get at it. The lower half of the socket should be first spread evenly with cement, and the spigot or the fresh pipe should then be introduced and pressed firmly home against the shoulder of the pipe in position, care being taken that it is kept concentric with it. The rest of the socket should then be filled with cement, which should all be pressed well home with a hardwood rammer, curved to work between the spigot and the inside of the socket, cement being added till the joint is full. In order to detect any defect, the underside of every joint should be inspected with a looking-glass, and should be felt and pressed with the fingers while green, to see if it is really full.

Cement joints are rigid and even a slight settlement of pipes can cause cracks and hence leakage. For this, joints are made with bitumastic filling instead of cement. The same type of gasket is used as specified for stoneware pipes. The gasket should be in one piece of suitable diameter, not less than 20 mm. A gasket of closely twisted hemp or *oakum* is used. (Oakum is the long, loose fibrous material obtain by untwisting and pulling old ropes.) A suitable runner should be placed arouud the pipe and against the face of the bell to close the socket opening before pouring in the hot bituminous compound or asphalt. Special patent joints are also available.

Jointing Concrete Pipes. Concrete spigot and socket pipes are laid and jointed as described above for glazed stoneware spigot and socket pipes, with yarn or rubber gasket and cement. Asbestos cement pipes are generally jointed by a collar and two rubber rings. These pipes should be bedded on concrete as joints require such suport. Pipes of large diameter should be encased in concrete. (Illustration given on the pages following)

Large size concrete sewers have "ogee" joints in which the pipe has mortise at one end and a tenon to suit at the other end, and are jointed with cement or asphalt. A concrete collar sufficiently wide to cover and overlap the joint is fixed on it. A combination of rigid and semi-flexible joints is sometimes used in hydraulic concrete pipe lines. The line is made up of rigid joints with semi-flexible joints at about 15 m intervals. Concrete

pipes having spigot-and-socket joints should be used where practicable in preference to those having ogee joints, as the latter are more difficult to make water-tight.

Cast-iron Pipes. Where for any reason cast iron pipes are to be laid on concrete, they should be laid on precast concrete blocks, two to each pipe.

Cast iron pipes with spigot-and-socket joints are jointed either with a 'fibrous-lead' (lead wool) joint or with sulpher-sand composition as explained under "Water Supply." The lead should be run at a single pouring. After each joint is made the inside of the pipe should be examined to ascertain that no lead has penetrated into the interior of the pipe. Fibrous-lead is particularly suitable for use under wet conditions and in headings where the use of molten lead would be a source of danger to the men working.

Junction Pipes should be inserted at intervals as required for present or future connections during the construction of the sewers. Any branch eyes which are not immediately connected up should be closed. The position of each such junction should be recorded on the "completion" plan of the work.

Sewer Crossings. If possible, keep all sewers 1.2 m away from the external walls of a building and do not pass any sewer or drain under a building, if must, surround the (glazed ware) pipe with 150 mm of concrete or lay in cast iron pipes and provide excesses at each end immediately outside the building. No branches should be connected to the portion of the drain under the building. Where pipes are passed on a bridge where vibrations occur, cast iron pipes with special couplings, rather than standard lead joints, should be used to avoid leakage. Where shallow depressions are to be crossed, the pipes can be supported on piers or bents built just behind the pipe sockets. Such piers should not be less than 300 mm in length (parallel to the pipe axis) ; cast-iron pipes with lead joints may be used.

Branch Connections. No bends whatever should be permitted in sewers, except at manholes. Where a change of direction cannot be avoided and this exceeds 45 deg., access should normally be provided at the bends or junctions. The use of quarter bends (90 deg.) should be avoided, except at the foot of vent pipes. All junctions should be oblique, and the contained angle be not more than 45 deg.

A branch should be connected with a main sewer at an angle making a Y junction so that the entering sewage will follow the flow in the main pipe and at as easy an angle as possible (and not at an acute angle). As far as possible the old pipe should be broken out and a new Y junction put in. If the new pipe must be jointed with the old sewer, an oblique saddle junction can be made.

Sewers should always be joined soffit to soffit and not jointed invert to invert. When a branch sewer joins an egg-shaped sewer, the connection should be made at a point at least 2/3rd up from the invert.

Junctions made at 45 deg. are called Y junctions, and junctions formed at a smaller angle than 45 deg. are called V junctions and are used to connect a very oblique branch drain with a principal drain. When a junction is fixed on the line of a drain, care should be taken to give it a tilt to the gradient of the incoming branch drain for which it is provided, and the oblique arm should be packed up with fine concrete to keep it in position.

House connections. In congested areas, it is desirable to shorten the spacing of manholes and to introduce the house-service through the manhole instead of disturbing the sewer. Sometimes a small duplicate sewer can be arranged.

Making of Connections

(*i*) *Connection to glazed-ware pipe or concrete pipe sewers.* For sewers of less than 230 mm diameter where it is not possible to fix a saddle, three pipes should be carefully broken out completely and replaced by two plain socketed pipes and an oblique junction pipe. Or alternatively, two pipes may be broken out and replaced by an oblique junction pipe and one double-spigot pipe (or one from which the collar has been cut off), the joint between this and the existing pipe line being made with a loose collar.

For sewers of 230 mm diameter and over, a saddle should be used in preference to the above method. A hole is carefully made in the top half of the pipe and trimmed so that the saddle fits, and at least half the breadth of the saddle shoulder bears on the sewer pipe all round. The saddle should be properly secured in position so that there is no movement, and jointed all round with cement mortar. After the mortar has set, the saddle should be completely surrounded with 150 mm of concrete.

Breaking into the sewer should be effected by the cautious enlargement of a small hole, and no connection should constitute a projection into the sewer.

(*ii*) *Connection to cast-iron pipe sewers.* For sewers of 230 mm dia. and over, a hole may be cut with a blow-pipe in the top of the pipe and a saddle fitted as described before. Or alternatively, a sufficient length of the existing pipe should be cut out and an oblique junction and a loose collar inserted and jointed in lead. Where a pipe is inserted into a large sewer, the joint should, wherever possible, be made good from inside the sewer with cement mortar to form a flush joint.

The number of houses that may be connected to a 100 mm private sewer may be four; but as many as twenty have been so connected satisfactorily. For more than twenty houses the pipe should generally be increased to 150 mm diameter. (Illustration given on the pages following)

3. MANHOLES

Manholes *or Inspection Chambers* are openings through the street surface to the sewer to provide access for inspection and cleaning. Provision of manholes is essential in all sewerage lines and are usually provided at all junctions, change of direction or alignment, change of gradient and size of sewer. In the case of large sewers, where a man can enter, it is not essential to have a manhole at every change in alignment up to 30 deg. curve; manholes on curves should generally be sited at tangent points. Manhole

chambers may be omitted in very large sewers where a man can stand conveniently. In any sewerage system the cost of manholes is 25 to 30 per cent of the total cost, excluding the cost of treatmens works.

Spacing of Manholes. Different authorities recommend very different spacings.

According to IS : 4111

Sewer dia. in mm	up to 300	301 to 500	501 to 900
Spacing in metres	45	75	90

British practice :

Sewer dia. in mm	under 600	900	1200	over 1500
Spacing in metres	91	136	182	270 to 360

Russian practice :

Sewer dia. in mm	150 to 600	over 600
Spacing in metres	not more than 50	75 to 150

On small sewers which cannot be entered for cleaning or inspection, a manhole should be built at the head of all sewers and branches and at about 90 to 150 metres on straight runs for sewers of 600 to 1200 mm diameters. A *rodding eye* may serve the purpose of a manhole at the head of a shallow drain. A spacing of 180 to 240 metres may be allowed on straight runs for sewers of 1200 to 1800 mm diameters and which may be increased to 300 to 360 metres for sewers of over 1800 mm diameter. On economic grounds, a *lamphole* may be substituted between manholes in lengths which have frequent changes of direction. Where silt and grit loads are usually heavy, catch-pits with by-passes should be provided at every 450 to 600 metres, for facilitating maintenance.

Size of Manhole—(Min: internal dimensions of chambers having not more than two branch channels on either side)

(*a*) 90 cm x 80 cm for house drainage constructed within the compound of the building or nearabout for depths of 1 m or less.

(*b*) 1.2 m x 90 cm for main drainage works for depths between 1 m 2.5 m.

(*c*) 1.4 m x 90 cm or circular with min: dia. of 1.4 m for main drainage works where depth is 2.5 m or more. For big depths the chamber opening is reduced towards the top by corbelling inwards on three sides or it is divided into two parts, the lower part is covered with a roof slab or an arch and the upper portion made into an access shaft up to the ground level.

The min: internal dimension of the access shaft in a deep manhole shall be not less than 70 cm square where step irons are used, and 80 cm x 70 cm in shafts with vertical ladders. Circular shafts shall be not less than 70 cm internal diameter. The lower portion of the chamber shall have a height of at least 1.85 m so that a man can stand inside it for cleaning. The access shaft is made on one side and not in the centre of the manhole chamber.

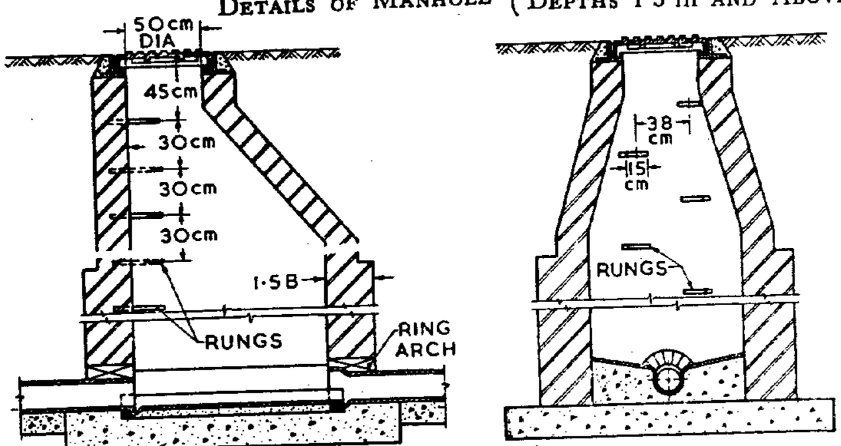
The width of the chamber should be such as to allow at least 150 mm benching on each side of the invert, one benching of at least 350 mm wide should be provided for a man to stand on.

The sizes recommended are the minimum and should be suitably increased according to the directions and the number of drains meeting by 30 cm for each additional branch, the longer dimension being in the direction of the main sewer. The minimum size of chamber in which a man can work efficiently is 1.22 m on the line of the sewer and 0.75 m across, and for a circular chamber not less than 1.22 m dia. (min: 1.07 m). The width of manholes shall be increased more than 90 cm on bends or junctions or pipes with diameter greater than 450 mm.

No drain from house fittings, *e.g.*, gully traps or soil pipe, etc. to manhole shall normally exceed a length of 6 metre unless it is unavoidable.

Manhole Covers are made of cast iron with double seal frames. The top surface of the cover is made rough or chequered. The size of manhole covers shall be such that there shall be clear opening of at least 50 cm (prefer 55 cm) for manholes exceeding 0.9 m depth. Circular form is the strongest in proportion to the weight of metal. Detailed drawing of manhole covers are given in IS : 1726. Ventilating manhole covers should not be used for domestic drains. The cover frame should be bedded on three courses of brickwork, corbelled over to form an opening. A shallow manhole should be covered by a stone or concrete slab.

Details of Manhole (Depths 1·5 m and Above)



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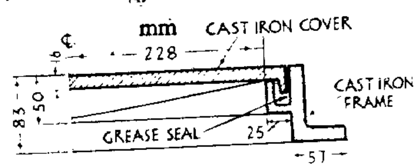
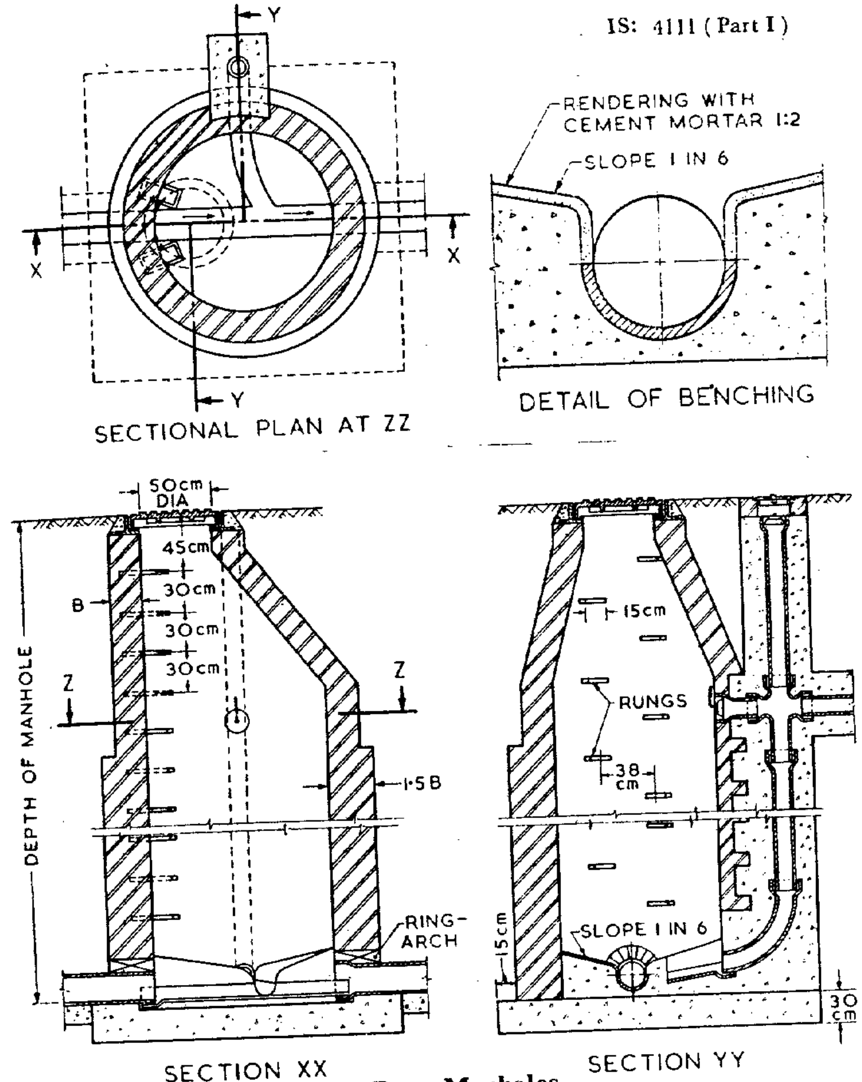
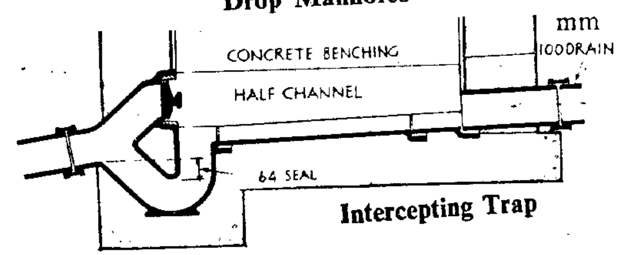


(Illustrations given on the pages following)

IS: 4111 (Part I)



Drop Manholes



Intercepting Trap

Manhole covers are usually of three grades: Heavy Duty (HD), Medium Duty (MD), and Light Duty (LD). The approximate weights of the various types of manhole covers and frames are as follows :—

Description of C.I. manhole cover Type	Weight of cover	Weight of frame
HD 560 mm dia. (clear opening)	108 kg	100 kg
MD 500 mm dia. „	58 kg	58 kg
LD 455 x 610 mm (single seal)	23 kg	15 kg

Covers and frames should be coated with a black bituminous composition.

When manholes are constructed on footpaths, these shall be provided with cover of medium duty casting, and when built within the width of the road under vehicular traffic, these shall be provided with cover of heavy duty casting.

Foot Rests (also called treads or rungs). All manholes deeper than 0.8 m have to be provided with foot-rests inside. These are made of 20 mm mild steel square or round bars, tar dipped. The foot-steps are either built into the brickwork every fourth course (about 30 to 38 cm vertical intervals) or embedded 20 cm deep with 20 x 20 x 10 cm cement concrete blocks. The foot-rests are staggered laterally in two vertical runs which may be 38 cm apart horizontally, and project 10 cm beyond the surface of the wall. The top foot-rest should be 45 cm below the manhole cover and the lowest not more than 30 cm above the benching. The foot-rests will have a minimum length of 30 cm (20 cm embedded inside the wall and 10 cm projecting beyond the wall)—total length of a bar will be 75 cm. In small manholes, steps may be built across the corners. (Where the bars have to be embedded in a concrete block, the portion in concrete should be painted with thick cement slurry and not tar.)

For depths over 4.5 m, a galvanized wrought iron ladder is preferable with stringers not less than 70 mm by 13 mm. Ladders should not be less than 30 cm between the stringers and the rungs at 25 cm centres and should be provided with brackets built into the brickwork. The minimum distance of the ladder from the wall should be 12 cm at the top (bent to a radius of 15 cm), which should be increased to 15 cm or more at the bottom.

All manholes on sewers of 1 m diameter and over should be provided with galvanized wrought iron *safety chains* on the down stream side, which should have close-links, 6 or 10 mm. Galvanized pipe handrails of 40 mm bore should be provided on the edges of all benchings from which a man might possibly fall into the sewer.

Drop Connections (or Drop Manholes). When a branch sewer (incoming drain) has to be connected with a main sewer in manhole and the invert level of branch line is less than 60 cm above the invert of the manhole, a drop connection may be provided within the manhole by constructing a ramp in the benching of the manhole. If the difference in

the two levels is more than 60 cm, connection can be made by providing a vertical drop pipe from the higher sewer to the lower one. This pipe may be either outside the manhole walls and encased in concrete or supported on brackets inside the manhole which should be made of a bigger size. If the drop pipe is outside the manhole wall, a continuation of the sewer should be built through the manhole wall to form a rodding and inspection eye, which should be provided with a half blank flange. If the drop pipe is inside the manhole, it should be of cast iron ; a water cushion of 150 mm depth should be provided. The drop pipe should terminate at its lower end with a plain or duck-foot bend turned so as to discharge its flow at 45 deg. or less to the direction of the flow in the main sewer, and the pipe, unless of cast iron, should be surrounded with 150 mm of concrete.

Scraper (Service) Type Manholes. All sewers above 450 mm diameter should have one manhole at intervals of 110 to 120 m of scraper type. This manhole should have a clear opening of 120 x 90 cm at the top to facilitate lowering of buckets.

Channels should be semi-circular in the bottom half and of diameter equal to the sewer. Above the horizontal diameter, the sides should be extended vertically to the same level as the crown of the outgoing pipe and the top edge suitably rounded off. The branch channels should also be similarly constructed with respect to the benching but at their junction with the main channel an appropriate fall suitably rounded off in the direction of flow in the main channel should be given. All channels in the manhole should be given a minimum slope of 1 in 30.

Brickwork. Walls of the manholes are made 20 cm (or one brick) thick up to 1.5 m depth and 30 cm (or one and a half brick) for depths greater than 1.5 m up to 4.5 m, in cement mortar. Actual thickness can be designed for lateral earth pressure and water pressure as described ir Section 7. In the case of manholes of circular type the excess shaft is corbelled inwardly on three sides at the top to reduce its size to that of the cover frame to be fitted. Walls of the manholes are cement plastered inside and all corners rounded off. Where a saturated soil is met with, there the external surfaces of the walls should also be plastered up to 30 cm above the highest sub-soil water level. Excavations for manholes and othe ancillary structures should have 30 to 60 cm clearance on all sides.

Bed Concrete. A manhole should be built on a bed of cemer concrete 1 : 4 : 8 graded stone aggregate of 20 mm size. The thickness c the concrete may be 15 cm up to 1 m depth, 20 cm for 1 to 2 m depth an 30 cm for greater depths.

Where sewers are to be laid in high sub-soil water condition manholes may be built of RCC 1 : 1½ : 3, and preferably of circular typ Walls may be constructed of brick masonry above the sub-soil water level.

If a manhole is in agricultural land not likely to be developed, tl chamber walls (in shallow manholes) or shaft walls (in deep manhole: should, if possible, be built up to about 60 cm above natural groui surface and mounded on all sides with earth.

Except in manholes not liable to super-imposed loads, all pir exceeding 150 mm diameter passing through the manhole walls should ha an arch formed over them in order to relieve the pipe of the weight of the wall above.

Benching. The bottom of the manhole should be "benched" to have a fall towards the invert of about 1 in 6. The benching should be at least as high as the soffit of the outgoing sewer, and should be floated to a smooth surface with cement plaster 1 : 2. In the case of branch drains the benchings should be so shaped round the channel branches as to guide the flow of sewage in the desired direction. Provide at the bottom either stone-ware glazed semi-circular pipes, or channels of cement concrete plastered with 25 mm cement-sand mortar 1 : 2, and finished smooth. The channel depth should be equal to the sewer diameter and the ends of the channel should fit the sewer ends accurately.

According to CPWD specifications, the depth of channels and benching shall be as given in the table :—

Size of Drain in mm	Top of channel at the centre in cm above bed concrete	Depth of benching at side walls in cm above bed concrete
100	15	20
150	20	30
200	25	35
250	30	40
300	35	45
350	40	50
400	45	55
450	50	60

Branch sewers should join the main sewer in curves of sufficient radii in a manhole to guide the flow and to avoid formation of eddies and disturbances of flows. Or, alternatively, Y-branch connections should be made as described earlier. The curved portions of the branch sewers should be wherever possible within the manhole walls, and for economy, the manhole chamber may be built of a shape other than rectangular.

Manholes at which a number of sewers meet are called *Junction manholes*. The manholes should be so placed as to prevent any back-flow of the sewage. Sewers should not meet at a sharp angle so as not to decrease the velocity of flow or cause disturbance of flows and settlement of suspended solids and formation of obstruction. Y-branch connections should be made. The curved portions of the branch sewers should be, whenever possible, within the manhole walls, and for economy, the manhole chamber may be built of a shape other than rectangular.

At a junction, the tops of all the sewers should be at the same level, so that the pipes of smaller diameter are not flooded when the bigger pipes are running full. The actual gradient of the smaller sewers may be steepened sufficiently to reduce the difference of the invert level at the point of junction to a convenient slope. The invert of the smaller sewer at its junction with the main should be at least 2/3rd the diameter of the main above the invert of the main.

Lamp-holes. When the length between two manholes on a straight run of sewer is more than usually allowed and there is insufficient space to construct a regular manhole, or at places of slight change of direction, a vertical pipe or shaft of about 225 mm diameter from ground level is connected to the sewer by a T-bend and covered with an iron grating. A lamp can be lowered down this pipe and the sewer inspected from the manholes from either side. Lamp-holes are not now in much use.

Ventilation of Street Sewers. Ventilation of a sewerage system is essential to remove dangerous, explosive and foul gases and to provide means for the escape of air and allow free flow as the sewer fills, and also to prevent breaking of the seals of traps of the sanitary fittings of buildings.

Ventilating shafts are provided at the starting point of the main sewer and at such points where the flow of the sewerage is disturbed, *i.e.*, at falls, syphons etc., and these should be at least 60 metres away from residential buildings.

According to C.P.W.D. specifications the distance between vent shaft and the diameter of vent shaft shall be as follows :

Dia. of main sewer	up to 900 mm	900 to 1800 mm	above 1800 mm
Internal dia. of shaft at top	140 mm	250 mm	300 mm
Distance between shafts	120 mm	240 mm	300 mm

A ventilating shaft for street sewers is made of RCC with overall length of 9.10 metres and 140 mm diameter top opening, tapering to 450 mm at the bottom (inside dimensions). The bottom it bedded in concrete of 1.50 metres depth and 90 cm width. The shaft is provided with a cowl or fitted with a wire guard at the top. Somethimes small vent shafts of cast iron of 100 mm diameter are provided about 30 metres apart.

There are conflicting views on the need for ventilation shafts in the sewer system. Some engineers prefer to dispense with the intercepting traps which disconnect house connections from street sewers and also the ventilating shafts and depend upon the ventilation of the sewers through soil and ventilation pipes of private buildings. Escape of air can also be effected on the surface sewers by providing ventilating manhole covers. It is considered that sewer air is free from bacteria and not dangerous to persons breathing it ; fresh sewage does not produce a bad smell.

In America connections from buildings are not generally trapped before joining the sewer except in the cases of certain factories, hotel, etc., for which grease traps are provided.

Flushing Tanks. (See also under "Flushing of Drains and Sewers" at page 16) A flushing tank is commonly a brick chamber in which a syphon is fixed and water stored and discharged automatically at fixed intervals for flushing sewers. A water tap fills the tank at regular intervals and can be adjusted to give the number of flushings desired in a day.

The quantity of flushing water should be sufficient to fill the sewer at least half-bore over the whole length of the sewer to be flushed. Where only small tanks are proposed to be installed, the capacity of these tanks should not be less than 1/10th of the cubic capacity of the sewer length to be flushed.

According to IS : 4111, the approximate quantity of flushing water per flush over a length of 75 to 90 meters of sewer is as follows, the period of flushing being once in 24 hours :

Dia. of pipe — mm	250	350	400	450
Quantity of water —Litres	1400 to 1700	1700 to 2700	2700 to 3600	3600 to 4500

For the various capacities of water tanks, the following sizes of cast iron automatic syphon pipes can be used :

Capacity of tank —litres	450	675	900
Dia. of syphon pipe —mm	65	80	100

An Adam's syphon is an automatic syphon made of cast iron with trapped outlet for flushing.

A manhole-cum flushing tank can be built at the head of a sewer where required.

Maintenance of Street Sewers. It is essential that street sewers are periodically inspected, cleaned and copiously flushed from time to time without waiting for the occasion when they are actually blocked. The following devices can be used for cleaning :—

(*i*) Flat steel bars of size 25 to 40 mm x 3 mm ;

(*ii*) Bamboos (usually halved) or cane rods ;

(*iii*) 0.91 to 1.2 m long rods of hard cane, bamboo, or some flexible wood, about 40 mm diameter, each fitted with a steel eye at one end and a hook at the other. The hook is made in such a manner which can be joined or opened only when the rods are at right angles to each other. Instead of hook and eye, brass or wrought iron male and female ends are fitted and the rods can be attached to one another and pushed into the pipe one after the other ; coarse thread connecting screws are fixed to the rods. At the forward end of the first rod some attachment, or tool, is fitted to help in disturbing the deposits. These arrangements can be made up to a length of about 60 m, but 45 m is about the limit for good practical work.

(*iv*) A double disc made of two circular pieces of wood held about 30 cm apart by bolts is dragged through a sewer by means of a rope attached to it. This arrangement is useful for removal of silt deposits from small pipes.

(*v*) Wooden or rubber balls called "beach balls" or "pills" are tied with a rope and allowed to flow down the pipe. The ball dams up water behind it until sufficient pressure is created when the water forces its way with a scouring velocity between the ball and the invert of the pipe. The ball is of slightly lesser diameter than the pipe.

(*vi*) Where roots have been formed in a sewer and cannot be removed by mechanical methods, copper sulphate should be thrown in the pipe line for killing roots. This method has also been described under "Water Supply."

(*vii*) A hose pipe can also be thrust into a sewer pipe through a manhole, connected to a fire hydrant.

Before entering a manhole, the covers of at least three manholes should be removed half an hour beforehand for ventilation, one on either side of the manhole to be inspected is opened. This precaution is essential to remove the dangerous gases and vapours present in the sewers.

4. HOUSE DRAINAGE

Different Systems of Plumbing for Building Drainage

(*a*) *The Single-Stack System.* Both the soil water and the waste water are discharged into a single pipe through their traps. Only one vertical pipe is used and which is connected to the ground drainage system. Such a pipe should not be less than 100 mm dia. up to four storeys. The depth of the water seals here should not be less than 75 mm. This system is economical as only one pipe is provided. But disadvantages are : (i) Air or water from the drainage pipes may be forced up through traps by back pressure due to blockage ; (ii) Trap seals may be empetied due to sudden discharge (self-siphonage) from a fixture or due to discharge (induced siphonage) of another fixture in the system. The appliances should be grouped as closely as possible round the main stack so as to keep the branch pipes short and reduce noise. Branch connections should be of large radius along the invert (flat gradients reduce self-siphonage and noise). Waste pipes should fall at between 1 : 50 and 1 : 10. (Illustrations given on the pages following)

(*b*) *The One-Pipe System.* Both the soil water pipes and the waste water pipes are connected to one main vertical pipe and which is connected directly to the drainage system. The traps of all the fixtures are connected to a separate vent pipe. This system is economical in the case of high buildings where the sanitary fittinge can be grouped close together. In this system it is essential that : (i) The waste pipe shall join the stack above the soil branch at each floor ; (ii) All the traps shall be of deep water seal type of not less than 75 mm ; (iii) Vent pipe shall be of not less than 50 mm diameter. Since all the traps are ventilated by a separate vent pipe, this system is more efficient than the single-stack system.

(*c*) *The One-Pipe System - partially ventilated.* Here the traps of only the wcs, urinals and slop sinks, etc. are connected to the vent pipe.

(*d*) *The Two-Pipe System.* Two separate sets of pipe are installed, one for the soil water and the other for the waste water. Both the soil and the waste pipes are provided with vent pipes. The soil pipes are connected to the drain direct and the waste pipes through a trapped gully. There are four stacks in a two-pipe system.

Choice of System. It is usually, though not always, cheaper to install the two-pipe system on a small two or three-storey building, or any building where the sanitary fittings are not grouped close together. On taller buildings, such as multi-storey, block of offices and flats, where the sanitary fittings are usually fitted in groups one above another, the one-pipe system is generally more economical to install. It is very often a practical proposition to install the one-pipe system in one part of a building and the two-pipe system in another part of the same building.

The gradient of an horizontal branch should not be flatter than 1 in 50, nor steeper than 1 in 10.

Sanitary Fixtures or Appliances

Sanitary fittings are made in a great variety of designs and sizes.

Wash Basins. The top of the rim of a wash basin is about 75 to 80 cm—78.5 cm common, above the floor level. Where wash basins are to be provided in schools, the following heights may be provided :

Age Group—years	5 to 7	7 to 9	9 to 11
Height from floor level to rim of basin in cm	58	63	68

When basins are fixed in ranges, there should be a space of at least 100 mm between the basins and 70 cm between centre to centre of basins, and the centre of the basin at least 40 cm away from the adjacent wall.

When several basins are installed in a range, especially at different floors, it is essential that waste pipe from each basin is taken into a main waste pipe which should be carried up about 90 cm above the roof, and should discharge into an open gully at the bottom.

Where non-siphoning traps are used under basins, tubs and sinks (non-slop sinks), vent pipes may be omitted, except when the length of the branch waste pipe exceeds 150 cm before entering the vented line, or where more than one fixture is connected to an unvented branch line.

Sinks. Height of a sink to the top of the front edge is 860 to 900 mm and that of a draining board 1170 mm above the floor level. In food preparation rooms, canteens, hospital sinks should be fixed 75 mm clear of the walls to facilitate cleaning.

Bath-rooms. Min: size of a bathroom is 140 cm × 110 cm. Prefer at least 1.86 sq. m area room. Ventilator 0.2 sq. m. Bath rooms with bath tubs fixed to be 170 cm × 170 cm. Bath and WC combined room to be 240 cm × 180 cm

Water-closets. (WC) the term is used for the "fixture" as well the room in which it is installed. In residential buildings water-closet rooms should be located according to the direction of the prevailing winds and should peferably be made against external back walls. WC compartments should not have direct communication with a habitable room or a kitchen. The access should be from a passage, landing, hall, lobby or a similar space. An exception to this may be made where a private WC is provided for a bedroom. In schools and public places, care should be taken in planning to avoid the transmission to the working rooms of noise caused by sanitary appliances. The elimination of sound transmission to hospital wards can be best achieved by not allowing any sanitary compartment to abut directly upon a ward.

The min: dimensions of a water-closet room are :—

105 cm wide by 140 cm along the length of the pan where the doors open outside. The width may be reduced to 95 cm where a pedestal type of closet is installed. The smallest space for a water-closet compartment, where the doors open out is 80 cm × 110 cm. If the doors open inside, this should be 95 cm × 150 cm. A good ordinary size is 125 cm × 180 cm. The height of the room should be at least 240 cm, and it should have a window on the outside of not less than 0.3 sq. m area and also should have 0.3 sq. m of ventilation in addition. The walls of the closet rooms should be cement plastered for a height of at least 90 cm above the floor.

Types of Water-closets. There are two main types :

European (as commode, pedestal or hopper type)

Indian—squatting pan. (Illustrations given on the pages following)

The pan, has an S-trap with outlet pointing vertically downwards for ground floor use, or a P-trap with inclined outlet for upstair use.

The closets are of glazed earthenware, fireclay or white vitreous China.

Wash-down type, in which the contents of the pan are removed by a flush of water discharged into the pan.

Siphonic type, in which the contents of the pan are removed by siphonage.

The standard pan in general use is the wash-down type. A siphonic closet is a superior type with two traps and a larger water area than in the wash-down type, in consequence with less possibility of soiling the pan; the contents of the pan are removed more efficiently and the action is silent ; atmospheric pressure is utilized to assist the flush, and the water seal is deeper.

In the case of commode type water-closets preference should be given to those closets which have the traps above the floor and to those in which the connection with the soil pipe is taken direct through the external wall of the building. In the case of Indian type closets, those built with pool of water to receive the faeces are objectionable for their liability to splash. The pan should be provided with a small after-flush chamber (on the back side above the trap) to ensure proper sealing of the trap after each flush. The back of the pan should be as near vertical as possible to prevent fouling.

The closet outlets should be unglazed on the outside for about 50 to 75 mm for making of a cement joint. The trap has a hole at the upper portion which is jointed to the anti-siphonage pipe. An inspection cover is provided in the soil pipe opposite the trap to clean it when necessary.

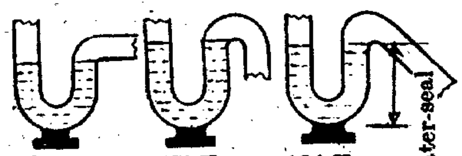
Installation of a Squatting Pan. The pan is sunk into the floor and embedded in a cushion of average 15 cm thick cement concrete 1 : 5 : 10. The concrete is left 115 mm below the top level of the pan so as to allow flooring and its bed concrete. The pan is provided with a 100 mm trap with 50 mm seal and 50 mm dia. vent horn for anti-siphonage pipe or vent pipe. The joint between the pan and the trap must be made leak proof with cement mortar 1 : 1 with fine sand.

Testing Closets for Flushing. The water-closet is filled with water to its normal water seal level and three or four loosely crumpled up papers made into balls of size about 150 × 150 mm and some cork bungs are thrown into the pan and flushed. This test is repeated four times and if the discharge of the pan carries away all at least three times through the trap, the pattern is so far a good one; (ii) The efficiency of the flushing rim can be tested by powdering the sides of the pan from above the water line to 40 mm below the flushing rim with lamp black or clay—there should be no powder remaining on the bowl.

Flushing Cisterns for Water-closets. Capacity is 10 to 15 litres. (In US flushing cisterns' capacity is as high as 20 to 30 litres.) Height from top of pan to bottom of cistern is 190 to 125 cm (min:) (180 cm average) for high level cisterns, and 30 cm for low level cisterns (siphonic type wcs).

Flushing pipe should not be less than 32 mm for high and 40 mm dia. for low level cisterns, and preferably of lead.

Traps. A trap is a bend or loop in a sanitary fitting which retains water and remains constantly full, shutting off air connection between the fitting and the outside soil pipe, thus preventing the escape of foul gases from the sewers into the house. The most common shape is P or S. The deeper is the water seal the more efficient the trap. Every inlet to a drain, other than a ventilating pipe to such drain, shall be properly trapped and such trap shall be so formed and fixed as to be capable of maintaining a water-seal of not less than 50 mm for inlet pipes up to 80 mm dia. and 75 mm water-seal for 100 mm dia. pipes.



'P' Trap 'S' Trap 'Q' Trap

Water Seal is the vertical distance between the dip and crown weir of a trap. It is the "depth of water" which when removed from a fully filled trap will permit the passage of air to pass through the trap.

Inspection Chamber is a sort of small manhole or chamber built in any house-drainage system which takes wastes from gully traps and disposes off to manhole, this is for inspection or cleaning. It is provided at every change of direction or gradient and at the point where the vertical soil pipe joins the house drain. Min : size can be —— 75×60 cm, the longer diamension is in the direction of the main drain line. Walls can be 20 cm thick.

Ventilation of House Drains. A fresh air inlet can be provided at the lowest end of the drain preferably in the lower-most inspection chamber and fixed with the compound wall. The fresh air inlet pipe may be 100 mm in diameter and about 3 m high above ground level, fixed vertically with an enlarged square head or chamber at top with a mica flap valve which opens inwards only and allows in fresh air but does not allow the foul gases to escape out. (These ventilating pipes are not, however, usually provided in small installations). In addition to this, house must be provided with ventilation on every soil and waste pipes coming out from every wc and bath-room. The soil pipes are taken up at least 1.2 m above the roof and which should be 4.6 m higher than the highest window of any house within 15 m radius. All such pipes should be provided with cowl ventilators at the top. (See also under "Anti-Siphonage or Vent pipe").

Intercepting Trap. Is also called a "disconnecting", or "sewer" trap. It disconnects the house drain from the street sewer and is fixed in a small chamber between the lowest end of the house drain and the street sewer. It has a deeper seal than an ordinary trap and an opening at the top called "cleaning eye". An intercepting trap should have a water seal of not less than 100 mm. Fresh air inlet described above is fixed in this chamber. House drains should have no direct connection with the street sewer.

Some engineers do not favour the fixation of intercepting traps and require the house drains to be connected direct to the street sewers as it provides ventilation to street sewers through the house drains. The intercepting traps are not properly flushed out and are thus of not much helyp.

Anti-Siphonage Pipe or Vent (or Ventilating) Pipe. When water-closets of more than one floor are connected with the same soil pipe, an anti-siphonage pipe is fixed to the outlet side of both the traps to prevent siphonic action and emptying of the traps in the lower water-closets when the top water-closet is flushed. The vent pipe should have an internal diameter of not less than 50 mm and be connected with the arm of the soil pipe at a point not less then 75 mm and not more than 300 mm from the highest part of the trap, and at a point above the over-flow of all connecting fixtures, to guard against the possibility of its being fouled where it joins the soil pipe, and eventually be choked by the waste water. The branch must always be made with the antisiphonage pipe bending in the direction of the flow. The vent pipe shall either be carried up as high as the top of the soil pipe and provided with a wire globe, or shall be connected to the soil pipe at a point not less than 2.1 m above the highest connection of any fitting to the soil pipe. A separate ventilating pipe shall be taken to a point 150 cm above the level of the eaves or flat roof, or the top of any window within horizontal distance of 3 metres. (See also under "Ventilation of House Drains").

Gully Traps and Gullies. The primary object of a gully trap is to cut off the house from direct communication with the drain and is an essential part of a house drainage, which is provided with a water seal. They are employed for the reception of waste water from sinks, baths, lavatory basins, rain water and surface water from paved yards, etc. There are two main types of gully traps. Those that are self-cleansing, and those that retain deposit, or catch-pit gullies. The former are used for foul water, and the latter for surface water or waste water. Gullies should be fixed as near the surface level as possible. There should be a grating on the top of the trap to intercept all solid matter; bars of gratings to be not more than 10 mm apart. Pipes should be connected to a gully below the grating or cover. Open gullies (*i.e.*, fitted with a grating) should be outside the building where required to take surface water, and sealed gullies inside the building.

Grease Trap. Is a device by means of which the grease content of waste water is removed. It is a variety of gully trap and is fixed outside canteen kitchens and wash-up rooms to intercept grease from entering the drainage system. (A small grease trap can be made underneath the kitchen and pantry sinks). In normal domestic drainage grease traps need not be provided. A grease trap is very essential outside a garage to intercept petrol and other oils which produce explosions in the sewage system, and such a trap should be effectively ventilated.

A kerosene oil tin placed in a pit can be used, the tin being well perforated towards the bottom and filled with sawdust and covered with grass. For big flows a masonry trap (chamber) should be built between the kitchen and the drainage system. This can be of size 1.8 m × 0.61 m or 1.7 m × 0.3 m (inside) and about 0.45 m deep, according to the flow. A galvanized perforated loose tray should be fitted at the bottom of the chamber, with handles long enough to reach above the level of the floating grease. The top of the chamber should be made air-tight. The inflow pipe should bend down and discharge about 100 mm to 150 mm above the floor and the out-flow pipe, which is fixed at the water level, should bend downwards inside the tank about 150 mm above the bottom.

The flushing cistern should have an overflow pipe one size larger than the supply pipe, with a minimum internal diameter of 20 mm. The overflow pipe should discharge outside the building whenever possible.

A screw-down isolation cock should invariably be fitted on the water-supply pipe to the flushing tank of a WC, urinal or slop sink, etc.

A reserve water tank of closed type, of not less than 350 litres capacity, should be provided for every seat, at the top of every building.

Flushing Range of Closets. Where a number of water-closets are fixed in a line, there will be difficulty of rapidly filling all the flushing cisterns. To overcome this, a levelling tank of small capacity should be provided in an adjacent position. From this tank is carried a large service pipe underneath the flushing cisterns with a 20 mm branch into the bottom of each, the orifice inside the cistern having a light copper tee valve. The levelling tank is provided with a large service pipe and a ball-valve which cause it to fill very rapidly. Thus each flushing cistern is filled rapidly, and the need of a ball-valve and overflow pipe is avoided.

Characteristics of a good flushing cistern : (i) A good flushing cistern should discharge by siphonic action, and the action should be induced by the movement of a part of the fitting within the cistern ; (ii) It should be certain in its action, whether the handle or chain is pulled slowly or quickly ; (iii) It should not be capable of being set in action when the cistern is not full ; (iv) It should not make much noise in discharging ; (v) The discharge pipe should be large enough to empty the cistern is about six to ten seconds.

Urinals. There are four types of urinals : (i) Bowl type—flat back and angle back ; (ii) Slab type ; (iii) Stall type, and (iv) Squating plate. Height of a bowl or lipped type urinal above foot level is 75 cm.

Urinals in Ranges or Stall Urinals

Centre to centre of partitions or screens ... 60 to 70 cm
(Prefer greater distance so that the users do not soil their clothes)
Depth of partitions ... ... 57 cm
Depth of end partitions ... ... 63 cm
Height of partitions ... ... 140 to 160 cm

Height of screen or partition where desired to be fixed above floor ... ... 67 cm
Depth of bottom slab ... ... 61 cm

The foot rests or treads should be of the non-slippery type and slightly sloping towards channels.

Any metal gratings in connection with urinals should be of gun-metal. (Illustration given on the pages following)

Ordinary brick walls cement plastered are not suitable for urinals. White glazed tiles should be used ; the joints must not be thick as they would present an absorbent surface.

Urinal outlets should either be directly connected to the soil pipes or provided with a gun-metal or brass dome-shaped removable grating. The waste pipe for each stall is 50 mm in diameter. Urinal waste pipes should preferably be of lead. The discharge from a series of stalls is usually through a glazed semi-circular drain which has a sharp fall and discharges into a gully at one end, from which it runs into a soil pipe or house drain under similar arrangements as for a water-closet.

The number of stalls draining into any one outlet should not be more than 7 (or 4.25 m run of channel). The maximum run of channel in any one stall should not exceed 2.1 m.

Flushing Cisterns for Urinals. For public urinals in single stalls or ranges, automatic flushing cisterns should be installed. The usual allowance of water for each stall in a public urinal is 2.5 litres per minute. A ball valve and overflow are not necessary in an automatic flushing cistern.

No pipe from a water supply distributing pipe or tank shall be connected direct to a WC or a pan, except through proper flushing tank.

Capacity of flushing cisterns and relevant size of pipes

No. of urinal stalls in range	Capacity of flushing cistern	Size of flush pipe Main		Distribution	
		B	S	B	S
One	5 litres	20 mm	25 mm	15 mm	20 mm
Two	10 litres	20 mm	25 mm	15 mm	20 mm
Three	15 litres	25 mm	32 mm	15 mm	20 mm
Four	15 litres	25 mm	32 mm	15 mm	20 mm

B is for bowl type and S for squatting plate type of urinals.

Chemical Toilet. This usually consists of a seat attached to a metal cylindrical tank in which a strong solution of caustic soda in water has been put. The solution of caustic soda sterilizes and liquefies the excreta in the tank and requires replacement only at intervals of several months. The contents of chemical toilet when emptied is liquid, sterile and practically free from odour. This is the most satisfactory method for the disposal of excreta without water carriage.

Size of Pipes and Traps for House Drainage

Soil pipes	100 mm
Main waste pipes	50 mm to 80 mm

Branch Soil and Waste Pipes :

	mm
For each WC	100
„ slop sink	80
„ urinal	50
For bath tubs	50
„ kitchen sinks	50
„ laundry tubs	50
„ wash basins	40

Short branch soil and waste pipes may be of heavy lead. Exposed branches at fixtures may be of brass. When branch soil and waste pipes receive a number of fixtures, their size may be increased as required.

Vent pipes : (main)	mm	*Sizes of Traps* : (min.)	mm
For WCs	80	For WCs	80
For other fixtures	50	„ slop sink	80
		„ kitchen sink	50
Branch vent pipes :		„ wash tubs	50
For each WC or slop sink	50	„ urinals	50
For other fixtures	40	„ wash basin & pantry sink	40

Galvanized and black wrought iron pipes should not be used for soil, waste, anti-siphonage, ventilating or drain pipes.

Testing of Drainage Pipes for Leakage

All drains and manholes should be subjected to water test and all vent and soil pipes (works above ground) should be tested either by a smoke test or water test. Works under ground should be tested before they are covered. Old installations are generally tested by a smoke test and new installations by water test. Not more than 1.8 m head should usually be applied to any part of a drain ; a greater pressure, especially when testing old drainage systems may develop defects not previously existing. In the case of extensive drainage systems, drains should be tested in sections. In the case of the water test, where there is a trap at the upper end of a branch drain, a rubber tube should be inserted through the trap seal so as to draw off the confined air as the pipes are filled with water.

Smoke Test. All traps are filled with water before starting the smoke test. A smoke testing machine consists of a length of flexible rubber tubing and bellows. The smoke is made by firing oily waste (brown paper or cotton waste soaked in creosote). Smoke is pumped into the drains and pipes through a gully outside the house or an inlet ventilator, or through a clay plug in an inspection chamber. In making a smoke test, the tops of soil and ventilating pipes are left open until smoke is seen to issue when the openings are plugged securely with wet cloth or wet clay tied in a cloth, and smoke pumped in for some considerable time.

If smoke-testing machine is not available, smoke rockets may be used which can be obtained from firework makers. The smoke produced is very dense and pungent, but the test cannot be prolonged as with a machine.

Water-Test for Stoneware and Concrete Pipes. After the joints have properly dried (for at least seven days) and before filling the trenches, the pipes should be tested for water-tightness by filling the pipes with water to the level of 1.5 m above the top of the highest pipe in the length to be tested, by closing the ends of the sections and maintaining this water level for one hour. Earthenware pipes should not be subjected to a head of more than 3 m of water. A plug is inserted at the lower end of each length and a right angled bend at the top and funnel fixed through a rubber tube or testing rubber plug is used. (A drain plug is a cylindrical bag of rubber and canvas to which a tube and a tap valve is fixed at one end. Air is pumped into the plug which is inflated and blocks the passage of water. If these plugs are not available use a wad of clay supported by a disc of wood.) After air bubbles have escaped after the first filling and absorption has ceased, water is again added to completely fill the pipe.

A slight amount of sweating which is uniform may be overlooked and a small amount of subsidence should not be taken as implying bad workmanship or defects. Absorption is at a diminishing rate of subsidence till no further subsidence takes place.

A tolerance figure of two litres per cm dia. per kilometer may be allowed during a water filled period of 10 minutes. It is considered satisfactory by some authorities if the water level does not fall more than about 14 mm in a length of 100 m.

The water put in the pipes for testing should not be drained out until the trenches have been filled in about 90 cm to detect if any joints have given way during the filling. Or alternatively, the test should be repeated after back-filling the trench.

Branch drains having a trap at their upper ends, unless they are provided with a cleaning or ventilating branch, become air-bound, so that the results of a water-test cannot be accurately gauged. To obviate this, the confined air should be drawn off by means of a bent tube.

Water-Test for Cast Iron Drain Pipes. The test for water tightness of the joint, the ends are closed by flanges. A small diameter pipe is inserted in the upper end and a valve for escape of air in the other end. The pipe is filled with water until water stands in the small pipe to the required height. The water pressure should be maintained for not less than 10 minutes. This method is used for heads up to about 6 m. Hydraulic pressure pump is also used, until gauge shows required pressure. If there are no leaky joints, the water level in the pipe or the pressure on the gauge will be maintained. Pressure pump is used for test of higher pressures than 6 m of head.

Whenever practicable, testing should be carried out from manhole to manhole. Short branch drains connected to a main drain between manholes should be tested as one system with the main drain. For domestic drains, a length of pipe line from 30 to 100 m is taken according to the diameter of the pipe (lesser length for bigger diameter).

Cast Iron Pipes for House Drainage and Plumbing work. Cast iron soil pipes are available in different lengths varying from 1.5 m to 3.0 m, with or without ears for fixing. (See under "Water Supply") The pipes should be blocked out at least 25 mm from the walls and securely fixed by means of short pieces of 20 mm galv. iron pipe and stout pipe nails. The pipe nails should be of wrought iron, and after allowing for thickness of distance pieces etc., to run not less than 75 mm into the wall. Joints in inverted and other difficult positions may be made with leadwool thoroughly well caulked instead of with molten lead.

Lead oxide or iron oxide are used for protecting waste soil and rainwater pipes. Hot coal tar is quite effective. Dr. Angus Smith's solution and other protective methods have been described under "Water Supply."

Approximate Weight of Lead and Gasket for Jointing
Cast Iron Soil Pipes

Dia. of pipe—mm	50	80	100	150	200	250
Lead—kg	1.2	1.8	2.0	2.9	4.0	5.8
Gasket—grams	56	76	76	90	90	112

Wrought iron or steel pipes should not be used for carrying discharge from water closets, urinals or slop sinks.

The threads of trap and fittings should be tapped so as to give a uniform fall to the branches of 1 in 50 for pipes 100 mm or larger in size, and of 1 in 25 for smaller sizes.

Lead Pipes. Should be used only for short branches of soil, waste or vent connections.

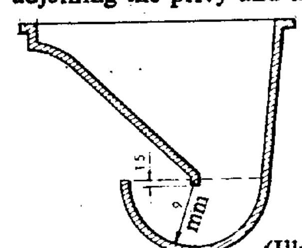
SANITARY LATRINES

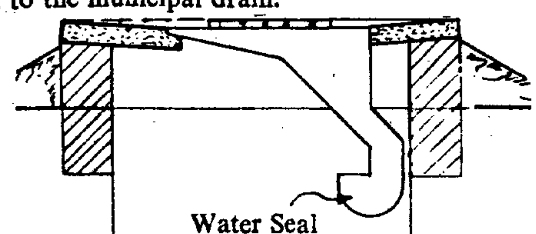
Bore-hole or Earth-pit Latrines. A vertical hole of size 25 to 30 cm is drilled with an earth auger up to a depth of about 3 to 6 m or more, without reaching ground water level. Where an earth auger is not available, a pit can be made of size 60 to 90 cm square. At the top portion of the hole an earthen, brick or concrete ring is provided and earth mounded up so that the squatting plate or the laterine seat is at least 25 cm above the general ground level to avoid rain or flood water running into the pit. A latrine seat or squatting plate can be made of stone or concrete with an opening of about 18 cm., for use. Size of the squatting plate should be 100×110 cm min. overall size and should normally extend to the superstructure walls. A cement concrete pre-cast slab 50 mm thick may be made which can be used for an unsupported span up to 75 cm. Foot-rests should form an integral part of the squatting slab where possible. Cover for the hole is desirable. A temporary soperstructure can be provided over the seat to afford privacy. When the hole gets filled up to about 45 to 60 cm below the ground level it should be filled with earth, closed and abandoned and the structure shifted to a new hole. It is of greatest importance to locate a privy on the downhill side of the ground. In sandy soils a privy may be located as close as 50 cm from a properly constructed household well if it is impossible to place it at a greater distance. In case of a higher-yielding public well not less than 15 m should be taken. A one-hole latrine is adequate for a family of five or six,

Water-seal Latrines. A pit is made as for the bore-hole latrines and the squatting plate used is provided with a water trap as for closets ; an ordinary water closet may be used. The pit can be made outside the building and the night soil led to it through a glazedware pipe fixed to the trap of the pan where a permanent structure is proposed to be built over the latrine. This follows the principles of a cess-pool. In areas with high water table a well is dug of 75 cm diameter extending to about 90 cm below ground water level, up to a depth of about 3 to 3.6 m. Half of the well is covered up by the squatting plate and the other half with a concrete or stone slab, which can be removed when it is desired to remove the sludge (after about 4 to 6 years), which can be used as manure.

Service Latrines. Special types of closets are now manufactured by some firms which are fixed in small compartments in place of the usual wc pans. These closets have their bottoms (front side) sloped at a greater angle than in the common closets and the back sides almost vertical, so that the contents can glide down with wash water only. Where a closet is not available, a 150 mm half-round glazed stoneware pipe can be fixed in a steeply inclined position. Such latrines are located on back side of external walls where removal of the excreta is convenient, A small chamber is made adjoining the latrine seat and under the wall in which a metal box is kept into which the night soil is led from the closet or the pipe. An automatic metal flap is fixed to the end of the chute of the closet which opens only when required to pass off its contents. The chamber has a shutter or flap of galvanized iron sheet on the outside opening so as to keep it closed. Where the night soil is not removed immediately a vent pipe should be fixed in the chamber for the foul gases to escape. If a lid is provided to the closet it will make it much better. This type of latrine can also be made on an upper floor ; the night soil is flushed down through a vertical glazed pipe in a collecting chamber at the ground level.

Septic Tank under Latrines (Aqua Privy). A small septic tank can be made adjoining the privy wall or underneath (former arrangement is preferable) a water-seal latrine instead of the pit. A vent pipe should be provided through the septic tank which can be fixed outside the privy wall. A manhole should also be provided for removal of the sludge. Where the tank is built underneath a privy, it should be so constructed that the manhole is made outside the privy wall. Inside dimensions of the septic tank may be 1.8 m×0.75 m and 1.5 to 1.8 m deep with 150 to 230 mm air space. The effiuent is passed on to a soakpit or a filter chamber built adjoining the privy and then to the municipal drain.





(Illustration given on the pages following)

5. PLUMBING & INTERNAL FIXTURES

Joints

Joints between pipes of	How made
Lead to lead	Wiped soldered joints. (Plumber's joints)
Lead and brass, copper	Plumber's wiped joint.
Lead with iron	Lead soil pipes should be connected with iron pipes by soldering on to the end of the lead pipe a flanged brass ferrule or thimble which may be caulked into the iron socket as in the case of iron pipes. Heavy brass soldering ferrules screwed to the wrought iron pipe fitting by socket and lock-nut.
Iron or copper and stoneware	Cement mortar joint as for stoneware pipes.
Lead and stoneware (Connection between earthenware WC trap and lead soil pipe.	A brass thimble (having socket at one end and spigot at the other) necessary to receive the trad outlet which have cement caulked joint. Wiped solder joint between brass spigot and lead pipe.
Cast iron and stoneware (Connection between earthenware WC trap outlet and cast iron branch)	It is preferable to introduce the brass thimble (as explained above) and a short length of lead pipe, otherwise the joint is too rigid to stand. Joint as above and as detailed below.
Lead pipe and socketed end of cast iron pipe	A brass or copper ferrule is used which is inserted over the lead spigot and in cast iron socket for connection which is caulked with lead joint and hemp yarn into the iron socket. A wiped soldered joint is made with brass ferrule and lead pipe.
Copper pipes (of light gauge) and cast iron	As above with brass ferrule. Joint between the ferrule and copper pipe is made with bronze weld.
Brass and stoneware	Neat cement.
Copper to copper	Bronze welding. Pipes are jointed by screwed joints. Compressed joint made with union nut or flanged couplings.

Joints between pipes of	How made
Cast iron water pipes	Red lead or cement.
Hot water cast iron pipes	Cement or red lead, or "rust cement" may be used as described further. Lead joints are not generally successful.
Asbestos cement	As for stoneware pipes.
Galv. Iron water pipe and a sanitary fitting	Lead pipe jointed to the plumber's brass union of fitting, with wiped soldered joint.
Flush pipe and flushing inlet arm of a sanitary fitting.	Red lead and white lead pressed into the joint and covered with a prepared lead cone.

Lead has the advantage of adapting itself to slight settlements and does not corrode. Brass and earthenware take cement but lead will not.

Min: length of wiped soldered joint for pipes of different diameters :—

Pipe dia. in mm	15	20	25	40	50	80	100	125	150
Min: length of joint, in mm	57	64	70	70	76	76	82	90	90

Joints between brass sockets or thimbles and cast iron pipes shall not be made with cement.

A lead wool joint is more expensive and more difficult to caulk than molten lead but its use is advantageous in some situations.

Joints of wrought iron pipes and fittings are screwed joints made up with a thick paste of white and red lead mixed. No slip joints or couplings in brass pipes, excepting flush pipes, should be permitted and these should be all screwed joints. Flanged pipes should be accurately aligned and tightly bolted up with proper flange rings or lead washers and red lead cement. Access doors are jointed with oil dressed leather and secured with gunmetal set-screws.

Ferrules, unions, etc., to be of heavy brass quality and when connected to lead pipes are to have tinned ends. Valves to be faced with leather for cold water, and with vulcanized fibre for hot water.

Solders for Plumbing Works. See Section 5—"Properties and Uses of Metals."

Cement for Hot Water Pipe Joints : (Rust cement)	Ordinary	Quick setting
	by weight	
Flowers of sulphur	1 parts	2 parts
Powdered salammoniac (Noshadar)	2 „	1 „
Fine cast iron filings	200 „	100 „

Sulphur and iron filings are first mixed dry and then salammoniac added. The mixture begins to rust and gets warm in the process. It should be kept covered with water until ready for use.

General Principles of Plumbing (Illustration given on the pages following)

(a) Stoneware pipes should not be carried through foundation walls as these pipes are liable to be broken by uneven settlement.

(b) No pipe carrying sewage should be buried under the basement floors. It should be placed in masonry-lined trenches with removable covers.

(c) Sanitary fixtures should be so fixed as to be completely exposed to view and easy to clean. As far as possible, sinks and water-closets should be installed against outside walls.

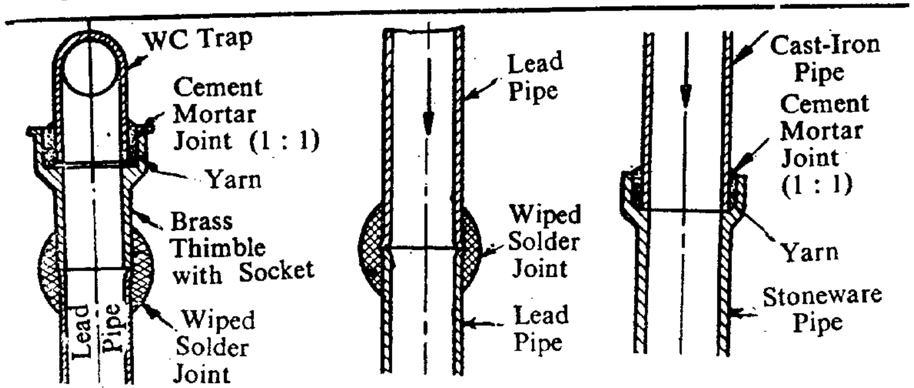
(d) It is preferable to provide white glazed floor channels to receive the wastes fiom series of wash basins, baths, etc., so as to avoid the necessasity for numerous traps and complicated pipe work. These channels to discharge into the waste pipe through 10 cm × 5 cm bell-mouthed floor traps provided with gratings.

(e) No soil, waste or vent pipes shall be drilled or tapped.

(f) The drainage and vent pipes shall be kept free of all other services, such as drinking water pipes. No soil, waste or vent pipe shall have any rain water pipe connected to it.

Max: No. of Fixture Units that can be connected to any portion of a Drain or Sewer for the Pipe Size and Gradient IS : 5329.

Pipe Dia.	Gradient			
in mm	1/200	1/100	1/5	1/25
100	—	180	216	250
150	—	700	840	1000
200	1400	1600	1920	2300
250	2500	2900	3500	4200
300	3900	4600	5600	6700
375	7000	8300	10000	12000



6. STANDARDS FOR PUBLIC SANITARY CONVENIENCES

"Sanitary convenience" means closets (or latrines), urinals, wash-basins, etc.

The following scales are recommened in IS:1172 :

Office Buildings

Fitments	For staff other than for officers — For Males	For Females
W.Cs.	1 for 25 persons	1 for 15 persons
Urinals	nil up to 6 persons 1 for 7 to 20 persons 2 for 21 to 45 ,, 3 for 46 to 70 ,, 4 for 71 to 100 ,, From 101 to 200 persons, add at the rate of 3% For over 200 persons add at the rate of 2.5%	—

Wash-basins—1 for 25 persons or part thereof.

Drinking water fountain—1 for every 100 persons with a min: of one on each floor.

Restaurants

Fitments	For Male Public	For Female Public
W.Cs. and Wash-basins	1 for 50 seats up to 200 seats. For over 200 seats, add at the rate of 1 per 100 seats or part thereof.	
Urinals	1 for 50 seats	—
Slop sink	1 in the restaurant	

Hospitals

Indoor Patient Wards

Fitments	For Males and Females
W.Cs.	1 for 8 beds or part thereof
Wash-basins	2 up to 30 beds ; add 1 for additional 30 beds or part thereof
Baths	1 bath for 8 beds or part thereof

Bed pan washing sinks—1 for each ward

Cleaner's sinks—1 for each ward

Outdoor Patient Wards

Fitments	For Males	For Females
W.Cs.	1 for 100 persons	2 for 100 persons
Urinals	1 for 50 persons	—
Wash-basins	1 for 100 persons or part thereof	

Medical Staff Quarters & Nurses' Homes
(Hostel Type)

Fitments	Same for Males and Females
W.Cs.	1 for 4 persons
Wash basins	1 for 8 persons or part thereof
Baths	1 for 4 persons or part thereof

For Administrative Buildings, take same as for office Buildings.

Hotels

Fitments	For Residential Staff	For Public Rooms
W.Cs. and Wash-basins	1 for 8 persons (Omitting occupants of the rooms with attached W.Cs.)	Same as for Cinemas and Theatres. 1 Wash-basins per W.C. and urinal provided.
Baths	1 for 10 persons	
Slop sinks	1 per 30 bed rooms ; min: 1 per floor	

Cinemas, Concert Halls & Theatres

Fitments	For Male Public	For Female Public
W.Cs.	1 per 100 up to 400. For over 400, add at the rate of one per 250 or part thereof.	2 per 100 up to 200. For over 200, add at the rate of one per 100 or part thereof.
Urinals	1 per 50 or part thereof	—
Wash-basins	1 for every 200 or part thereof.	

A closet (room) should have a min :—
(a) Floor area of 1.35 sq. m, with 0.90 m min: width ; (b) Height—2.4 m ; (c) Windows area—10 per cent of floor area, located in an exterior wall.
The water requirements per person is 15 litres per head per day.
See also at page 16/34.

Art Galleries, Libraries & Museums

W.Cs.	1 per 200 up to 400. For over 400 add at the rate of one per 250 or part thereof.	1 per 100 up to 200. For over 200 add at the rate of one per 150 or part thereof.
Urinals	1 per 50 persons	—
Wash-basins	1 per 200 persons or part thereof.	

Provision for Non-Residential Staff
for Cinemas, Theatres, Hostels, Hotels, etc.

Fitments	For Male Staff	For Female Staff
W.Cs. and Wash-basins (one for every W.C.)	1 for 1 to 15 2 for 16 to 35 3 for 36 to 65 4 for 66 to 100	1 for 1 to 12 2 for 13 to 25 3 for 26 to 40 4 for 41 to 57 5 for 58 to 77 6 for 78 to 100
Urinals	Same as for office buildings	—

Factories

Fitments	For Males	For Females
W.Cs.	1 for 1 to 15 persons 2 for 16 to 35 „ 3 for 36 to 65 „ 4 for 66 to 100 „ From 101 to 200 add at the rate of 3%. For over 200 persons add at the rate of 2.5%.	1 for 1 to 12 persons 2 for 13 to 25 „ 3 for 26 to 40 „ 4 for 41 to 57 „ 5 for 58 to 77 „ 6 for 78 to 100 „ From 101 to 200 persons add at the rate of 5%. For over 200 persons add at the rate of 4%.
Urinals	Same as for office buildings	

Washing traps—1 for 25 persons or part thereof
Drinking Water fountains—Same as for office buildings.

Schools and Hostels

Fitments	Nursery Schools	Other Schools For Boys	Other Schools For Girls	Boarding Schools & Hostels For Boys	Boarding Schools & Hostels For Girls
W.Cs.	1 per 15	1 per 40	1 per 25	1 per 8	1 per 6
Urinals	—	1 per 20	—	1 per 25	—
Wash-basins	1 per 15	1 per 40	1 per 40	1 per 8	1 per 6
Baths	—	—	—	1 per 8	1 per 6

Drinking water fountain—1 per 50 or part thereof.

For teaching staff, the schedule of fitments to be provided shall be the same as in the case of office buildings.

Sanitation Requirements for Railway Platforms & Bus Stations

The minimum sanitary conveniences provided at any railway station or bus station shall consist of one non-service latrine each for males and for females, and one non-service type urinal for males for a daily passenger volume up to 300 people.

For large stations the following may be taken :

(A) Junction stations, intermediate stations and bus stations :

W.Cs. for Males : 3 for first 1000 passengers and 1 for every additional 1000 or part thereof.

W.Cs. for Females : 4 for first 1000 passengers and 1 for every additional 1000 or part thereof.

Urinals for Males : 4 for first 1000 passengers and 1 for every additional 1000 or part thereof.

(B) Terminal stations and Bus terminals :

W.Cs. for Males : 4 for first 1000 passengers and 1 for every additional 1000 or part thereof.

W.Cs. for Females : 5 for first 1000 passengers and 1 for every additional 1000 or part thereof.

Urinals for Males : 6 for first 1000 passengers and 1 for every additional 1000 or part thereof.

General Notes :

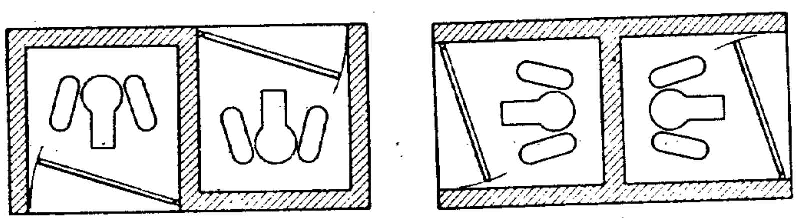
(i) Ablution taps : Provide 1 in each w.c. One water tap with draining arrangements shall be provided for every 50 persons or part thereof in the vicinity of w.cs. and urinals.

(ii) Cleaner's sink : Provide 1 per floor min:, preferably in or adjacent to sanitary rooms.

(iii) For public places like Cinemas, Theatres, Museums, Libraries, Hotels and Restaurants, it may be assumed that two-thirds of the numbers are males and one-third females.

(Based generally on IS: 1172)

PRIVY DESIGNS ENSURING PRIVACY AND SEPARATION OF THE SEXES



Latrine doors should preferably open inwards

7. HOUSE DISPOSAL WORKS

Disposal of Sewage from Single House or Small Units

SEPTIC TANKS

A septic tank (and which is a settling tank) is a rectangular chamber of brickwork cement, plastered inside, or of RCC, usually built underground. The function of a septic tank is to produce certain biological and chemical changes by partial liquefaction and gasification (or decomposition) of the human excreta discharged into it, through the action of anaerobic bacteria which flourish in the absence of free oxygen, humidity, darkness and warmth, and which are the conditions created in a septic tank, thus reducing the bulk of the sewage.

During the course of action in a septic tank the lighter matter (grease and fat etc.) rise to the surface and form a thick floating layer called "scum" while heavier matter sink to the bottom to form "sludge". The tanks are made air-tight, water-tight, and light is also excluded to help decomposition of the sewage. These layers of scum and sludge are not disturbed by the flow of water and the inflow and outflow from the tank are so arranged as to give least disturbance. Both the inlet and the outlet pipes are bent downwards and should have their open ends midway in the water. The centre of the outlet pipe is generally kept about 150 mm below the centre of the inlet. Inlets and outlets should be standard "T" fittings of glazed earthen-ware pipes. When the tank width is more than 1.5 m there should be two inlets. Another precaution taken against possible disturbance of the scum is by making a vertical partition in the tank, called a hanging "baffle wall", extending from above top-water level to 45 cm from the floor, and from 25 to 45 cm away from the inlet pipe. The baffle walls need not be made in small tanks. In long tanks a second baffle wall is sometimes provided which is built from the floor to a little below the water level in the tank and at a distance of about 60 cm from the outlet end, Instead of the second baffle a "scum board" may be provided to prevent movement of the scum by wave action or any other disturbance. A scum board consists of a thin slate, flagstone, RRC, or even wood, often suspended from the top into grooves in the sides of the tank to submerge about 25 to 30 cm into the liquid. The outlet pipe may be straight, instead of bent down, where a scum board has been provided. (Illustration given on the pages following)

(Designs for septic tanks recommended in IS: 2470 are somewhat different as shown in the appended illustrations.)

Design of Septic Tanks. The size of a septic tank is based on the number of users and the amount of dilution water in the sewage. Average retention period (for the septic action to take place) of sewage in a septic tank is 12 to 24 hours or even more where the sewage is fresh as in residential installations, and which are generally designed to hold 24 hours supply, and 8 to 12 hours retention where the sewage has travelled a long distance and has been subject to a process of some disintegration.

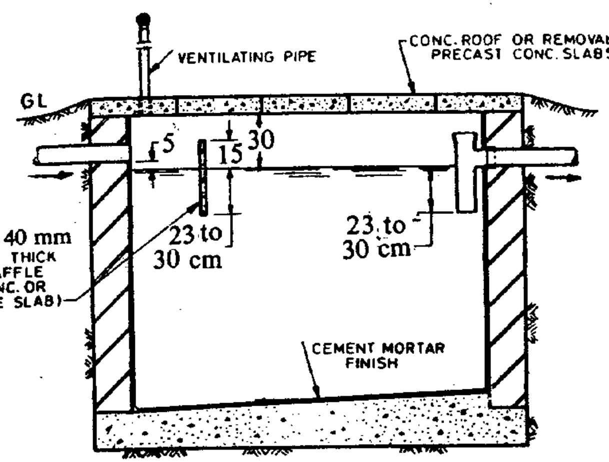
According to IS: 2470—Septic tanks shall have minimum width of 75 cm, minimum depth of one metre below water level and a minimum liquid capacity of one cubic metre. Length of the tanks shall be 2 to 4 times the width. Sizes of septic tanks as recommended are given below :

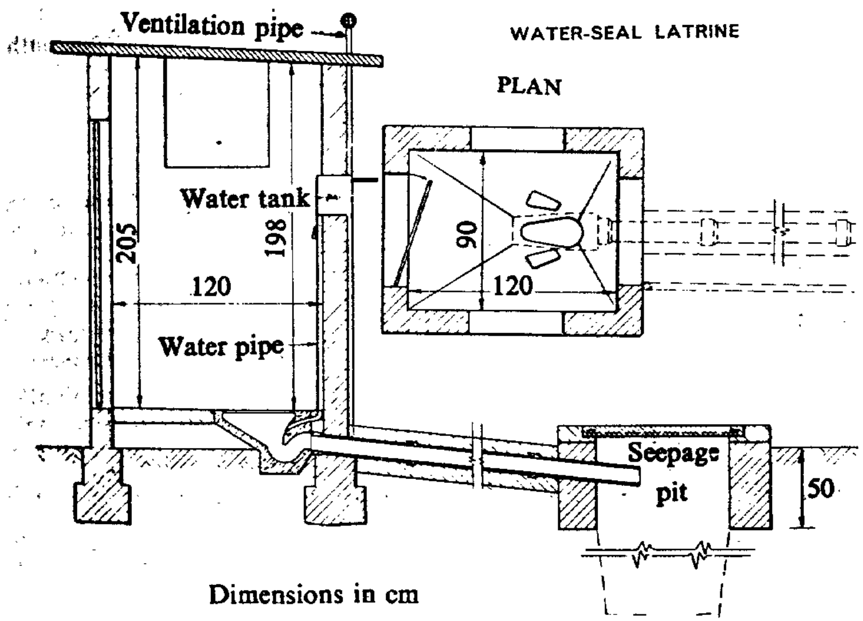
It is considered that as long as the dilution per head is more than 25 litres and less than 180 litres, a septic tank having a content of about 0.071 cu. m per user will do the work required. It has, however, been reported that septic tanks with as low a dilution as 9 litres per head per day built in water scarcity areas have also worked satisfactorily. According to the experiments carried out in India by Col. F.C. Temple, if the sewage is stronger than 25 litres per user, there is not enough water for the necessary reaction to take place in the tank, while if it is weaker than 180 litres per user it goes through the tank too fast and remains under treated. He limits the capacity of the tank between 0.0476 to 0.0672 cu. m per user. Effluents from both the over and under-treated tanks clog the filters. However, it is the general opinion that, for the proper operation of a house drainage system, a minimum water supply of 90 litres per capita per day should be available.

As it is not possible to estimate the exact number of users or the amount of dilution water for any proposed installation, it will be safe to provide a capacity of about 0.084 to 0.112 cu. m per user for small tanks and 0.07 cu.m for large tanks for Indian conditions where the baths and sinks are also connected with the system. Where only foul water from wcs flows into the septic tanks, a space of 0.042 cu.m per user may be provided for domestic tanks. Additional tank volume should be provided for sludge storage on the basis of about 0.05 cu.m per capita. The minimum capacity of a septic tank should not be less than 1.14 cu.m which is sufficient for 10 to 15 users. Width should not be less than 60 cm for a man to enter into the tank for cleaning. The length is generally 2 to 5 times the width which may be more to make the tank long and narrow so as to increase the length of travel of the sewage. Width of a septic tank should not be more than 275 cm for any single unit as great width induces the danger of uneven flow and local stagnation. Liquid depth should not be more than 1.8 m at the inlet end which is considered as ideal depth by Col. F.C. Temple. Liquid depth should not, however, be less than 1.22 m for a proper action. For more than 100 users it is preferable to make more than one tank in parallel units. The largest practicable size for a single unit is considered to be 22×2.7×2.1 metres which should be sufficient for 2500 users.

Surface and sub-soil water should be excluded from finding way into the septic tank. Waste water may be passed into the septic thank provided the tank and the means for for effluent disposal are designed to cope up with this extra liquid. Depending on the location of the water table and the nature of the strata, the type of disposal for the effluent form the septic tank is decided.

Septic Tank





Sizes for Septic Tanks

No. of Users	Length	Width	Liquid Depth
	metres	metres	metres
Domestic Tanks			
5	1.5	0.75	1.0
10	2.0	0.90	1.0
15	2.0	0.90	1.3
20	2.3	1.10	1.3
50	4.0	1.40	1.3
Tanks for Housing Colonies			
100	8.0	2.6	1.0
150	10.6	2.7	1.0
200	12.4	3.1	1.0
300	14.6	3.9	1.0
Tanks for Hostels and Boarding Schools			
50	5.0	1.6	1.3
100	5.7	2.1	1.4
150	7.7	2.4	1.4
200	8.9	2.7	1.4
300	10.7	3.3	1.4

The manhole caver is to be of size 50 cm diameter if circular or 60×45 cm if rectangular (min. sizes).

A vent pipe for removal of gases of 50 mm min: dia. should extend to a height of 2 metres above the top of the building when the septic tank is located closer than 15 metres from the nearest building. A newly constructed septic tank should be filled with water to its outlet level before the sewage is let into the tank. A small quantity of decaying organic matter such as digested cowdung should, preferably, be put into the tank to start with.

Capacities recommended provide for waste water also from bath-rooms, sinks, etc.

An air space of 30 cm is provided above the water line.

These sizes include for one year sludge storage.

(Based on IS: 2470)

The tank should be built as far away from the house as possible on the leeward side. If the tank is located at a greater distance, the vent pipe may be taken to a height of about 3.5 to 4.5 metres. The floor of the tank should slope at 1 in 30 towards one side or the centre to facilitate cleaning of deposits, and the manhole should be above this. IS: 2470 recommend a slope of 5 to 10 percent towards the inlet. The tank is usually covered with RCC slabs, removable or fixed, with manhole covers.

Working and Care of a Septic Tank. A septic tank should be initially filled with water. Disinfactants should not be used beyond very small quantities which may be absolutely necessary, since they kill bacterial life and the septic tank will not function. Soap and grease from bath-rooms are also harmful.

The working of a tank can be judged by the scum, it should be thick and unbroken. If there is thin scum or no scum at all and the liquid is quiet and grey, the sewage is very dilute, and passing through largely unchanged. If the liquid is black and bubbling, or "foaming", the tank is too large for its work, and over-septicization is taking place due to the sewage and the sludge having been too long in the tank, and removal of the sludge is indicated. Sludge deposited in a tank must be removed at

least once a year and should never be allowed to accumulate to a depth of more than one-third of the depth of the inlet compartment. The scum is not removed as it never interferes with the working of a tank.

Where the effluent from a septic tank is taken either in soak trenches or for land irrigation, the water is absorbed by only a small area near the end of the discharge pipe, as the flow is more or less of the nature of a drizzle, and which gets saturated with it leaving the remaining length dry. This can be remedied by storing the effluent in an extra chamber called a "Dosing Chamber" buit in continuation of the tank. A siphon is fixed in this chamber or a tipping trough (described earlier) which discharges in automatic flushes at intervals after a certain quantity has been filled in. This also helps in flushing the drain. Size of this chamber can be 75×90 cm or of the same width as the tank according to the flow expected. Depth may be 80 to 110 cm (floor to ceiling).

For a single house of ten to twelve inhabitants built on gravel or chalk or other permeable strats, there is nothing to equal a well for sewage disposal, provided the sub-soil water in the neighbourhood is not used for domestic purposes. The well may be about 9 m deep and 60 to 90 cm diameter and only lined sufficiently deep to prevent the sides falling in. Such a well will work for many years, and when one well has lost its efficiency, it can be filled in and another one dug on the opposite side. Such wells should be about 15 m away from the building, and should be covered.

Use cast iron pipes for sewage up to a length of 1.2 to 1.8 m from the house to the septic tank and beyond that glazed earthen pipes ; diameter is about 100 mm to 150 mm. Give a slope of 1 in 40 to the sewage pipe to enable the solids to flow down. Provide manholes for lengths over 90 m.

Disposal of Effluent from Septic Tanks

Cesspool or Seepage Pit. The effluent may be disposed of in a cesspool. The cesspool should be built 15 m away (min:) from the building. It is a small circular chamber built below ground level. Depth is 1.5 m from effluent pipe inlet to bottom. It may be built of brickwork or stone, with joints without mortar towards the bottom. For large flows, two or more cesspools can be made at distances of not less than 3 diameters of cesspool apart from each other. Size of the pool will depend upon the absorptive capacity of the soil and the number of persons using the system. Diameter may be 1.2 to 1.5 m for rapid absorption soils and 1.8 m for medium absorption soils. (IS: 2470 recommend min: size of 1 m depth and 0.9 m dia.) For slow absorption soils it is preferable to provide in duplicate, of minimum size. For size of the cesspool, take absorptive area per user equal to 1.4 sq. m for rapid absorption soils, 2.3 sq. m for medium and 3.7 sq. m for slow absorption soils. Absorptive area of a cesspool to be the area of the bottom plus area of walls with open joints.

A cesspool is lined with bricks or stones with open joints below the inlet pipe level and with mortar joints towards the top forming a pacca masonry ring and which may be made conical or narrowed to reduced the size of the opening for a cover at the top. The pit may be filled with gravel or brick-bats for about 30 cm. Where the lining is to be omitted, it should be all filled with gravel brick-bats or coarse sand, etc. The effluent from the septic tank is led in a pipe of 100 mm dia. to pour its contents in the centre of the pit.

The bottom of a cesspool should not be less than 60 cm above the sub soil water. A soak-pit should not be less than 30 m away from a well or any source of water-supply.

Relative absorption	Type of soil	Approx. range of loading rates in litres/sq. m/day
Rapid absorption	Coarse sand, gravel	140
Medium absoption	Fine sand, sandy loam	140 to 70
Slow absorption	Clay with sand or loam	70 to 30
Semi-impervious	Dense clay	30 to 23
Impervious	Rock	Below 23

8. DISPOSAL OF SULLAGE FROM TOWNS

Site of Disposal Works. Disposal works generally are the main problem in a sewerage scheme and require very careful consideration. Place of disposal of sewage is of primary importance and very often it governs the entire scheme. Accurate levels of the whole town with its out-skirts and proposed site of the disposal works, are very essential. Selection of site will depend upon the method of treatment to be adopted. Ideal conditions are very seldom met with and therefore, in most cases a compromise has to be made.

The British Ministry of Health Recommendations for the quantity of sewage to be treated at Sewage Purification Plants :—

Where the sewers are upon the "combined system" the works shall be capable of providing full treatment for a quantity equal to three times the mean dry weather flow and in addition to this there shall be means of partially treating a further quantity equal to the above, any excess to be discharged over the stormwater overflow.

In towns sewered on the "separate system" the full treatment to be provided for a quantity equal to twice the mean dry weather flow, and partial treatment for four times the dry weather flow.

The above degree of dilution are to be calculated on the average rate of flow thoughout the 24 hours.

Due allowance must be made for future expansions and increase of population as explained earlier.

Sometimes it will be found more advantageous and convenient to divide a town into separate high and low level zones at different sites and to make their own independent sewerage systems and disposal works. Or, to collect the sewage of the low level zone in a sump and pump it to the high level zone site for treatment and disposal.

However, an ideal site would be, to which all the sewers will flow by gravitation with self-cleansing velocities and where the last sewer will still have fall enough to discharge itself at such a level as will allow the water

to pass through all the operations required for purification. This means that the site of purification works should be roughly 3 to 5.5 m below the lowest level in the town end, according to the length of the sewers and the magnitude of the works, and it should also be below the level of the surrounding properties.

The land selected for purification works should not be very far from the twon as it will need long lengths of outfall sewers at extra expense and there would also be likelihood of sewage getting septic during the long travel ; works too near the town will also be source of nuisance. There should be no danger of the works being inundated and damaged by floods or heading up of the sewage water. The sub-soil water level should also stay sufficiently low even during the wet season. If possible, a site on the leeward side of the town should be selected.

Sewage Treatment. The types of sewage treatment are divided under two heads, "primary" and "secondary". Primary treatment comprises of processes which include grit settling, plain settling and precipitation of colloidal particles by the addition of chemicals. Secondary treatment includes dilution of sewage by water, land irrigation, trickling filters and activated sludge process.

Screens or Racks. Screens are provided in front of pumps and sewage treatment plants to remove the large suspended or floating matter other than faeces, generally consisting of pieces of wood, paper, rags, and animals. It is important that the screening chamber is so designed that the velocity of the sewage is not checked, for if it is, grit and putrefactive matter settle on the inver. The screening chamber is made 2 to 2½ times wider than the waterway and the floor kept steeper than that of the sewer to keep up the velocity of flow. Length and height of chamber is so fixed which will not cause overflow if removal of refuse is delayed for sometime. For a small 150 mm dia. sewer on a gradient of I in 100, a chamber of size 3 to 3.6 m long, 1.2 to 1.8 m high and 30 to 60 cm wide, with a 1 in 40 floor slope, with duplicate screens, is considered suitable.

Opinions differ as to the size of spacings between the bars of screens. Coarse sereens are made of bars with openings of 5 to 15 cm. These are followed by fine screens with openings of 2.5 to 10 cm. The bars are rectangular in cross-section, usually about 10 mm × 40 mm, and are placed with the long dimension parallel to the flow. The bars of the screen are fixed up and down (not horizontal) with the top inclined downstream at an angle of 30 to 60 degrees to minimise clogging and facilitate cleansing. Curving the bars at top will permit a rake to be drawn through more conveniently. The bars may be fixed with an angle iron at the top and bottom and the screen may be fixed or movable. Where two screens are fixed in the same chamber, there should be sufficient space between the two screens to permit easy removal of screenings. At small works hand-raked screens are most suitable. Mechanical sreening is recommended for flows in excess of 23 million litres per day.

The quantity of screenings removed varies with the size of openings, and may be about 6 to 60 cu per million cu. m of sewage. The simplest method for disposal of the screenings from small installations is to bury the same in shallow trenches and cover with earth, while for large installations, they may be passed to sludge digestion tanks where provided, or destroyed by incineration. Sometimes the screenings are returned to the flow of sewage after maceration. Detrius and screening chambers are generally combined for small projects. In big projects cutters and shredders are installed to reduce the screenings to small pieces so that they can pass through pumps and be digested with the other sewage solids.

Grit Chambers or Detritus Tanks. These are small longitudinal detention chambers made to remove heavy inorganic matter such as sand, gravel, grit or road metal. As sand and ashes are used in India for the cleaning of utensils, removal of grit is essential for the protection of pumps where sewage from combined systems is to be treated and for the efficient working of the purification works where sludge is to be treated by digestion. A grit chamber should be provided at the end of outfall sewer and prior to the septic tank (or setting tank). It should be small and narrow in proportion to length to check the velocity of flow, which should not be more than 30 cm per sec. so that it may deposit heavy solids and allow the floating faeces to pass on to the septic tank. The capacity of a grit chamber may be about 1/100 of the daily average flow with detention period of 1/2 to 3/4 minute. The area of a tank is determined by surface loading and the depth by the detention period. The grit is considered to settle 30 cm in about 16 seconds, and the chamber should be about 30 cm long for each 2.5 cm that particles must settle. A length of 10 m is considered sufficient for small installations. The depth should be about 1/16th of the length: the usual depth of a grit chamber is 90 to 120 cm The quantity of grit is considered to be about 20 to 30 cu. m per million cu. m of sewage. The bottom of the chamber is made trough or hopper shaped with narrow invert for the collection of grit which may be removed manually at intervals of about 4 to 10 days. The inlet and outlet ends should be flared to avoid eddies. Tanks should be built in duplicate.

For small works the detritus tank may serve as screen chamber and its size is mainly determined by the proportions of the screen.

The term "grit chamber" is used for a small size tank and "detritus tank or chamber" for a big size tank.

Skimming Tanks are provided where grease and oil are found in sewage, such as from garages and hotels, as they are likely to form scum in sedimentation tanks or clog filters and interfere with oxidation in aeration tanks. Skimming tanks are about 90 cm deep and have a detention period of about 3 to 5 minutes. These tanks usually work with compressed air which is passed through porous plates or perforated pipes fixed at the bottom of the tanks. The scum is removed by hand. Scum boards or baffles (also called partitions or curtain walls) are fixed about 10 to 15 cm below the scum surface for the liquid to flow under them continuously.

Sedimeneation Tanks or Settling Tanks or Clarifiers

Grit chambers, skimming tanks and screens are provided ahead of sedimentation tanks.

Sedimentation tanks are made to remove by gravity the settleable or

suspended solids thereby greatly reducing the strength of a sewage for its further treatment. The detail design of sedimentation tanks is more important than their capaeities and the efficiency of a tank is greatly dependent on the design of its inlets and outlets. A plain sedimentation tank is a long narrow horizontal tank with length varying from 4 to 5 times the breadth; depth is 1.8 to 3.6 m usually about 2.7 m. Detention period is 2 to 3 hours with a velocity of about 30 to 45 cm per minutes, and overflow rate of 30 to 40 cu. m/sq. m per day to bring about sedimentation of the suspended matter. Shallower the tank, the shorter nced be the detention period. Additional provision for sludge deposit at the bottom has to be made, which is removed after a week or 10 days. Large tanks are made into two or more compartments lengthwise. Inlets and outlets should be so arranged that do not cause disturbance to the flow. Inlet may be led into the tank downward for about 30 cm. The sewage should enter the tank quietly and at a low velocity and any turbulance should be dissipated in a silting chamber.

The outlets should be weirs of such length that the flow over them is very shallow. Baffles can be provided in front of them, about 30 cm below and 23 cm above the surface of sewage. Instead of single large pipes for inlets and outlets, a number of pipes produce less disturbance. Two or three submerged overflow baffles placed across the direction of the flow, extending upwards about a metre above the bottom will break up bottom currents. Floor is made inclined towards the inlet with an outlet valve for sludging where there is maximum of sludge deposit; slope is 1 in 20 to 24, or the tanks should have a central narrow invert to which the flow of the tank slopes. Where tanks are made with pyramidal bottoms, they are usually sloped at 60 deg. or steeper. There should be sufficient capacity for sludge storage below the inlet level; a practical rule of thumb is to allow 10 litres per head of population. The usual method is to use the tanks continuously by drawine off over a long weir extending the whole or part of the width of the tank, in a thin film at the same rate as the filling.

The number of tanks should be sufficient to provide for the maximum capacity required, and in addition at least one extra tank for cleaning. There appears to be no advantage in covering a tank from the working point of view, but if covered it should be by creosoted planks laid loosely; permanent covering is not recommended.

Dosing Tank is a tank into which raw or partly treated sewage is collected, let to stay, and discharged at such a rate as may be necessary for subsequent treatment. The capacity is equal to five litres a sq. m of the filter bed.

Chemical Precipitation. Certain chemicals are added to facilitate the removal of suspended solids where sewage contains high percentage of solids or industrial waste. Chemical precipitation is employed to increase the quantity of solids removed by sedimentation and also to expedite the process of settlement. Much smaller size of tanks are required. It is possible to remove as much as 85 to 90 per cent of the suspended solids by this process. As the addition of chemicals is quite expensive, they are used only under special circumstances. Chemicals are added to the sewage prior to its entrance into the settling tanks and after having passed through screens and grit chambers. Chemicals generally used are : lime, alum, iron salts, sulphuric acid, copperas, etc., either alone or in combination with others. Lime is most frequently used, and where sewage contains waste from breweries, 150 to 400 grams per cu. m of sewage is the dose. If alum is used, 90 to 130 grams per cu.m are required. Exact dosage depends upon the character and the strength of the sewage. The coagulant should be added in the form of solution ; it should be quickly and thoroughly mixed with the sewage which should then flow into the coagulation tanks with detention period varying from 15 to 30 minutes, before going to the settling tanks. The process will produce more sludge than by plain sedimentation and this sludge has no value as a fertilizer. Effluent is fairly clear with very fine suspended colloidal solids.

Sewage Filtration : General Principles

After the sewage has been passed through a settling tank, the more or less clarified effluent is applied to an aerated bed or filter in a finely divided form, it trickles down through the interstices, and during its slow passage through the filter it becomes oxidized and rendered free from putrefaction.

Where disposal of sewage by land treatment or by dilution is not possible or suitable, filtration methods are employed which consist of passing the sewage through some filtering media such as gravel, crushed stone, broken bricks, slag, cinders. All these materials have about the same efficiency, therefore, availability and cost should be considered while choosing. Size of the filtering media depends upon the strength and character of the sewage, coarseness or fineness of the materials and the thickness of the proposed bed ; size generally varies from 6 to 75 mm. Coarser materials are used in deep beds and for filtering strong sewage, and finer materials for shallow beds and weak sewage. Smaller size produce better results but the filters are liable to get clogged earlier. A bed of from 1.8 to 3 m depth is generally considered suitable for sizes varying from 40 to 75 mm and of 1 to 1.5 m depth for sizes 12 to 25 mm. A filter should never be very shallow otherwise the liquid to be treated would not be long enough in contact with the material. The deeper the bed within certain limits the better will be the effluent. Shallow filters can be arranged to operate in series to give better results as the effluent can be recirculated. Materials of uniform size are generally preferred as they have more voids and give better circulation of air ; different sizes are placed in separate layers with coarser materials at the bottom.

The effluent of the primary filter is collected and resprinkled over the secondary filter and becomes re-aerated and redistributed. The primary filter may be made of somewhat large material, and the secondary filter may be composed of much finer material, which gives better purification.

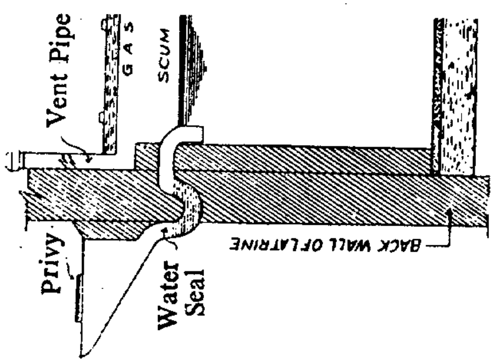
Presedimentation (screening and removal of grit and grease) of the applied sewage is essential to good performance of the filters otherwise suspended solids will cause clogging. Most of the suspended solids are removed by the above preliminary treatment but the very fine suspended solids and the dissolved materials still remain to be removed. When the

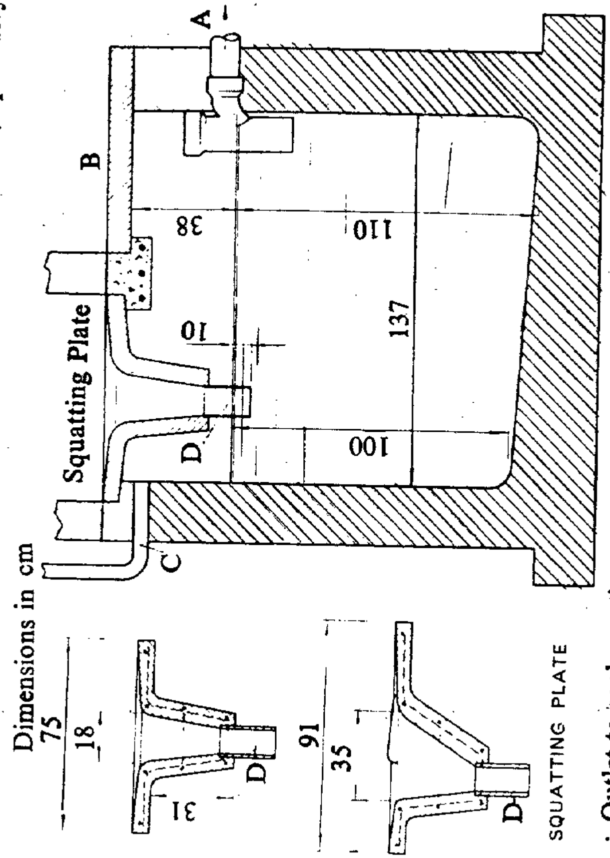
sewage is passed over the filters, the surface of the contact materials is coated with a slimy gelatinous organic film which acts upon the bacteria and the sewage solids coming in contact with it and transform the organic matter and purify the sewage. This film is formed only in the course of about 15 to 20 days, which may be decreased if filter effluent is returned to the bed ; the action is incomplete till this film has been formed.

Contact Beds are water-tight compartments or tanks of masonry, usually made below ground level, filled with filtering materials about 1.2 to 1.8 m deep. Concrete floors with under-drains are provided for collecting and draining out the effluent. The pre-treated sewage is filled in slowly in about 1 to 2 hours through an opening near the surface, or is preferably, distributed on the bed by means of troughs placed at the bed surface (as described later under trickling filters). or it may be discharged over perforated corrugated iron plates laid over the filter, without disturbing the gelatinous film. The tanks are allowed to stand full for about 2 hours and then emptied slowly in about 1 hour; are then given rest for about 4 to 6 hours, and the operation repeated again. A full day's rest is occasionally given to each bed in turn, and a thorough flushing and cleaning of the filtering materials is necessary in about 4 to 6 years. It will remove about 60 to 70 per cent of the organic matter and about 80 to 90 per cent of the suspended solids. The size of a filter should be in accordance with the volume and strength of the sewage. This method is not so efficient as the Trickling Filters described below but is adopted where head is limited and pumping is not desirable. The efficiency can be improved by repeating the process on a finer bed.

Sludge Digestion. The sludge accumulated at the bottom of a sedimentation tank and the scum of fresh solids floating on the surface have to be dealt with at intervals. In small installations these materials are removed and disposed of in shallow trenches and lightly covered in with the excavated earth. For big installations, sludge tanks or beds are constructed near the sedimentation tanks into which the sludge from the latter is run at intervals and allowed to dry. The amount of sludge collected at the bottom of the sedimentation tanks is taken equal to about 2.25 cu. m per thousand of population per day. The slude is discharged on to the beds to a depth of about 25 cm, left to dry, (for about 4 to 8 days) and removed for further drying on a dump after it has become semi-solid and can be removed with a spade. One square metre of bed is required for every six to seven heads of population, and at all works there should be 4 to 6 individual beds to permit rotation of the drying process. When dried, sludge is removed as an inert material which is valuable as a fertilizer.

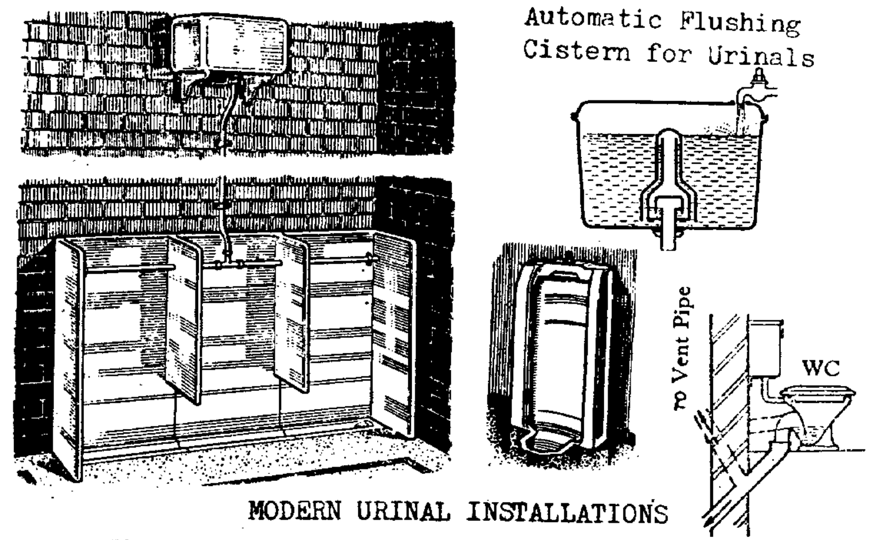
Percolating or Trickling Filters. This method of treatment is by far the most common method of sewage aeration and is used for works of all sizes. Percolating filters are more or less like the "contact beds" described above except that the walls are made honeycombed or otherwise provided with openings for the free circulation of air all through, and are usually built above ground level. Sometimes walls are not provided and the filtering material are stacked in their natural angle of repose. The pretreated sewage is applied continuously to the surface by sprinkling or



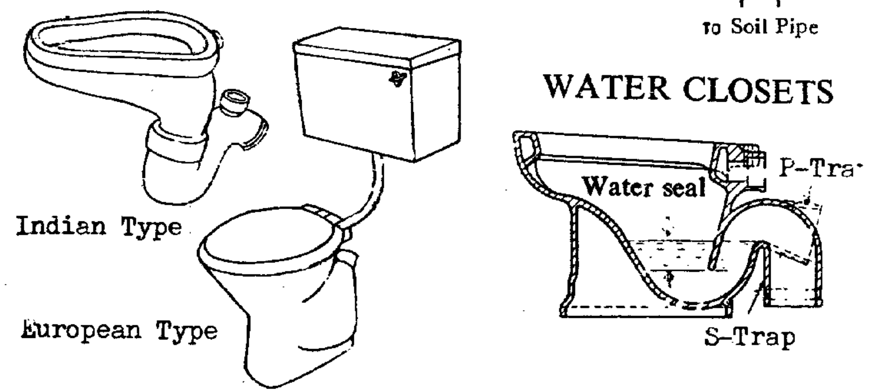


Septic Tank under Latrines (Aqua Privy)

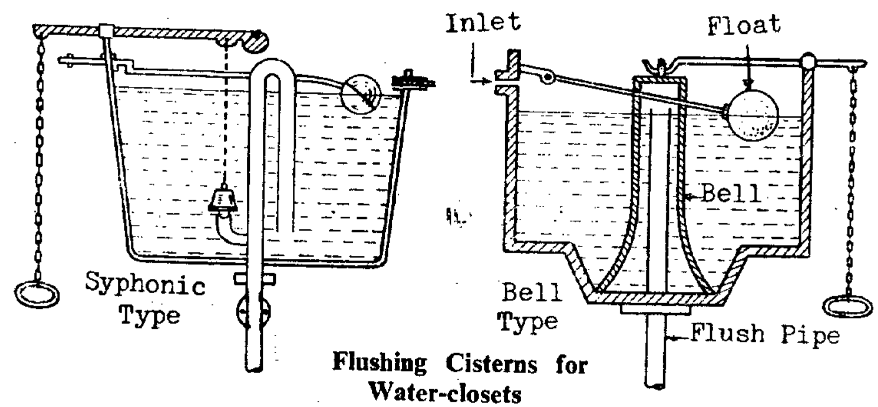
A : Outlet to soakage pit, B : Removable RC cover slab, C : 25 mm dia. pipe ventilator, D : Drop pipe.



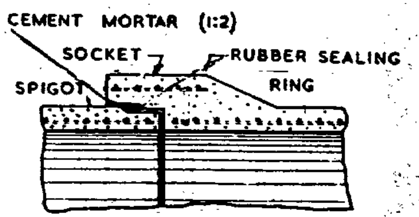
MODERN URINAL INSTALLATIONS



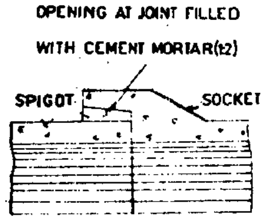
WATER CLOSETS



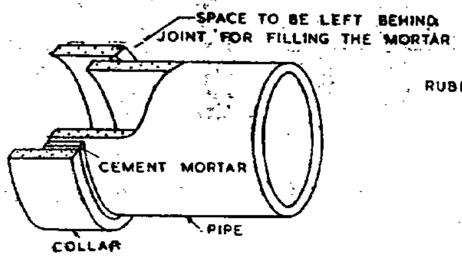
Flushing Cisterns for Water-closets



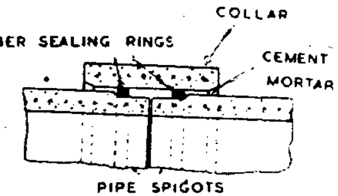
SPIGOT AND SOCKET JOINT (SEMI-FLEXIBLE TYPE)



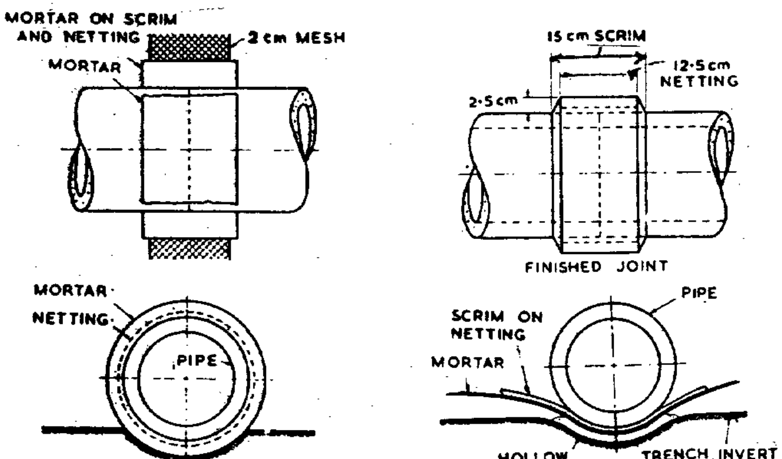
JOINTS OF CONCRETE PIPES

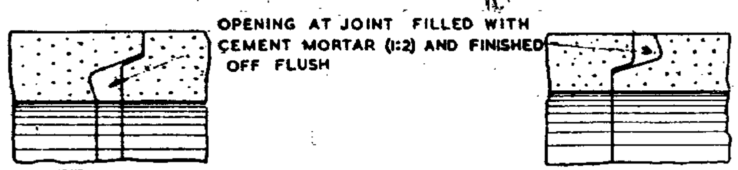


REINFORCED CONCRETE COLLAR JOINT (RIGID TYPE)



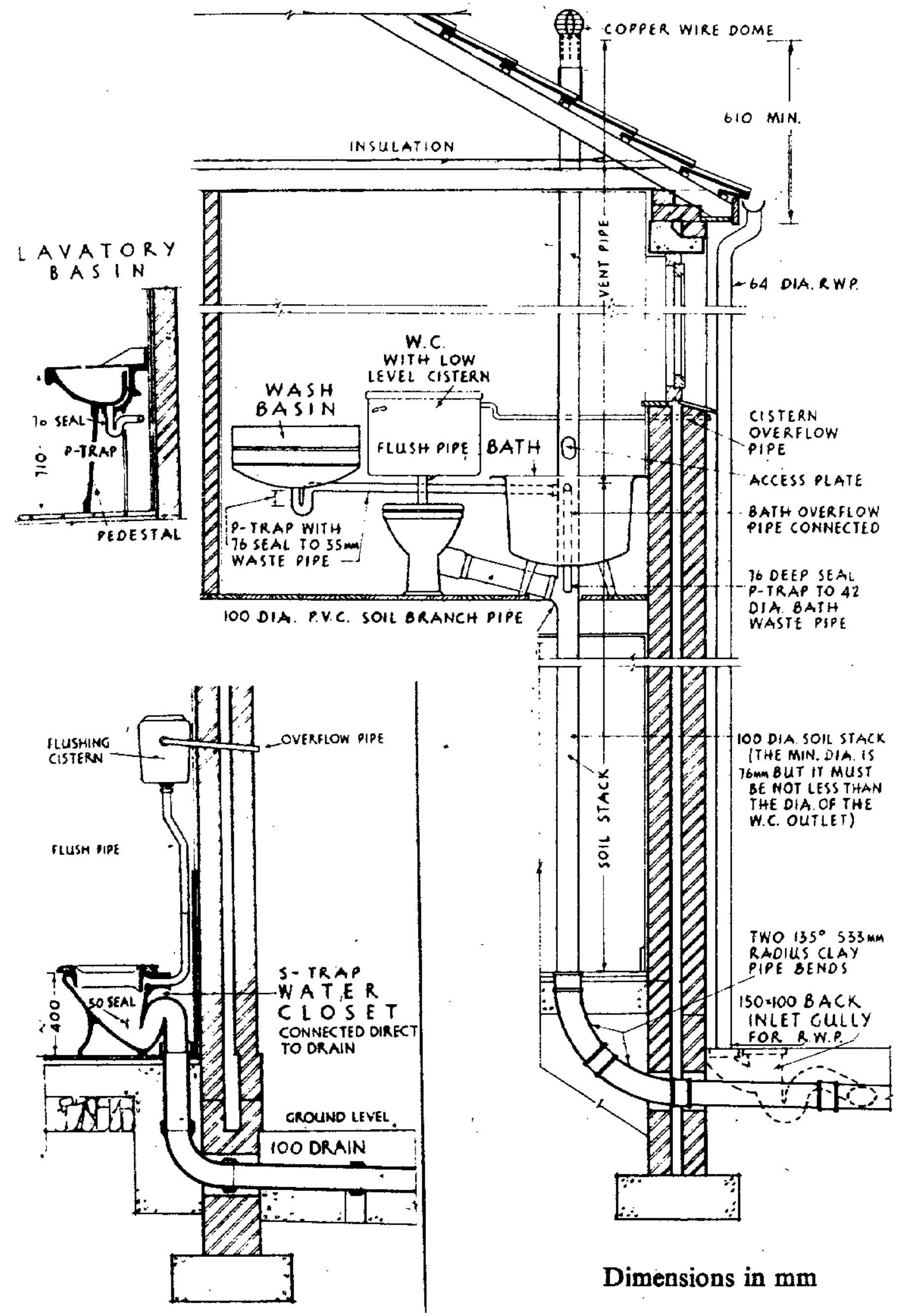
COLLAR JOINT (SEMI-FLEXIBLE TYPE)





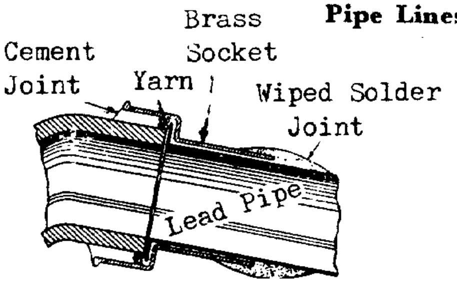
INTERNAL FLUSH JOINT

EXTERNAL FLUSH JOINT

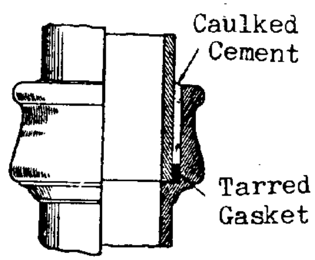


Dimensions in mm

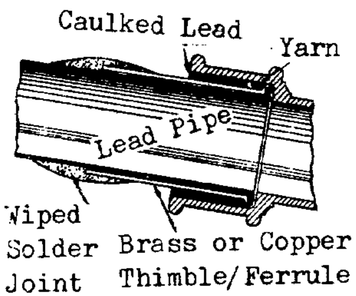
Pipe Lines and Jointing



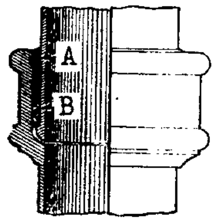
Joint Between the Out-go of a WC Trap & a Lead Pipe



Asbestos Cement Pipe

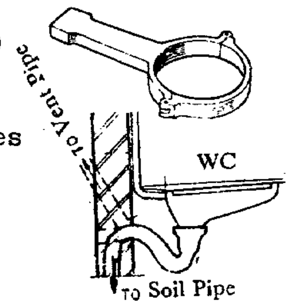


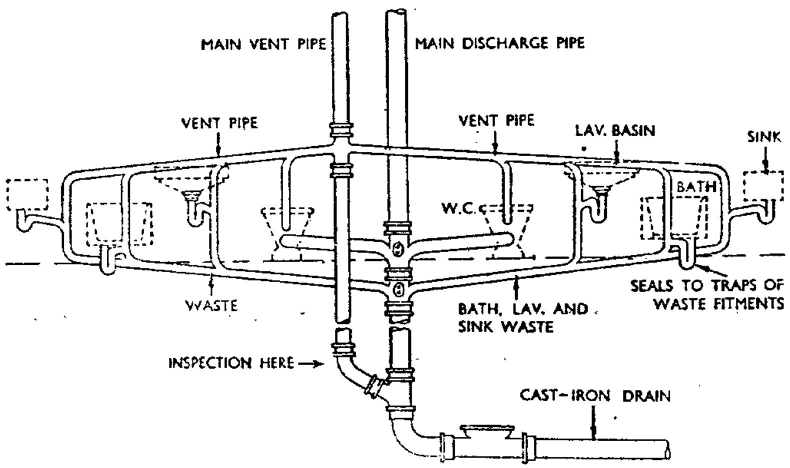
Jointing Lead Pipe to Cast Iron Pipe



Cast-iron Pipes A-Lead, B-Yarn

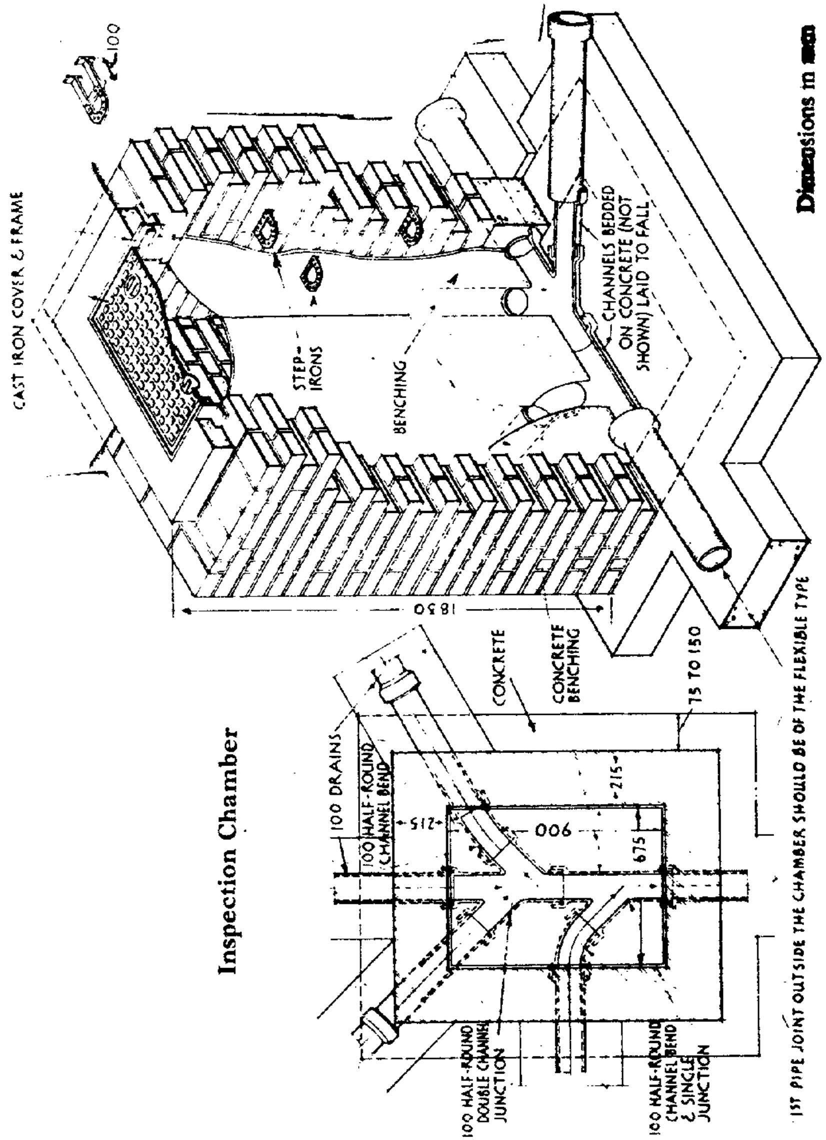
Holder-bat for Cast-iron Pipe



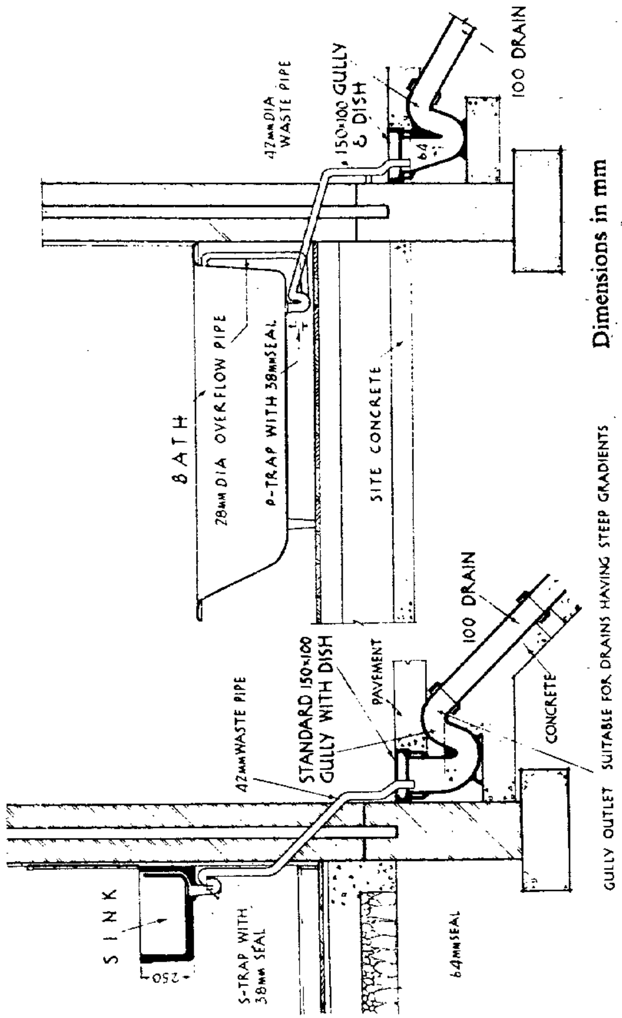


LAYOUT OF A ONE-PIPE SYSTEM

Dimensions in mm



Inspection Chamber



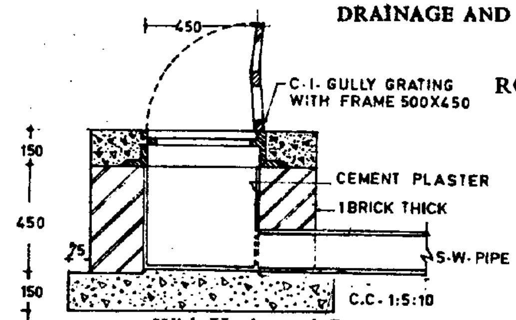
GULLY OUTLET SUITABLE FOR DRAINS HAVING STEEP GRADIENTS

Dimensions in mm

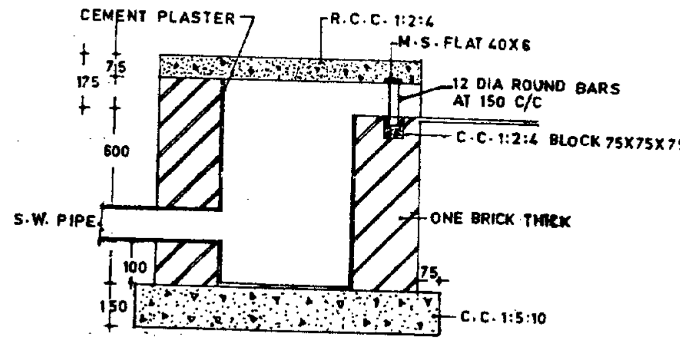
DRAINAGE AND SEWERAGE

ROAD GULLY CHAMBER

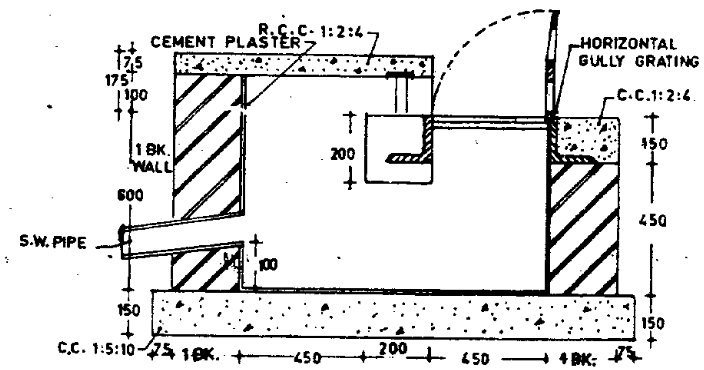
Dimensions in mm



With Horizontal Grating



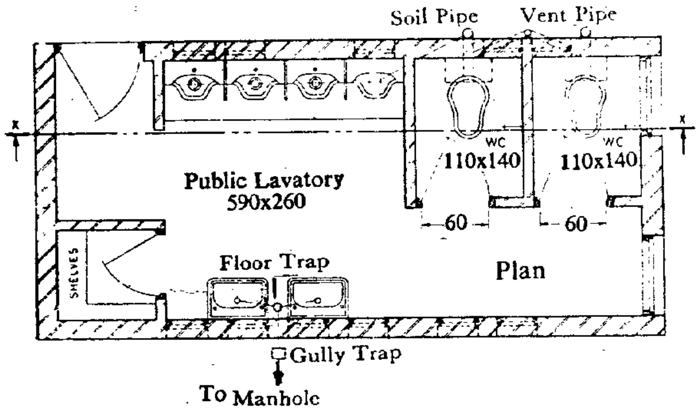
With Vertical Grating

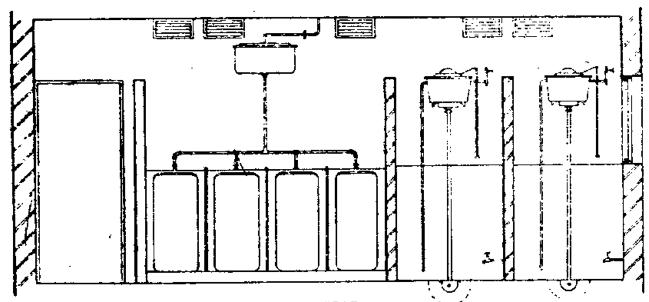


With Hortzontal & Vertical Grating.

CPWD

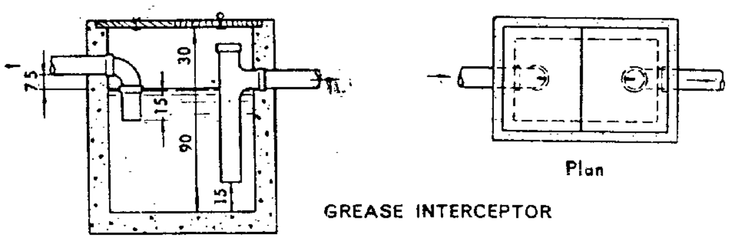
TYPICAL ARRANGEMENT IN PUBLIC LAVATORY





Based on IS:2064 Section—XX

Dimensions in cm



GREASE INTERCEPTOR

some such other arrangements, and it percolates slowly through the filtering media reaching the floor wherefrom it is collected by drains provided for the purpose and led out. This gives an opportunity for the organic matter to be oxidized by biochemical agencies. The filters may be made circular or rectangular, circular are preferred for small sizes. The depth is 1.2 m to 3 m and the size usually recommended is 400 litres/cu. m/day, with average sewage (and 200 to 230 litres with very strong sewage) and 3 m depth using 50 mm to 100 mm stones or bats, or an average of 800 litres/sq. m/day, with 1.8 m depth and 50 mm size of stones. With strong sewage the quantity is reduced to about half and with weak sewage using fine size filters, increased to about 2 to 3 times of what has been recommended above for average sewage.

Various devices in the form of spray nozzles, either fixed or rotating, are employed for the distribution of tank effluent over the filters. A simple device for small works is to make long troughs in the shape of W by bending iron sheets, with notches cut into the ridges. These troughs are placed in parallel rows about 15 cm above the surface of the filter on small masonry pillars, and when the sewage flows along the gutters it overflows in small quantities through the notches in the ridges. Pipes can also be used instead of troughs with holes drilled towards the bottom. Multiple jets or outlets are more efficient than single outlets in getting better dispersion and less sick area.

A thin transparent film upon the stones indicates favourable condition. When a filter is overdosed, the gelatinous film formed over the surface becomes too thick which prevents air penetrating into the lower layers (free circulation of air through the filters is very essential), clogs the bed and reduces the rate of flow. This can be remedied by a period of rest or intermittent distribution ; raking the bed surface is also beneficial. A 20 per cent solution of caustic soda, or in some cases a strong solution of copper sulphate has been found effective.

Some engineers consider that the efficiency is much increased if there is alternate dosing and aeration, i.e., 15 minutes rest, and for which a dosing tank is provided with a capacity equal to 30 minutes' flow of sewage and with a simple mechanism for its working. This tank is built in between the filter bed and the septic tank (or sedimentation tank). The filter bed should be provided in duplicate.

A cement concrete floor is provided with open jointed half-round under-drains, 100 mm size and about 45 to 90 cm apart, laid radially from the centre with suitable falls (say 1 in 100), led to an external peripheral channel, for collecting and disposing off the effluent. Unglazed earthen clay tiles are also commonly used.

Simple Filters for Small Installations. Make a circular platform of concrete, cement rendered, laid 75 mm higher at the centre than at the circumference, surrounded by a drain to carry off the filtered effluent. A 1.5 m high pillar is built in the centre of the platform the top of which is corbelled out to form a basin 60 cm diameter, the lip of which must be exactly level all round. The treated effluent (from a septic tank) is led into this basin so as to spill over (all round) the level edge of the basin. The filtering material is piled up around this pillar with coarser materials at the bottom and finer materials at the top. At the bottom 75 mm two layers of dry bricks or stones may be laid with 20 mm spaces betweeu them. Quantity of filtering material required is about 1 cu. m per ten users of the septic tanks where the filtered effluent is to be used for irrigation or is to be discharged in a stream. The height of the filter will depend upon the head available, but 1.5 m is considered to be the ideal. Where more clear effluent is desired, increase the quantity of filtering material or refilter.

Intermittent Sand Filters. Sand is used as filtering media. These filters are very efficient but require large area, therefore, are not suitable for treatment of town sewage, but can be installed for infectious diseases hospitals or where a highly treated effluent is desirable, or as a finishing treatment to other secondary methods. Beds are usually made rectangular ; at least 3, and preferably 4, beds are required for one installation. Bed depth is 45 cm to 75 cm. Loading rate is 100 litres to 250 litres/sq. m/day for preliminary treated sewage, 50 litres/sq. m/day for raw sewage and 500 litres/sq. m/day for finishing treatment. Single dose per day per bed is given, this gives a resting period of 2 or 3 days to each bed. The sand used has an effective size between 0.20 to 0.50 mm. Open jointed pipe drains are laid in trenches under the sand filters for collecting the filtered effluent. These pipes are surrounded with gravel or crushed stone of size 25 to 50 mm followed by smaller size up to about 6 mm in 7.5 cm layers, to prevent sand sifting into the pipes. The bottom of the sand bed is sloped gently towards the underdrains.

A dosing tank of masonry is constructed on one side of the sand beds which are arranged in series, and has a capacity equal to the desired single dose for a bed. Sewage is applied through troughs or perforated pipes placed in such a manner that the flow is distributed evenly over the whole area of the sand bed in successive doses. When clogging occurs, the surface should be raked and bed given more rest. If a layer of solids collects upon the sand, it should be scraped or swept off.

Humus Tanks or Secondary Setting Tanks are installed when a high degree of purification of the effluent from filters is required for discharge into rivers or streams. The effluent from trickling filters has a large amount of light flocculent humus like matter which is removed by settlement in the humus tanks. This humus, although inert, cannot be allowed to pass into a water-course, as in due time it would produce mud in large quantities and seriously impede the course of such a stream.

These tanks are similar to plain sedimentation tanks, and may be hopper bottomed or flat bottomed ; are provided next below the filters, (or primary filters where double filtration is adopted). The capacity is usually about one-sixth of the dry weather flow. The humus is collected and dried on sludge beds. Where filter effluent is finally disposed of on land, the provision of a humus tank is unnecessary.

Nature's Methods for Purification of Sewage. Sewage is a very complex material containing highly putrefactive matter both in a solid and a liquid form.

Purification of sewage is effected by living micro-organisms which are

always present in the human excreta. They break up all harmful complex organic substance of sewage into harmless simple compounds. There are three varieties of these organisms :—

(*i*) *Anaerobic Bacteria* which exist without oxygen and thrive in absence of light and bring about decomposition of waste matter, the breaking down of the solids, and eventually reduce the organic matter into liquid form accompanied by an inert form, which is the first stage towards purification. This liquid is devoid of oxygen and gives off foul gases. They are found in septic tanks.

(*ii*) *Aerobic Bacteria* which require oxygen for their existence, thrive in presence of light and bring about oxidation of the sewage ; come into play in sewage filters, contact beds and activated sludge process, which is the second stage towards purification.

(*iii*) *Facultative Bacteria* which exist with or without oxygen, thrive in abundance in absence of air and come into play in covered tanks just like anaerobic varieties.

Activated Sludge Process. Is a system of sewage treatment in which certain amount of oxidized or activated sludge is intimately mixed with the sewage which greatly hastens the process of oxidation of organic matter. The space required in this process is much smaller than required by the trickling filters. It is usual to submit sewage to preliminary treatment before activation. The quantity of activated sludge to be mixed varies from 5 per cent to 20 per cent according to the nature of the sewage to be treated and it takes 1 to 3 weeks to establish normal operating conditions in the plant. The period of detention is about 4 to 6 hours for a diluted fresh domestic sewage, which may be 8 to 10 hours for a strong sewage and sewage containing industrial wastes. Tanks are usually rectangular in plan with equal to about 1 1/2 to 2 times the depth which is about 3 m in small plants and 4.5 m in large plants. There are two methods of aeration and activation : (i) Compressed air is introduced through porous slabs or "diffusers" into the sewage as it flows through tanks. Air is supplied through vertical pipes which run longitudinally along the tank. (ii) Mechanical aeration, in which stirrers are employed to aerate the sewage and to keep the tank contents in circulation.

Trade Wastes or Industrial Wastes. Are the waste waters produced by manufacturing processes which are of no commercial value. If these wastes are of such composition as would damage the sewers or interfere with the treatment processes, then pretreatment of these wastes is essential before discharge into the main sewers or treatment plants. Provision of screens for removing suspended solids and sedimentation tanks or chemical precipitation tanks will be adequate for most of the trade wastes.

Pumping Stations. Certain considerations are essential for the location of pumping stations. Long lengths of pumping mains (or rising mains) are undesirable as they result in septic sewage, the pumping station site should be chosen to keep this length to a minimum. The pumping station should however, be located as distant as possible from residential quarters. Pumping operations may be concentrated to one point or may be distributed to a number of strategic points of a drainage system. Sewage pumping mains should not discharge directly into a sewer but into a manhole or a chamber of suitable size which is ventilated by a shaft or other means. Overflows at or near the tail end of the sewer system are absolutely necessary to deal with inadequate pump necessasity or power failure.

Pumping Sewage. Pumps have been described in detail under "Water Supply".

There are special types of pumps suited for lifting sewage and the pumps most commonly used are the centrifugal pumps, which should be of the open impeller or bladeless impeller type to avoid chokeage. The pumps must be "full-way" able to pass a ball of 75 mm diameter without chocking. Such pumps are comparatively low efficiency pumps but do away with the necessity of installing screens. It is considered better to screen the sewage and pass it through ordinary pump of high efficiency.

All sewage should, however, be screened and passed through grit chambers to remove suspended solids before allowing through pumps. Propeller-type pump may be used for dealing with storm-water flow. The mixed-flow-pump should be adopted where the volume of sewage to be pumped is large and where the minimum head exceeds about 12 m. The axial-flow or propeller-type pump should be used where a large volume of sewage has to be pumped against a low head.

Vertical-spindle centrifugal pump should be preferred. This pump has the advantage of permitting the motor to be placed above the pump pit to safeguard against damage by flooding. The space occupied by this set is less than that occupied by a horizontal set of similar capacity, and the vertical shaft can be made of any length. Direct-coupled vertical pump is the most economical. The horizontal direct-coupled pump set may be preferable when the depth of the installation is 4.5 m or less. This set is more easily maintained. Except where current is not available, starting should be automatic by float, pneumatic or electric control.

Reciprocating pumps are less used for pumping sewage as they are susceptible to choking and fine screening of the sewage is necessary. They are particularly suitable for dealing with thick sludges or for pumping against variable heads. Reciprocating pumps, where used, should preferably be of the plunger type. Turbine pumps are not suitable, and also not rotary.

Number and capacity of pumps. (i) For small installations, there should be two sets each capable of handling an average daily sewage flow in six to eight hours. (ii) For medium installations, a minimum of three sets would be necessary, one set—average daily flow, one set—twice average daily flow, and one set capable of handling three times average daily flow. Some engineers recommend that all the three sets should be of equal capacity, each capable of dealing with half the maximum rate of flow, so that frequent cutting in and out is avoided.

The economic diameter of the rising main which depends on the velocity at the normal rate of pumping, not the peak rate, should not be less than 100 mm and the velocity of flow not less than 75 cm and not more than 180 cm per sec.

Pneumatic Ejectors are installed where small quantities of unscreened sewage are to be lifted to a high level in isolated locations and the installation of a pump would not be justified. They are very simple, reliable, easy to maintain, and convenient in working having no parts likely to be clogged, require very little attention, are silent in action, but their efficiency is not very high and are useful only for small jobs. Ejectors work on compressed air and several types are available in the market based on the same principles of construction. Usual capacity is 1 to 5 cu. m per minute, with lifts of about 6 m. "Shones" Pneumatic Ejector is a well known make generally used. They should be provided in duplicate.

"Wet wells" should be covered, and capacity at small installations may be equal to one hour's average flow. In larger installations the detention period in the wet well should be limited to from 15 to 20 minutes. The average sewage depth should be 1.5 to 1.8 m and the bed of sump sloped towards the suction side.

Valves on Pumping Mains should be located as follows :—

(*a*) *Reflux valves.* Immediately above the pump to reduce back surge and water hammer, and should be placed on the horizontal portion of the main.

(*b*) *Sluice valves.* As an isolated valve at pump (above reflux valve to enable this to be readily isolated in the event of its requiring attention), also at points on the main where required to isolate sections.

(*c*) *Air Rolease valves.* At summits of mains.

Methods of Sewage Disposal

By Dilution in river or sea. Flowing fresh waters are a natural source of purification for the presence of oxygen in them. River water is better than sea water. Sewage must be quickly and thoroughly mixed in the diluting water which should be adequate in depth, sufficient in quantity and strong in forward current at the outfall so as to prevent the deposition of solids and their decomposition. Sewage should be thrown in sea water only when helpful tidal currents are present. A dilution exceeding 500 parts of water is considered sufficient for crude sewage but it is always preferable to pass it through screening chambers and settling tanks with capacity of about 2 hours flow, or subject it to some form of primary treatment so that the solids which would otherwise float on the surface or form deposits at its bed are removed or reduced. Sewage should be discharged under the surface of the water, a depth of about 3 m minimum is considered necessary depending upon the quantity of the sewage, its condition and the strength of the current of the water. Site of the outfall is also an essential consideration keeping in view if the river water is utilized for drinking or other domestic purposes.

Treatment of Sewage on Land. The clarified effluent from sedimentation tanks can be effectively treated on land economically where cheap and suitable land is available. There are two methods of treating sewage on land, depending upon the physical nature of the land available :

(*i*) *Filtration* : by which the sewage filters down through the land where the soil is light and porous ;

(*ii*) *Broad irrigation* : which is applied on a non-porous soil ; the sewage is run over the surface and collected again in ditches or drains for retreatment on a fresh plot.

In the filtration method the settled sewage is irrigated into trenches and the treated effluent collected in underdrains which run parallel to and between the trenches.

The quantity of effluent that can be dealt with by a certain area of land depends upon the character of the soil, sub soil water level, weather, and the strength and nature of the sewage to be treated. Best soils are sandy loams overlying a dry gravel or other porous sub-soil, about 1.5 m deep in all. The worst soil is stiff clay. Black cotton soil or heavy red or yellow soil is unsuitable ; peat is almost useless. Pure sand is also not favourable as the sewage will pass through very quickly leaving a colloidal slime at the surface thus clogging the pores. Irrigation or land filtration with raw crude sewage is not practicable as the solids choke the surface and prevent the oxidation of the liquid which is necessary for a good effluent. As a minimum treatment, a dilution of two parts fresh water to one part sewage is recommended. When dilution water is not available, the conventional treatment of screening and settling is essential.

When the sewage, after previous sedimentation, is applied to land filtration, the quantity in cubic metres per hectare per day generally varies from 45 to 330 that can be absorbed by a land depending upon the porosity of the soil, strength of the sewage and other conditions. Some engineers base their estimate on an average of 1 hectare of land per 250 persons when the sewage has been diluted with twice its own volume of fresh water. Good crops can be grown on such lands. Surplus land up to 25 per cent is usually required for rest. When the sewage is applied to land by broad irrigation, about three to four times more land will be required, with surplus land up to about 25 to 50 per cent.

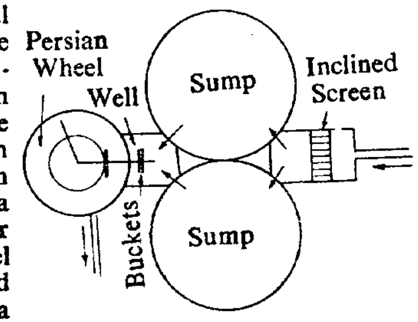
Sewage should not be applied in greater quantities than can be absorbed in about half an hour. Purification takes place due to the action of aerobic bacteria in the porous soil. The sewage should be applied intermittently on the land which should be divided into 3 or 4 parts, each receiving sewage for a day or two and allowed to rest double this period. The surface should be loosened from time to time. If the soil does not readily absorb the sewage, aid of irrigation is taken. The land should be levelled with a gentle slope towards one side and the distribution of sewage should be as uniform as possible.

Where sewage is passed on to a land for irrigation, about 50 to 100 mm of sewage can be applied at a time on a porous soil at 8 or 10 days intervals. The common method is by means of furrows opening out from a header ditch. The crops are grown on the ridges between furrows. Another method is to apply the sewage on a plot of land about 30 cm deep and allowed to percolate into it and the flow directed to another plot. Sometimes cross drains are provided at the lower edge to catch the surplus water for re-treatment. Resting period is allowed between each application. The main disadvantage of the treatment of sewage on land is that it cannot be applied for all the twelve months of the year at the same rate. During rains and harvesting periods no land or less land is available.

Absorption Trenches. Open jointed pipe drains, about 100 mm dia. or more are laid to a gradient of about 1 in 300 to 500 in a trench and filled all round with broken stone, gravel or brick-bats of size 12 to 63 mm, which should surround the pipes with a minimum of 10 to 15 cm at the bottom and 5 cm at the top. The pipes may be laid to a gap of 3 to 6 mm between them and the top of each joint covered with half tile pieces or with aggregate of bigger size than the gap. The width and depth of the trenches is governed by the absorptive capacity of the soil increasing with decrease in percolation rate. The usual dimensions will vary from 45 to 90 cm width at the bottom and 45 to 120 cm depth. The trenches should be laid at a distance apart equivalent to three times the width of a trench with 1.8 m minimum. The trenches are filled with the excavated material above the broken stone over the pipes. The effluent can be distributed to the various trench drains through a distribution pit made at the end of the outfall. The above method applies to small flows. Where large flows from towns are to be handled in this manner, and which will be necessary where land irrigation is not possible for the soil being poor, the trenches should be dug about 15 to 30 m apart and *kachha* side drains (or open trenches) made at right angles to the main drains, about 1.8 to 3 m apart.

Raw sewage or effluents from septic tanks or settling tanks should not be used for irrigating growing crops or vegetables.

A sketch is given showing typical arrangements for the disposal of sullage water from a small town by land irrigation. The sullage is screened through a screen built in a small chamber. The water is collected into sumps made in duplicate to hold one day's flow. From these sumps the water is passed on to a small well where from it can be either pumped or taken out by a Persian wheel and the effluent disposed of by broad irrigation. The sumps are used one at a time and the water is made to deposit the solids, which are cleaned by hand. The disposal works are generally made to deal with only the sullage flow and not the storm water flow.



The sumps may be made rectangular or circular, depth 1.8 m min. to 3 to 4.6 m max. If rectangular, length to be 3 to 5 times the width. This is to help settlement of the solid matter. Silt and sand carried down the drains should be deposited in a catch pit and any floating solids should be screened out. It is sometimes preferable to make a grit chamber in between the screen chamber and the sumps for the sullage-water to deposit its silt and sand instead of depositing the same in the sumps. It is considered that the amount of sand and grit is about 0.3 to 0.6 cu. m per million litres, and the size of the grit chamber should be made accordingly giving about 2 hours detention, and built in duplicate.

After continuous application of sewage on land or ballast (in filters) the pores often get clogged preventing oxidation. This is called *sewage sickness*. The remedy is to break up the surface of the land and give it rest for some days; in the case of ballast, it is all washed up with fresh water and re-used.

9. SUB-SOIL DRAINAGE

(Sub-soil Drainage has also been described under "Roads and Highways" at page 18/21.)

When the subsoil water is within 3 m of the surface of the ground it very often becomes essential to drain out this water for successful building operations or for sanitary reasons. Open jointed drains are generally laid below ground level, which are of the size that will permit draining out all the water. Min. size is 50 mm. The drains are laid in the direction of the greatest slope, which ensures the greatest velocity and capacity. High velocities of water assist in scouring out the drains and keeping them clean. It is very important that drains in sandy land should have velocities great enough to scour out the sand. The drains should be laid as straight as practicable.

The depth and frequency of the drains that will produce most satisfactory results depends largely upon the character of the soil. The more porous the soil the deeper the drains can be placed, and deep drains are placed further apart. A depth of 0.61 to 1.22 m is generally suitable for clay soils and 1.22 m for sandy soils. Distance apart in clay soils varies from 6 to 7.6 m or even 9 m; in strong loams 9 m apart and in light soils 12 m. In sand, a spacing of 60 m may be used. Clay is a soil which is very retentive of moisture and one cu. m of most dry clay will absorb about 160 litres of water.

The sub-soil drainage system should be graded with a fall of 1 in 100 to 1 in 200 and the outfall properly arranged.

Before the trench is filled the drains should be surrounded by coarse hay, twigs, small stones or pieces of brick. This provides for a free entrance of water and helps to exclude fine sand from the drain. The trench is wider at the top, tapering inside towards the bottom. Sub-soil drains can also be made by filling the trench with large size of stones at the bottom and smaller stones or brick-bats above, giving full gradient for the flow, thus doing away with pipe drains.

10. PREPARATION OF DRAINAGE SCHEMES

A layout plan of the whole area should be drawn to a scale of 1/2500 or smaller according to the size of the proposed scheme. The smallest scale to which a useful plan for a drainage project can be drawn is 1/4000. It is generally convenient to split up the area into three or four parts or blocks, or high level and low level zones. The scale to be adopted for drawings should not be less than 1/500 for a group of buildings and 1/200 for a single building. The block plan should indicate clearly and accurately: (*i*) The whole of the site within which the buildings are to be erected with existing buildings and drains upon it. (*ii*) Ground levels and the lowest floor level in each separate building. (*iii*) The size, invert levels and direction of flow of the existing drains or sewers into which drainage is to be taken.

Levelling. Levels should be taken at the following intervals:—

(*a*) 60 m along the centres of all roads and at summits and lowest points. (*b*) 30 m on the beds of all pucca drains and 60 m for *kachha* drains, or of the proposed drainage lines. (*c*) At every 120 m (should the ground allow of it) readings should be taken at 60 m and 120 m to the right and left of the line of section and at right angles to it.

Levels should be observed of the following places and their positions surveyed:

(*a*) If the ground level at the road-side differs by more than 23 cm from the level of the centre of the road, levels of the ground on both sides should be taken. (*b*) Floor of every culvert, width and height of the opening, the road or formation level, etc. (*c*) Cross sections of all drains. (*d*) Levels of plinths and courtyards of houses. (*e*) All flood levels, if any.

Contour lines should be drawn on the general plan (of 1/400), which should be not less than 12 mm, nor more than 75 mm apart.

Longitudinal sections should be drawn drain by drain and not road by road. The horizontal scale should be the same to which general layout plan is drawn (or prefer 1 to 1200) and vertical scale 10 times that of the longitudinal scale, except in a hilly country where a smaller scale may be necessary.

Detailed drawings to a scale of not less than 1 to 50 should be provided for each type of manhole to be constructed. Such drawings should indicate the shape and arrangement of channels and benchings, spacing of step irons, type of cover, etc.

For a new scheme possible developments for the next 30 years are usually taken and sizes of street sewers and disposal works designed accordingly. Rate of water supply and its likely increase affecting the size of the sewers and other building and pavement developments due to increase of population should be considered.

Writing a Report for a Drainage Scheme

(See also Section 20 "Estmating")

Deal with the following items:—

1. Situation of the town; its present population and the expected rise in the next 30 years.
2. Present condition of existing drainage and its disposal.
3. Position regarding water supply.
4. Quantity of expected sewage or sullage and storm water; rainfall.
5. General level conditions of the whole town and its effects on the design of sewers as regards self-cleansing velocities. Main and branch sewers. Disposal works and out-falls. Any pumping required; power for working pumps.
6. Design of sewers; material for sewers.
7. Annual maintenance and working expenses of the scheme. For *Annual Maintenance* 1 per cent is generally provided on the initial outlay.

8. Depreciation of machinery and buildings.

9. Revenue from effluent sold for irrigation. Savings of sweepers and disposal of town refuge of the existing arrangements.

11(A). GLOSSARY OF DRAINAGE TERMS

Activated Sludge : Sludge settled out of sewage previously treated in the presence of oxygen.

Baffles : Deflectors of wood, metal, or masonry placed in flowing liquid to divert, guide or guide and agitate the flow of such liquid.

Baffle Wall : See under Scum Board.

Barn Sewage : Wash water from stables containing considerable quantities of animal waste.

Barrel : That portion of a pipe throughout which the diameter and wall thickness remain uniform.

Bedding : A layer of concrete on the trench floor to provide simple support for the pipe. Page 16/19.

Benching : The sloped floor of a manhole on both sides and above the top of a channel, on which a man can stand for cleaning the sewers. Page 16/30.

B.O.D. : Biochemical Oxygen Demand—A test indicating the loss of oxygen in the process of decomposition of sewage.

Broad Irrigation : The disposal of sewage by application to farm land; benefit to crops growing there is only incidental. Differs from *sewage farming* in which the primary purpose is the disposal of sewage and the raising of crops is only incidental.

Centrifuge : A device in which sludge is dewatered by rapid rotation and automatically discharged.

Cesspool : A pit with open joints towards the bottom in which the effluent from a septic tank or other household liquid waste is discharged and from which the liquid leaches into the surrounding soil or is otherwise removed.

The correct word for a pit doing the functions of a cesspool is "seepage pit" through which the effluent seeps into the surrounding soil, and into which the treated effluent is collected. A pit into which raw sewage is collected (and which is used as a substitute for a septic tank) is really a cesspool.

Clarification : The process of removing suspended and colloidal matter from a (turbid) liquid or sewage.

Clarified Sewage : Loosely used for sewage from which suspended matter has been partly or completely removed.

Cleaning Eye : An access opening having a removable cover to enable removal of deposits obstructing flow.

Colloids : Finally divided suspended matter which will not settle and the apparently dissolved material which may be transformed into suspended matter by contact with solid surface or precipitated by chemical treatment.

Conservancy : The system of collecting night soil, in pots or pits for periodical removal outside the town area for burial.

Crude Sewage : Sewage that has received no purification treatment.

Deep Manhole : A manhole of such depth that an access shaft is required in addition to the working chamber.

Diffuser : A porous plate or other device through which air is forced and enters the sewage in the form of minute bubbles.

Digestion : The biochemical decomposition of organic matter in a sewage.

Dispersion Trench : A trench in which open jointed pipes, surrounded by coarse aggregate media and overlaid by fine aggregate, are laid. The effluent gets dispersed through the open joints and is absorbed in the neighbouring soil.

Ditch : A trench dug out in the ground for the purpose of receiving and conducting drainage water.

Drop Connection : A branch drain of which the last length of piping of the incoming drain before connection to the sewer is vertical. Page 16/28.

Drop Manhole : A manhole incorporating a vertical drop for the purpose of connecting a sewer or drain at a higher lever to one at a lower level. Page 16/28.

Dry-Weather Flow : Total average discharge of sanitary sewage and is the normal flow in a sewer during the dry weather.

Duck-foot bend or Rest bend : A bend supported in a vertical position by a foot formed at its base.

Effluent : Partly or completely treated sewage flowing out of a sewage treatment tank or a plant.

Free Water : The water that moves in a soil under the force of gravity without being retained by the soil.

French Drain : A small shallow trench filled with coarse rubble, clinker or similar material. (See under "Roads".)

Gully or Gulley : A receptacle of stone-ware, concrete, cast iron, or other material provided with a grid, placed at the side of a road (or a building) to receive drainage from a gutter or channel. Pages 16/37, 69.

(The term is also used for the erosion formed on road shoulders by rain water.)

Gully Trap : A trap provided in a drainage system with a water seal, to collect waste water from the scullery, kitchen sink, wash basins, baths, and rain-water pipes. Page 16/37.

Gutter : An open drain constructed along the sides of a carriageway to carry away the water drained from the surface of the pavement.

Haunching : Concrete bedding with additional concrete at the sides of the pipe.

Herring-bone Drains : A system of interconnected drains laid out in zig zag pattern, commonly used for subsoil drainage.

Hydro-Iso-baths : Contours of similar depth of sub-soil water-table below the ground surface.

Imhoff Tank : Is a deep two-storied tank invented by Karl Imhoff in Germany as an improvement over the septic tank. An Imhoff tank is similar to a septic tank except that the sedimentation and sludge digestion go on in different chambers. These tanks may be rectangular or circular in plan ; usually circular tanks are made for small flows. The upper compartment is known as the sedimentation chamber and the lower one, sludge digestion chamber. The sedimentation chamber is made comparatively shallow and its bottom or floor is given a steep slope of 50 to 60 degrees to the horizontal. The sewage enters the upper (sedimentation) chamber and the solids settle down and slide down the sloping walls (bottom or floor) into the lower chamber through the slots provided into the bottom of the upper chamber. The lower chamber receives no fresh sewage directly and the sludge collected into it undergoes digestion and decomposition. The slots in the bottom of the upper chamber are trapped to prevent escape of gases given off by the sludge in the process of decomposition into the upper (sedimentation) chamber. The lower chamber is provided with gas vents for the escape of these gases, and with means for drawing out digested sludge near the bottom.

Infiltration : The percolation flow of sub-soil water into a drain, sewer, or a water-bearing stratum.

Influent : Raw or partly treated sewage flowing into a sewage treatment tank or a plant.

Inspection Chamber : A water-tight chamber constructed in any house drainage system which takes wastes from gully traps and disposes off to manhole with access for inspection and maintenance. Pages 16/36, 67.

Interceptor Manhole or Interceptor Chamber : A manhole incorporating an intercepting trap, and providing means of an access thereto and equipped with a fresh air inlet on the upstream side of the trap. Pages 16/37, 27.

Intercepting Sewer or Interceptor : A sewer which receives its flow from a number of transverse sewers or outlets.

Invert : The lowest point of the interior of a sewer or drain at any cross-section.

Inverted Syphon : A portion of a pipe in which sewage flows under pressure, due to the sewer dropping below the hydraulic gradient and then rising again. Page 16/17.

Isohyet : Is a line on a rainfall map showing places having the same average annual rainfall.

Kerb Inlet : Aperture formed in a kerb to let in surface water from the pavement to a gully. Page 16/69.

Lateral Sewer : Is a street sewer into which sewage from house connection pours. Page 16/6.

Liquefaction : The results of the action of bacteria for the decomposition and purification of sewage or waste matter.

Night-soil : A mixture of human excreta, urine and personal cleaning materials.

Out-fall : Is an outlet of the main sewer or drain at the point of disposal.

Oxidation : Is the breaking down of the organic solids into stable organic or mineral compounds through biological activities in the presence of oxygen.

Putrefaction : The first stage of sewage purification by the action of anaerobic bacteria.

Rodding-eye : An access opening having a removable cover for the obstruction to be cleared by means of a drain rod.

Saddle : A purpose-made fitting so shaped as to fit over a hole cut in a sewer to form branch connections.

Scum : The greasy and other subtances which float at the surface of sewage in a sewage treatment tank.

Scum Board : Also scalled Baffle wall—Is a thin partition wall built in a sewage treatment tank to prevent the incoming sewage disturbing the scum. (See under Septic Tanks.)

Seepage : Percolation of water into or from the soil. (Seepage into a soil is termed Influent Seepage and that away from a soil as Effluent Seepage).

Seepage Pit : See under "Cesspool".

Sewage : Combination of liquid wastes conducted away from residences, public buildings or industrial establishments, with ground surface or storm water that may be connected with the pipes or drains leading to the sewers. Also called Domestic Sewage or Sanitary Sewage. See also "Sullage").

Sewers : Closed drains which carry the sewage (night soil) to a point of discharge or disposal.

Shallow Manhole : A manhole of such depth that access can be obtained to the chamber direct from ground level, without the need of an access chamber.

Sludge : The solid matter deposited at the bottom of a tank (during treatment of the sewage), which is in a semi-solid condition.

Sludge Digestion : The biochemical process by which organic matter in sludge is converted into more stable organic matter.

Soak Pit (Seepage Pit or Soakway) : A pit dug in permeable ground to which the soil water is led so as to leach into the surrounding soil.

Soffit or Crown : The highest portion of the interior of a sewer or drain.

Soil Sewage : The sewage from water-closets, slop sinks and urinals, and a mixture of this sewage with any other drainage or waste water. Page 16/4.

Soil Pipe : That receives the discharges from soil fitments, such as water closets, urinals, and slop sinks.

Soil Waste : The discharge from water closets, urinals, slop sinks, stable or cowshed gullies and similar other appliances.

Stack : Is a general term used for any vertical line of drainage.

Sullage : Waste water from bath-rooms, lavatory basins, kitchen sinks, street and roof washings, etc., which does not contain human excreta. Page 16/4.

Sub-soil Water : Water occuring naturally below the surface of water.

Sump (or Wet Well) : The underground portion of a sewage pumping station which receives the sewage and from which it is drawn by the pumps.

Surface Water : The run-off of natural water from the ground surface, including paved areas, roofs and unpaved land.

Sub-irrigation : Is the name given to the irrigation of settled effluent by means of pipes laid at shallow depths below the surface of the land.

Trunk Sewer : Is the main sewer into which all the smaller sewers discharge and which in turn discharges at the point of disposal.

Vent Pipe : A pipe line installed to provide flow of air to or from a drainage system or to provide circulation of air within such system to protect trap seals from siphonage and back flow.

Water Carriage System : Removal of sewage by a network of underground pipe lines or sewers. Another name for Sewage System not very commonly used now.

Water-shed : (*i*) The line of separation between adjacent catchment areas or drainage basins. (*ii*) The area drained by a stream or a stream system.

Water-table : The upper surface of zone of saturation in soil or permeable strata or beds. The upper surface of sub-soil water.

Waling : A longitudinal member supporting the sheeting in an excavation.

Waste Water : Same as Sullage.

11(B). GLOSSARY OF SANITARY INSTALLATIONS TERMS

Antisiphon Pipe : A ventilating pipe connected to or close to the outlet side of a trap seal. Page 16/37.

Automatic Flushing Cistern : A flushing cistern arranged to discharge its contents by siphonic action at regular intervals, determined by the rate at which water is fed into the cistern.

Ball Cock : A faucet opened or closed by the fall or rise of a ball floating on the surface of water.

Ball Valve : A simple non-return valve consisting of a ball resting on a cylindrical seat within a fluid passageway.

Bell Mouth : An expanded rounded entrance to an orifice.

Bib Tap : A tap with an horizontal inlet and a nozzle bent to discharge in a downward direction.

Box Union : A device for jointing together two threaded pipes.

Branch : The various types are called T, Y, T-Y, double Y, and V branches, according to their respective shapes.

Caulking :—

(*a*) The process of driving, pouring or forcing lead oakum plastic or other material into a joint to make it leakproof. (*b*) The material used in the caulking process.

Caulked Joint : A spigot and socket joint in which the jointing material is compacted by means of a caulking tool and hammer.

Chase : A continuous recess in a wall, floor or ceiling for the purpose of holding pipes and conduits.

Cistern : A fixed container for water. The water is usually supplied through a ball valve.

Collar : A pipe fitting in the form of a sleeve for jointing the spigot ends of two pipes in the same alignment.

Cowl : A hood on the top of a vent pipe or soil stack.

Float Valve : A valve in which the closure to an opening such as a plug or gate, is actuated by a float to control the flow into a tank.

Gasket : A piece of compressible materials, often perforated, used to make a joint between two flat surfaces.

Oakum : Hemp or old hemp rope soaked in oil to make it waterproof.

One Pipe System : In this, a single soil waste pipe conveys both soil and waste directly to the building drain. Page 16/33.

Single Stack System : This is the name given to a simplified one-pipe system wherein all ventilation pipes are omitted. The stack itself is made to cater (or provide) for all the vent requirements by restricting the flow into the stack to certain predetermined limits. Page 16/33.

Socket : The female part of a spigot and socket joint.

Soil Pipe : A pipe which conveys to a drain the discharge from water closets or urinals. In 'One pipe' and 'single stack' system the soil pipe also conveys to a drain the discharges from baths, wash basins, sinks and similar appliances.

Spigot : The male part of a spigot and socket joint.

Spigot and Socket Jaint : Joint in which the end of one pipe enters the enlarged end of the next pipe.

Stack : A main vertical discharge or ventilating pipe.

Trap : A fitting or device so designed and constructed as to provide, when properly vented, a liquid seal, which will prevent the back passage of air without materially affecting the flow of sewage or waste water through it. Page 16/36.

Two Pipe System : In this, the soil pipe conveys discharges from water closets, urinals, and similar soil appliances directly to the drainage system, and the waste pipe conveys waste from ablutionary and culinary appliances to the drainage system directly or through a trapped gully where desired. Pages 16/33, 66.

Union **:** A pipe fitting used for jointing the ends of two pipes neither of which can be turned.

Valve **:** A device used for controlling the flow of liquid in a line of pipe.

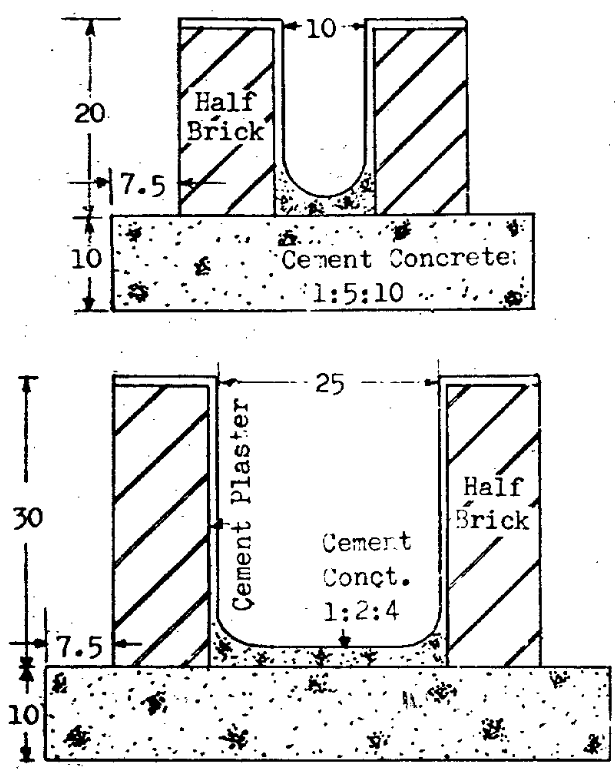
Ventilating Pipe (Vent Pipe) **:** The pipe which provides a safe outlet into the atmosphere for the foul gases in a drain or sewer.

Water Seal **:** The depth of water which should be removed from a fully charged trap before air can pass through the trap. Pages 16/36, 63.

Waste Pipe **:** A pipe used to convey liquid waste not containing human excreta.

Waste Stack **:** A vertical pipe used to convey liquid waste containing human excreta.

BRICK MASONRY OPEN SURFACE DRAINS



CPWD

Dimensions in Centimetres

(Type Designs of Drains are also given at pages 16/11,12, but both types are meant for different locations.)

Pour-Flush Water-seal Latrine *Low Cost Latrine For Rural Areas*

In India, a lack of sanitary toilet facilities is the biggest problem in most rural areas and urban slums.

The pour-flush waterseal latrine consists of a squatting pan with a steep bottom slope (25° to 30° to the horizontal) and a 20 mm waterseal P-shaped trap of 70 mm dia and 20 mm waterseal with foot-rests, set in a cement concrete floor. After use it is flushed by hand using a small container holding about 1.5 to 2 litres of water. The excreta are carried through a pipe or drain into two honeycomb leach pits, which are used alternately. The liquid in the pits percolates into the sub-soil and gases are absorbed by the soil, leaving the solids behind. Each pit is designed to last for about three years before it gets filled; when one is filled, it is taken out of use and excreta are then diverted to the second pit. When the filled pit is left for about two years, the contents turn into a rich organic humus which is safe for handling. When convenient it is emptied and the contents may be used as manure. It is then ready to be put back into use when the second pit fills up.

With simple care the pour-flush waterseal latrine is a very satisfactory and hygienic sanitation system. It can be located inside the house, since the waterseal prevents odour and flies nuisance. Spent water available from kitchen and bath keep the latrine clean.

Squatting Pan

Details of the squatting pan and trap are shown in the drawing. The horizontal length of the pan should be at least 425 mm. The pan can be of ceramic, fibre-glass reinforced plastic (GRP), PVC, mosaic or cement concrete. Ceramic or GRP pans have many advantages over the concrete ones. They are smooth, require less water for flushing and are more aesthetic. A GRP pan is cheaper, lighter and easier to transport than a ceramic one. The concrete pans are heavy, difficult to transport and get roughened and unattractive after use due to the action of uric acid, but initially they are less expensive. Fibre-glass pans are most commonly used.

Trap

P-shaped trap fixed in the floor of the lavatory should be as shown on the drawing. It should be 70 mm. dia with a 20 mm waterseal. Ceramic, GRP or PVC traps are smooth and need less water for flushing than concrete ones, but initially cost more.

The trap should be connected to the pits either by a pipe or a covered drain. If a pipe is used, a junction chamber of minimum size 250 mm x 250 mm internal should be provided at the junction point. The non-pressure AC pipe should be used as it is cheaper, and its size should not be less than 75 mm. The drain can be made of bricks or stones with a minimum size of 75 mm x 75 mm with semi-circular bottom. The slope provided should be 1 in 5 to 1 in 15. Bends and curves in the drain should be avoided. The inlet pipe of drain into the pits should project a minimum of 150 mm into the pits.

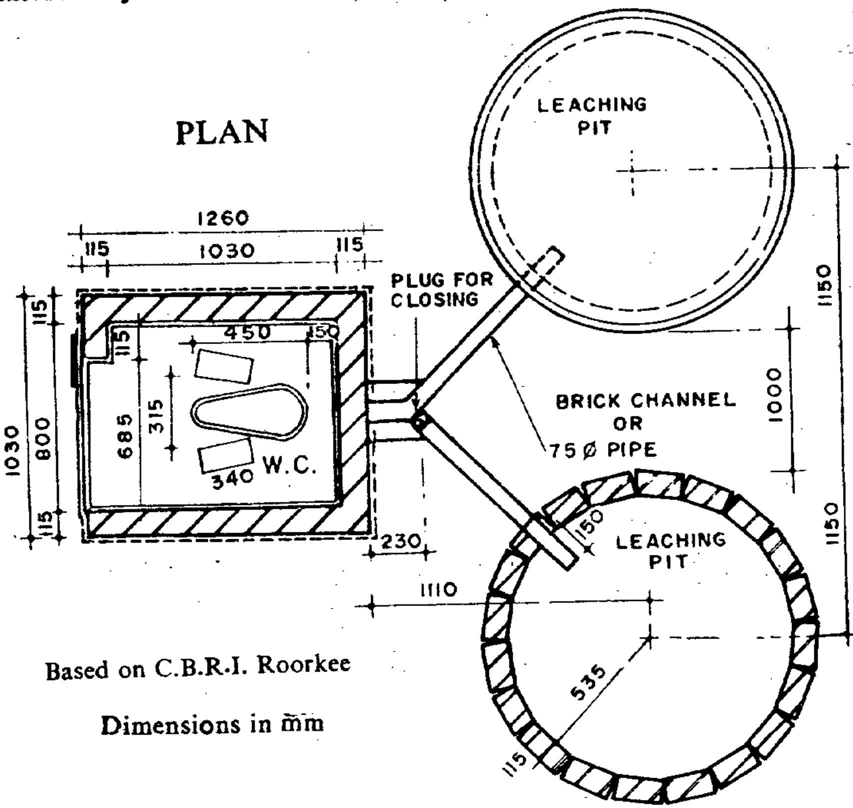
Volume of Leaching Pits

The size of the leach pits which are generally provided at the back of the squatting pan depends upon a number of factors such as : number of users, cleaning interval, soil properties (including its permeability), water table condition, and the quantity of water used for flushing and anal cleansing.

Effective volume is the volume of the pit below invert level of pipe or drain.

Non-pressure asbestos cement pipes can be used of 75 mm dia. with slope of 1 in 15 connecting pipe is projected into the leach pit by atleast 15 cm length.

A pit about 1 cu. metre capacity will serve 5 users for about 4 years in sandy soil and 6 years in clayey soils. The leaching pits can be made either circular or square. Optimum diameter and depth of a circular pit should be 1.07 m and 1.22 m; and width and depth of a square pit should be 0.92 m and 1.2 m respectively for 5 users. (circular pits are not, however, recommended). Two square pits can be built together with solid impervious wall (dividing wall) which should be extended by 30 cm below floor level and cement plastered on both sides.

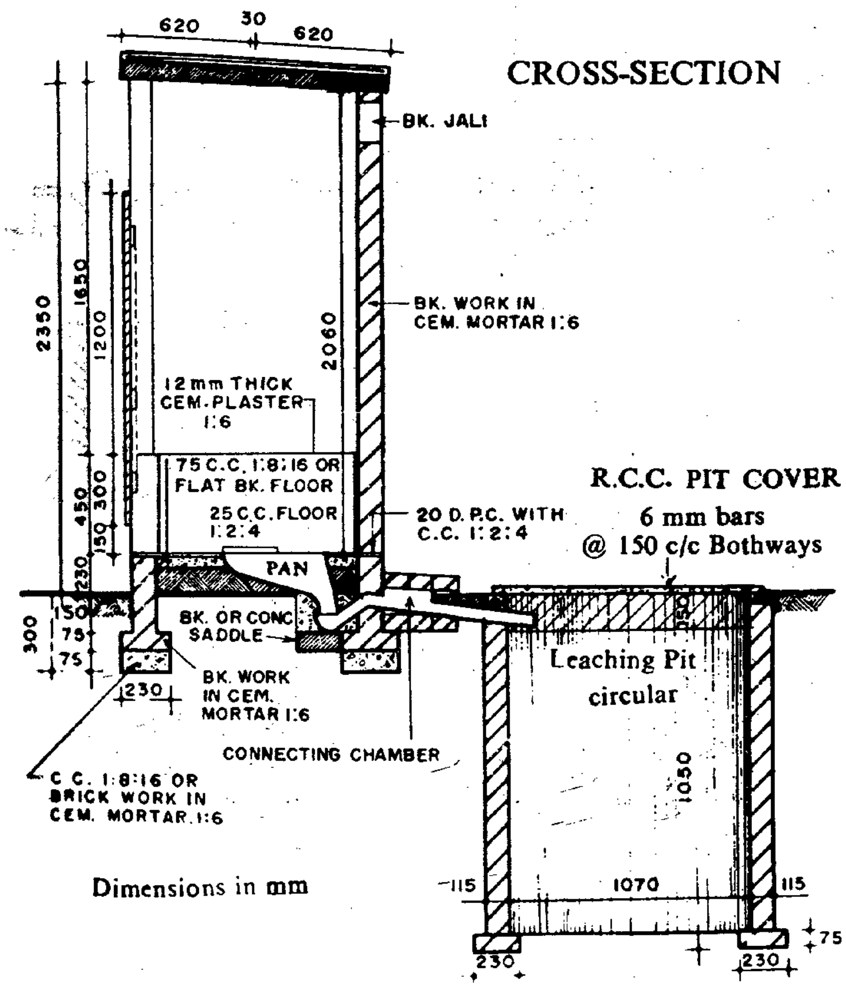


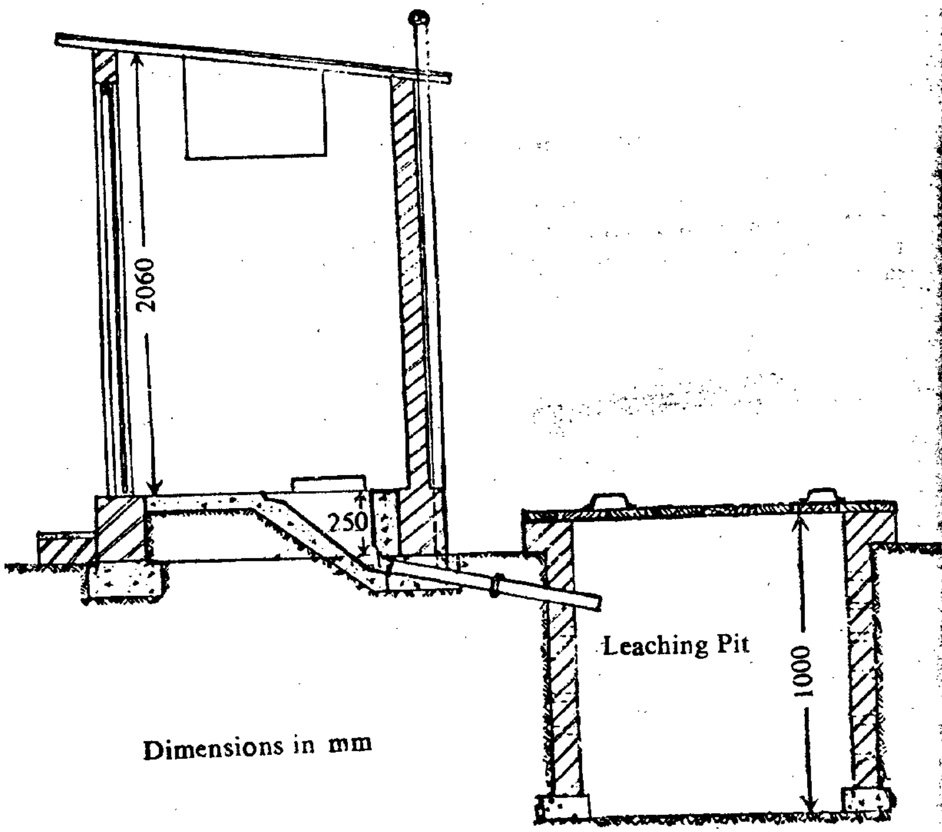
Based on C.B.R.I. Roorkee

Dimensions in mm

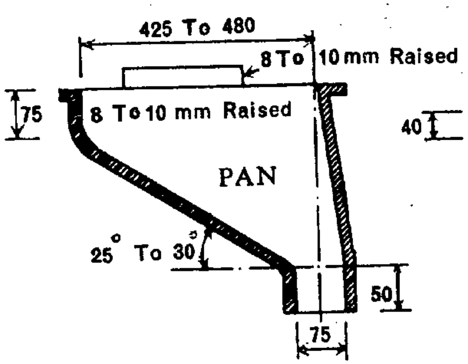
Walls of leaching pits should be half-brick thick solid and without plaster or pointing. The floor of leaching pit should, however be without lining except in the high sub-soil water table areas where it has to be impervious to reduce chances of pollution. Honeycomb brickwork is not recommended as the soil filled behind the walls will flow into the pit during rains.

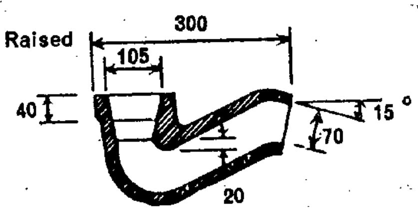
Leach pits when constructed in the area having high water table or depression around them, the height of the pit should be raised by 60 to 80 cm above the ground level together with the squatting pan and earth filling.





Dimensions in mm





Water Seal Trap

SECTION 17

IRRIGATION

1. GLOSSARY OF IRRIGATION TERMS

(Based generally on the Central Board of Irrigation & Power India.)

More terms are given under respective headings in the following pages, and also in Sections Hydraulics, Water Supply, Drainage & Sewerage, and Bridges.

Acre-foot—A unit of volume used in Irrigation practice. It means the volume of water required to cover an area of 1 acre to a depth of 1 foot. It is 43,560 cu. ft. or 1233 cumecs.

Actual Command—Is the area on which water will flow from a complete canal system as constructed or is likely to be constructed.

After-bay—The tail race of a water-power plant; a pond or reservoir at the outlet of the turbines.

Alluvial Soils—These soils are formed by materials like sand silt or clay, transported and deposited by flowing water or as a result of river floods in the course of time and are found in flood plains and flat country. Such soils are found in Bengal, Bihar, Orissa, Punjab and U.P.—in the Indus and Gangetic plains.

Non-alluvial Soils are formed as a result of disintegration of rocks carried over a long time. The area of a non-alluvial soil has usually an uneven topography and hard beds. There are no silt problems in the channels built through these soils. The major portion of Bombay district is an area of non-alluvial soils. In actual alluvial channels, the grains are not closely packed.

Annual Intensity—Is the percentage of the culturable irrigable (commanded) area irrigated during the year to the total C.C.A. on an outlet or a channel.

Aqueduct—Where the bed level of the canal is higher than the high flood level of the drainage, then the cross drainage works is called aqueduct.

Area Assessed—The area irrigated on which water rates are levied.

Arid—A term applied to lands or climate that lack sufficient water for agriculture, without artificial irrigation.

Apron—A floor or lining of concrete, stone, masonry etc., to protect a surface from erosion or to withstand hydrostatic pressure. Aprons are provided on the upstream side of crest walls, below chutes or spillways, at the toes of dams, at the entrance or outlet of a culvert or water-way, etc., to prevent scour.

Ayacut—is a Tamil name for culturable area.

Back Water Curve—A form of the surface curve of a stream of water which is concave upward. It is caused by an obstruction in the channel such as a weir or a regulator.

Baffle Wall—Is a sort of low weir or cross-wall constructed in a channel downstream of a cistern to head up water and dissipate energy.

Balancing Tank—Is a subsidiary reservoir for storing excess water which is utilised during periods of short supply.

Bandalling—A temporary spur composed of stakes (bamboos) driven into the bed of a river about 1 m apart along a line inclined at an angle to the current of the river; one or two (occasionally more in deep water) rows of mats are fastened to the face of the stakes extending from the surface of the water to 15 cm above the bed of the river. Its object is to induce scour under the mats, so that the sand scoured out will deposit in a bar parallel to and behind the bandal.

Base—Is the period (in days) for which water is supplied to the crops, and on which duty is calculated.

Basin Irrigation—A method of irrigating by which each tree is surrounded by a border to form a pool when water is applied. A method of irrigation by which land surrounded by natural or artificial banks is flooded, and when the water dries up crops are sown.

Barrage—A structure provided with a series of gates, erected across a river to regulate the water surface level and flow upstream, extending right across a river with the crest of the weir at one uniform level. (It is a gate controlled low weir.) A barrage is distinguished from a weir in that it is gated over the entire width and may or may not have a raised sill. The entire ponding up of the water is effected by the gates (and not the solid masonry weir) which are operated from a regulating bridge or platform above the high flood level. A barrage is sometimes called a *Regulator* or a *Diversion Dam*. The cost of barrage is much more than that of a weir.

Bell Bunds—Guide banks for training a river at the site of a bridge or weir; named after Mr. J.R. Bell who designed and introduced them first.

Berm—(i) The space left between the upper edge of a cut and the toe of an embankment: (ii) A horizontal strip or shelf built into an embankment to break the continuity of an otherwise long slope; (iii) The portion of a bank with horizontal top at a lower level than the top of the main bank. An addition to a bank at a lower level.

Berming—The deposition of material on the side of a channel forming a berm.

Bifurcation Gate—A structure that divides the flow between two conduits or channels.

Branch Canal or a Branch—Taking off from a main or another branch and having a head capacity of not less than 8 cumecs. There is no direct irrigation through it.

Breach—The gap created in the bank of a canal.

Breast Wall or Face Wall—When applied to irrigation practice, a wall immediately above the face of a submerged orifice.

Broad Irrigation—Irrigation with sewage from a town (instead of with natural water), in which the disposal of the sewage is the primary object.

Capacity—The authorized or designed full supply discharge of a channel.

Capacity Co-efficient—Is the number of acres irrigated in one day per cusec of supply.

Capacity-Factor—Is the ratio of mean supply of the canal to its authorized full supply or capacity.

Capillarity—The rise of water through soil pores without gravitational force. (See under "Soil Mechanics")

The capillarity factor differs for different soils depending upon their texture.

Capillary Fringe—The water held by the forces of capillary above water-table by the interstices of soil.

Capillary-Water—Water held in the pores of the soil above the water-table by capillary force, and which is not drained by gravity.

Caving—The erosion of a river or canal bank by the undermining action of water.

Chute—(i) A high velocity conduit for conveying water to a lower level; (ii) An inclined drop or fall.

Cohesive Soils—Clayey soils and sandy clayey soils are cohesive soils, while silt, sand and gravel are non-cohesive soils.

Command—The height of an outlet site of the water level in a channel above the general level of the land in the area to be irrigated by that outlet.

Conservation Works—Works like dams and reservoirs built for storing water during the days of plenty of supply for use in adverse times.

Control Point—A free fall, so designed that the water surface level above it bears a fixed relation to the discharge passing. The level is usually fixed with reference to the authorized full supply discharge.

Contracted Weir—A measuring notch with sides designed to produce a contraction in the area of the overflowing water.

Core Wall or Diaphragm Wall—A wall of masonry, sheet piling or puddled clay build inside a dam or embankment to reduce percolation.

Creep—The movement of the water under or around a structure built on permeable foundations.

Crest—(i) The top of a weir, dam, dike or spillway, frequently restricted to the overflow portion. (ii) The summit of a wave, peak of a flood.

Crib Dam—A barrier made of timber, built in compartments or bays which are filled with stone or other suitable material.

Critical Depth—The depth of water in a channel corresponding to (one of the recognized) critical velocities.

Critical Velocity—Critical velocity for a channel is that mean velocity which for a channel of a given depth will just keep the channel free from either silting or scouring its bed, when the water is running fully charged with silt up to the standard usually found in rivers. In open flumes critical velocity occurs when the energy of flow is minimum

Crop Intensity—Percentage of the area irrigated to the irrigable command.

Crop Ratio—The ratio of area under different crops of a particular channel. Crop ratio is fixed in order to make the discharge of the canal uniform.

Cross Drainage Works—When irrigation channels have to cross streams or drains in an uneven country, the works necessary to dispose of these drains are called cross drainage works.

Culturable or Cultivable Commanded Area—Is the portion of 'gross commanded area' which is cultivated, or is culturable. (Gross commanded area minus the area of uncultivable land such as roads and paths of a village.)

Culturable Irrigation Area—The gross irrigable area less the area not available for cultivation.

Curtain Walls—Cross walls (provided across the stream) built under the floor of a hydraulic structure (such as a culvert) at the upstream and downstream ends of the pavement to avoid scour and protect floors, abutments and wing walls, etc., and is carried up to scour depth.

Cut-off Trench or Key Trench—An excavation in the base of a dam or other structure filled with relatively impervious material to reduce percolation.

Deliquescence—Is the ability of a material to absorb moisture from the air and thus to dissolve and become liquid.

Delta—Is the total volume of water delivered to a crop (at the field or at the outlet or at the head of a canal), divided by the area on which it has been spread. In other words, it is the total depth of water required by a crop during the entire period the crop is on the field

Distribution System or Distribution Works—Water from a river or reservoir carried through canals, distributaries or minors, etc. on to the fields for irrigation.

Distributory—A small branch of a canal meant for direct irrigation. Its discharge varies from 1 to 5 cumecs.

Ditch Channel—A channel constructed by the side of and generally parallel to the parent channel, usually with different bed slope. A clear space of 9 m for sub-branch and 15 m for main branch should be left between the two channels. These widths may be varied by ± 3 m to allow min. shift in the alignment of the ditch channel.

Diversion Works—An obstruction thrown in the bed of a stream with a view to divert water into an off-taking channel. The diversion works are divided into two principal classes — (temporary) Spurs and (permanent) Weirs or Barrages.

Drift—The distance a boat (used for measuring discharge) travels downstream with the current (whether anchored or not) during the time taken to make a velocity observation.

Drop—A structure for dropping the flow in a conduit to a lower level for dissipating its surplus energy. A drop may be vertical or inclined.

Drowning Ratio—The ratio of the tail water elevation to the head water elevation, when both are higher than the crest, the overflow crest of the structure being the datum of reference.

Dry Crop—A crop which is raised entirely with the help of rainfall.

Duty—Is the relation between the area irrigated and the quantity of water used. (These terms have been explained in detail later)

Dyke—Is an earthern embankment built on each side of a river some distance away from its banks, to control floods. They are more or less like earthern embankment, kept about 1.2 to 1.8 m above the highest flood level. As far as possible curves should be avoided in earthen embankments.

Escapes—Weirs (with or witout sluices) through which surplus or excess water is removed from a canal, reservoir or stream into an escape channel. They may be built as separate structures or combined with outlets and located near aqueducts or drainge crossings. They are also used for flushing the canals to remove bed silt.

Escapes are made in the head reach of main canal and are different from silt ejectors.

Field Channels—See water-courses.

Feeder—A channel of short length constructed primarily to convey water from one source of supply or system to another, or within the same system, when the off-taking channels are already in existence and have to be grouped together (on the feeder).

Fender Piles—Wooden or RCC piles fixed 2.5 to 3 m centre to centre, at surplus sluices of off-takes for the canal to facilitate the movement of boats.

Field capacity—It is the amount of moisture content in the soil after the excess gravitational water has drained away. It depends upon the porosity of the soil.

Fish Ladder or Fishway—Is a device provided near weirs or dams to facilitate the migration of fish upstream or downstream around the weirs. It usually consists of an inclined chute from dam to the downstream river bed and is divided into compartments by cross walls. Each cross wall has a small hole at its bottom and in one corner for the fish to pass through, the holes in adjacent cross walls are staggered to reduce the velocity of water passing through the chute. The difference of water levels between upstream and downstream sides is thus divided into water steps by these cross walls.

Flank Wall—The retaining wall in continuation of abutments both upstream and downstream.

Flared Wall—Is a sort of retaining wall with its profile gradually changing from one slope to another, usually from vertical to 1: 1 or 1/2: 1 as required. The flared walls may be straight or curved.

Flash Board—A plank or slab (usually of timber) held horizontally by end girders or other supports, in vertical slots, on the crest of a weir dam, spillway, regulator or any check structure to head up water or control water level; a stop plank.

Float Gauge—A chain or tape gauge in which a float is substituted for the weight.

Float Gauging—Measurement of the discharge of water by floats to determine velocities.

Float Run—The fixed distance over which a surface float is timed.

Fore-bay—A reservoir or pond at the head of a pen-stock or pipe line.

Free-board—The margin between a canal bank or the crest of a dam and the full supply level. (Also see under "Bridges")

Free Weir or Free Fall Weir—A weir that is not submerged and in which the tail water stays below the crest. This is also called *Free Overfall*.

Free-Water—Water is soil which is in excess of the hydroscopic and capillary water and which can move freely downwards when the soil is porous and drainage available. This is also called "gravity water," Top surface of the "free-water" is called *water-table*.

Friction Blocks—Obstructions placed on the downstream floor of a weir or a fall to dissipate the velocity of the flowing water and maintain the standing wave on the glacis.

Fringe Water—Water in the zone immediately above the water-table. It may consist solely of capillary water, or it may be combined with gravity water in transit to the water-table.

Full Supply Factor—The area proposed to be irrigated in a project during the base period divided by the authorized full supply discharge of the channel at head.

Full Supply Co-efficient—The number of acres irrigable per cusec of capacity of a channel at its head; or, the area estimated to be irrigated during the base period divided by the designed full supply discharge of the channel at its head.

Ghat-fed Canal—Is a canal from a storage which derives its supply from monsoon rains in the "ghats".

Glacis—The sloping floor below and in continuation of the raised crest of a weir. Slopes between 1 : 3 to 1 : 5 for both the upstream and the downstream glacis are commonly used to give the maximum co-efficient of discharge. The downstream glacis should be flatter than the upstream except when there is considerable heavy material rolling over the crest when a flatter upstream slope would be provided.

Gross Command—Is the total area included with the irrigation boundry of a project or a channel including the farthest limits up to which the canal water is supposed to be supplied. It usually includes the roads and paths.

Gross Commanded Area—Is the total area which can be irrigated by a certain channel. Gross Commanded Area controlled by a water course is called the *outlet area* or the *chuck outlet*.

Gravity Dam—A dam depending solely on its weight to resist the water pressure. It may be straight or slightly curved in plan.

Gravity Water—*(i)* Water that moves through soil under the force of gravity. *(ii)* Supply of water by gravity as distinguished from a pump supply.

Groyne—See Spurs. A groyne with a curved head is known as "hockey groyne".

Guide Bank—A protective and training bank constructed at the site of a bridge or weir to guide the river through the waterway provided in the structure.

Head Race—A channel leading water to a water-wheel.

Head Regulator or Head—This term is usually applied to the control works constructer at the off-take of a channel subsidiary to a main canal. Piers with grooves are provided for the use of shutters to regulate the water flow for distribution.

Head-wall—A wall built across a small channel and provided with a regulating arrangement to head up water on the upstream side.

Head-water—*(i)* The water upstream from a structure ; *(ii)* The source of a stream.

Working Head Available—The minimum difference between supply and delivery water levels available.

Headworks—The works constructed at the off-take of a main canal from the river; includes the weir on the river, the dam at the storage site, etc.

Hydraulic-Fill Dam—A dam composed of earth, sand, gravel, etc. Generally the fine materials are placed towards the centre for greater imperviousness.

Hydraulic Jump—Is the hydraulic phenomenon, which is a distinct jump of water with a sudden and usually turbulent passage, produced when a shallow stream of water moving with a high velocity strikes a more slowly moving wall of water of sufficient depth. This phenomenon is quite distinct from the formation of the "Standing Wave". In the Punjab Irrigation practice no distinction is made between the hydraulic jump and the standing wave; the term standing wave is applied to both.

Hydraulic Drop—Is a local phenomenon like the standing wave formed as a result of a drop in bed level, a steepening of bed gradient or a sudden widening of the section, leading to an abrupt lowering of water surface. It is distinguished from the gradually varied flows as occur when so provided for fluming. When the depression of the bed is slight no hydraulic drop in the sense described above occurs though some depression of surface on the upstream takes place. In the hydraulic drop continuity of surface is maintained while in the hydraulic jump it is broken. Hydraulic drop is the reverse of the standing wave. In the former the flow changes from the subcritical to the hypercritical (velocities) while in the latter the conditions are reversed.

Hygroscopic-Water—Water found on the surface of the soil, and which is not capable of movement either by gravity or capilliarity and can only be driven off by heat.

Hygroscopicity—Is the ability to absorb and retain moisture without necessarily becoming liquid.

Hygroscopic Co-efficient—It is the moisture, in percentags of dry weight, that an oven dry soil will absorb in saturated air at a given temperature, or the moisture that an air dried soil is able to hold. It denotes the limit of moisture that can be retained at the ground surface in equilibrium with atmospheric water vapour.

Hyper-critical Velocity—A velocity in excess of the critical velocity.

Incoherent Alluvium—Is a soil composed of loose granular graded material.

Intake Weir or Diversion Dam—A barrier built for the purpose of diverting part or all the water from a stream into a different course. The weir raises the level of the water upstream.

Intensity of Irrigation—The ratio of the actually irrigated area during a year or during a crop to the culturable irrigable area. (Certain percentage of the cultivable area is generally left fallow every year and not irrigated).

Inundation Canals—Channels excavated (during old times) directly from rivers, with or without some form of head regulator, not essencially based on modern scientific principles, and dependent upon the surface level of the water in the river for their supply. They usually flow only during the summer months and bring in large quantities of silt beneficial to crops.

Inverted Siphon—When the central portion of a closed conduit or pipe is depressed below the entrance and outlet levels for conveying water under an obstacle, such as a river, canal, road or railway, the structure is termed an inverted siphon. Sometimes incorrectly termed as siphon. Known as "sag pipe" in America,

Irrigable Area—Area within the command which can be irrigated (both by flow and by lift).

Irrigating Head—The flow used for irrigation of a particular tract of land.

Jetty—A dike of piles, rock, or other material, extending into a stream or sea to induce scouring or bank building or for protection.

Kharif : Rabi Ratio—The ratio between the anticipated areas to be irrigated of these two crops. The usual ratio is 1 : 2, *i.e.*, rabi area is double the kharif area.

A Lateral—Name for a distributary in U.S.A.

Leaching—The washing out of salts from the upper zone of the soil by flooding. The salts are dissolved in the water which is drained off either on the surface or through the sub-soil.

Lift Irrigation—Water raised by pumps or other lifting devices to an area or some point in the supply system of which the level is too high for irrigation by flow.

Load—It is the weight of silt in movement in a channel and is usually expressed as grams per second.

Long Crop—The term is used generally to denote a crop that takes more than four months to mature. As a relative term, it denotes the longer of the two crops on a double-cropped land, the other crop being called *"short crop"*.

Loop-Bund—Is a subsidiary bund placed some distance behind the main bund where the main bund is threatened by erosion of the river bank. It forms a second line of defence.

Main Channel or Canal—This takes its supply directly from the Head-works on a river. A main canal generally does not do direct irrigation through it except to isolated patches inaccessible to other channels.

Major Distributary—(Commonly known as a Distributary). A channel taking its supply from a main line or branch for distributing water to minors and outlets.

Marginal Bund - An embankment constructed along the river at a short distance from the margin with the object of preventing inundation of the area behind the embankment.

Meandering—A meandering river is one which follows a sinuous path due to natural physical causes not imposed by external restraint, and is characterised by curved flow and a ternating shoals and bank erosion.

Minor Distributary—A direct irrigation channel taking its supply from a major distributary for supplying water to outlets. One cumec and under discharge.

A Minor—Is a small channel taken from a distributary where the water courses carrying water to the fields have to run longer than three kilometres.

Navigational Canal—Artificial canal, primarily meant for transport by water.

Non-perennial Canal—A channel which is designed to irrigate during only part of the year, usually in the summer reason and at the beginning and end of the winter season.

Non-uniform Flow—When depth varies in steady flow in a stream with constant discharge. Flow of which the velocity is undergoing a positive or negative acceleration.

Notch-Fall—A fall the crest of which is usually at or near the be l level, generally without a glacis.

Ogee—The overfall of a spillway in the shape of a double or S curve, which is convex at the top and concave at the bottom. An ogee shape serves the purpose of a sloping apron which ensures the formation of a standing wave for varying discharges and the residual velocity obtained on the downstream side is less than that on the vertical face type and as such are best suited for high falls.

Open Discharge—Is the total of the daily discharges in cusecs divided by the number of days the canal is allowed water for irrigation.

Outlet Discharge Factor—The 'Duty of Water' with reference to a suitable 'base' and the 'place' of its measurement as decided upon.

Overlap Allowance—When the time of one crop is about to finish and that of the next has already started, more discharge in required in the irrigation channel due to overlap. This excess discharge required is called overlap allowance.

Parabolic Weir—A measuring weir whose notch is bounded on the sides by parabolas such that the flow is proportional to the head,

Penstock—(i) A closed conduit or pipe for supplying water under pressure to a turbine (for producing electricity). (ii) A sluice or flood gate for restricting or regulating the flow from a head of water.

Percolation—Flow of water through the particles of soil (porous substance) due to the force of gravity or pressure of head.

Perennial Canal—A channel which carries water all the year round. Irrigation is said to be perennial when water is applied at a fairly equitable rate during the whole season of the crops.

Pick-up Weir—A weir constructed across a river at the headworks of a canal to raise the level of water sufficiently high for it to flow into the channel. This term is generally applied to a weir across a river on which there is a storage reseevoir or a dam. A pick-up weir serves the same purpose as an intake weir but is constructed as an adjunct to a reservoir.

Puddle—A mixture of clay, sand and gravel or clay and moorum, in the proportions of 2:1, (or only clay) well kneaded with water, which is placed in structures to form a compact mass to reduce percolation of water.

Pug—To pack with clay or similar material generally for the purpose of checking leakage or to render the surface water-tight.

Rabi Capacity-Factor—Is the ratio of mean supply of rabi season to 'capacity'.

The controlling factor in the design of a channel is the rabi "full supply co-efficient".

Race—The channel that leads water to or from a water wheel (water-power plant) ; the former is called "head-race", and the latter "tail-race".

Rapids—A sudden fall of level in the ground along the alignment of a canal joined by an inclined bed is called a rapid. Instead of falls or drops in the bed of channel, rapids are provided to get the necessary change of level where a sudden drop is not practicable. Water flows down a steep incline on which energy is dissipated by friction, impact against stones, and small natural standing waves caused by unevennesses. Rapids are generally made of stone boulders and protected for some distance upstream and downstream and the bed widened to decrease the velocity. The slope of a rapid should be reduced as it descends so as to gradually assimilate with the bed of the canal.

Rating— (*i*) The relation, usually determined experimentally, between two mutually dependent quantities such as gauge and discharge of a stream ; (*ii*) Current-meter vane revolutions, and water velocity, etc. ; (*iii*) Calibration of the meter.

Reach—A comparatively short length of a stream or channel.

Regime or Regimen Flow—Is that state of stream flowing in self borne alluvium when its slope and shape have reached a stable form as the result of its flow characteristics and there is neither silting nor scouring. The channel after attaining its section and longitudinal slope, is said to be in *final regime*.

Regulator—A structure through which the discharge can be regulated or varied as required ; also applied to a structure provided with mechanism for varying the water surface level above it. (Also see Barrage)

Regulating Notch—A trapezoidal notch built on a floor across the channel to regulate the supply where change in the depth of the channel is made without any drop in the bed level. This type of a notch is not fitted with planks or shutters.

Ripples—(*i*) Surface ripples are small undulations caused by unevenness in the bed. (*ii*) Sand ripples result from the movement of bed sand not being uniform.

Riprap—Broken stones (usually without dressing) placed on earth surfaces for their protection against the action of water or weather. (This is sometimes known as "pitching"). Also applied to brush or pole mattresses, or brush and stone, or other similar materials under

protection.

Rotation (Rotational Working or Roster)—When the demand exceeds the available supply recourse is had to the system known as Rotational Working. This system is applied to channels or to groups of outlets. Each channel or group of outlets takes a turn of full supply for a certain number of days, the others being closed to admit of this. The unit period for which the channels or outlets run, or are closed is known as a Rotational Turn.

Scour—The removal of material from the bed of a channel by flowing water.

Sediment or Detritus—Non-floating fragmental material transported by, suspended in or deposited by water. Classification of sediments is given under "Silt Flow in Channels".

Seepage—The water which by action of capillary attraction passes underground from channels or tanks through close soils and does not appear visiby in the vicinity but the area becomes "water logged" The percolation of water into or from the soil; Ifiltration.

Semi-Arid—A term applied to lands or climate, neither entirely arid nor strictly humid, in which inferior crops can be grown without irrigation.

Silt—Water-borne sediment consisting of fine earth, sand or mud, including both suspended and bed load carried in natural river waters. A general term meaning sediment of any grade in a river or canal. Silt is sometime defined as a substance that will fall in still water through a distance of 10 cm. in period of not less than 8 hours. (See under "Soil Mechanics").

Silt-charge—Proportion of silt per unit volume by weight in water.

Silt Excluder—A silt regulator located at the head of a channel.

Silt Extractor or Ejector—A silt regulator located on a channel other than at the head.

Also see under "Escapes". Such devices are generally provided a little below the head regulator. A velocity of about 4.5 to 6 m per second is required to dislodge silt.

Silt-grade—Average diameter of the silt particles.

Silt Regulator—A regulator provided with under-sluices for the escape or wash-out of the heavily sand laden bottom water of a channel.

Silt Vanes—Vertical vanes arranged in the bed of a channel with the object of diverting the heavily sand laden bottom water in order to control the sand entering an off-take.

Siphon or Syphon—A closed conduit (pipe or tube) for conveying water over an obstacle shaped like an inverted V by raising it above it's original surface level so that a part of it rises above the hydraulic grade line, and delivering it at a lower level. It utilizes atmospheric pressure to effect or control the flow of water through it. (An inverted siphon has none of the properties of a siphon-the term is a misnomer).

Siphon Aqueduct—Where the water level in the drainage or the stream is about at the same level as that of the canal or above the canal, and the stream is passed below the canal by lowering its natural bed level while passing under the canal and raising it again on the down-stream side. A siphoned stream or drainage is liable to be filled up with debris and thus exert heavy upward pressure on the covering of the siphon-vent or the bottom of the canal, especially when there is no water in the canal. To guard against this, the bed of a drainage channel is dropped at the entry but is not raised again at the exit and is continued at the depressed bed level. This is termed an "Aqueduct and Fall combined". The working of a siphoned structure is more difficult in alluvial soils because of the large quantities of silt coming in.

Siphon Spillway—Is a discharging device on the siphon principles for discharging surplus water over dams, and is automatic in action. It is sometimes adopted in place of the ordirnary overfall weir in water-supply storage dams for overflow of flood water. Siphon spillway consists of a number of masonry siphon units placed side by side in the body of a storage dam. Full head between the reservoir level and the level of tail water is utilized. Much higher discharge can be passed through the same waterway and the length of the waste weir can therefore be considerably reduced.

Siphon spillways reduce flood lift adding to the capacity of the reservoir and are flood proof. As the inlet level is below the top of the dam they do not discharge debris, but scour silt from the bed of the dam. There are two types of siphons. Hood siphons and Volute Siphons.

Sluice—(i) An outlet for the water from a canal to the fields; (ii) A conduit for carrying water at high velocity; (iii) An opening in a structure for passing debris; (iv) To cause water to flow at high velocities for wastage for purposes of excavation, ejecting debris, etc.

Sounding—Measuring water depth of a river or a channel (with a rod or a string and weight).

Spillway—A passage for the flow of surplus or waste water in a weir or conduit.

Spoil Bank—Where the excavated earth from a canal is more than the bank work, this extra earth is dumped in the form of another bank parallel to the canal bank and usually of the same height.

Spur or Groyne—Spurs are made to train the flow and reduce the velocity in a channel and cause silting. They are made transverse to the river flow and extend from the bank in the river. They are generally constructed of bushwood or wooden piles driven into the bed or berms.

Standing Wave—A sudden rise in the water surface formed when a rapidly moving stream of water strikes a slowly moving wall of water downstream, this is accompanied by white foamy splash of water in the region of impact. A standing wave persistently forms at the same place. This condition occurs downstream of irrigation falls when there is insufficient depth available downstream to form a hydraulic jump. Standing waves can be noticed either above or at the foot of a weir, or on the downstream side of bridges discharging in flood. There is no impact and no energy loss in the formation of standing waves.

Staunching Walls—Walls provided to prevent leakage at the junction of earthwork and masonry walls (say wing walls). When water passing through comparatively permeable earthwork meets practically impermeable masonry it has a tendency to creep along the face of the latter in order to gain an exit. Staunching walls are constructed to increase the length to be traversed by creeping water and consequently resistance to its flow. A staunching wall has almost the same function as a curtain wall It is a wall constructed behind the abutment.

Steady Flow—Is that state of flow in a stream where the discharge remains constant across any defined section, at all times.

Sub-critical Velocity—A velocity less than the critical velocity.

Submerged Weir—A weir which in use has the tail-water level higher than the weir crest, by which the discharge is effected.

Sub-critical Flow—Flow at velocities less than of the recognized critical values.

Super-critical Flow—Flow at velocities greater than one of the recognized critical values and also termed *Hyper-critical Flow*.

Superpassage—A work which carries one channel over another lowering the bed level of the lower channel; or when the drain is over the canal.

Suppressed Weir—A weir whose length is the same as the water surface width of the channel upstream of it, and sides are flush with the channel and whose base is at the same level as the bed of the channel upstream of it, thus eliminating (suppressing) end contractions of the over-flowing water. A welr may be suppressed on one end or both ends.

Tail—This term is usually applied (prefix) to the works built at the finishing end of a channel for the distribution of its water thereat, *e.g.*, tail cluster, tail regulator, etc.

Tail Race—(i) The channel between the silt extractor and the river through which the escapage water is discharged. The gradient of the tail race should be steeper than that of the canal. (ii) A channel conducting water away from a water-wheel.

Tail Tank—Is a reservoir supplied with water from a canal whenever in excess of canal requirements, having its own command and usually situated near the tail of a canal.

Tail Water—The water just down-stream of a structure.

Talus—A protection at the downstream end of a weir or fall, consisting of blocks of concrete or masonry.

Time Factor—It is the ratio of the number of days a channel actually runs to the number of days (crop period) it should run as per design.

Time-Lag or Lag—The difference in time between the occurrence of any alteration in discharge, level, pressure, etc., on any point on a stream or a structure and its occurrence taken to reflect at another point.

Toe Walls—Longtudinal shallow retaining walls built near ground level (at the foot of the slope) for supporting the pitching on the face of earthern embankments or flared walls.

Uniform Flow—Is steady flow in a stream when the depth does not vary with constant discharge.

Waste Weir (or Spillway)—Is an escape provided for the passage of surplus water from a tank or a reservoir.

Water-Courses or Field Channels—Are small channels generally made by cultivators which receive water through outlets fixed in the banks of distributaries or minors to the fields. Water courses are owned and maintained by the cultivators.

Water-Logged—A condition of land where the water-table is at or near the ground level and becomes detrimental to plant life. Water-logging may result from over irrigation or seepage due to inadequate drainage.

Weir—Weir is a general term for a continuous solid barrier (wall of stone or masonry) built across a river or channel over which water may flow and which raises the water surface level upstream in order to supply a canal taking off above it and to pass over its top the excess water. Water level in a canal must be at a certain level to command the land under irrigation by gravitation, but the water in the river is not generally available at that level, and especially all the year round, therefore, the river water is tapped at a considerable distance higher up the area under command and a weir is also constructed to raise the water level in the river to the height required by the canal. A weir also serves to store water to some extent for tiding over small periods of short supplies Silt movement is also controlled through a weir although it encourages silting on the upstream side. In Madras an "escape" is sometimes called a weir. *Anicut* is Tamil name for weir. Weirs on impermeable foundations, i.e., rock or hand clay, are usually made of concrete or masonry. They will stand a depth of water flowing over equal to their own height if well anchored by grouting steel bars into the rock.

Wilting Point or Wilting Co-efficient—Is that (low) water content of the soil at which plants can no longer extract sufficient water for their growth.

2. SILT FLOW IN CHANNELS

The silt carried by a river is the result of erosion by water on the soil in the catchment area of the river. The proportion of silt to water and the size of silt particles carried depend upon the nature of the surface soil, its slope, and the rainfall in the catchment area. Silt carrying capacity of the water in a channel depends on discharge, surface-slope, grade of silt and the silt charge. The water in the main canal carries silt with a high silt factor which is gradually reduced in branch canals and minors. The channel section slope is therefore fixed according to silt analysis (silt charge and silt grade).

There are two classes of silts (i) bed silt, which is dragged or rolled along the bed, and (ii) suspended silt. In a particular channel, the heavier the particle the closer to the bed it remains, depending upon the velocity. Silt that may be suspended in a bigger canal on account of its high velocity may start rolling in a small channel or even be dropped in the bed. A *non-silting velocity* is the velocity at which a channel is just able to carry its silt load without dropping any of it in the bed.

Variation of Silt-Charge or Silt-Grade in a Channel

Considerable variations are noticed in the silt-charge and silt grade which are carried daily by the water in a channel. The silt during summer is muddy and it becomes fine and coarse sand in winter. The silt charge by weight is maximum in rainy season when the parent river is in floods. Irrigation channels berm up most in July and August when fertilizing fine silt-charge is very high, and pass high silt-charge during June, July and August without silting up. They silt up most in October and November when they pick up coarse silt from the falling parent river, due to large reduction in silt-charge by weight in their own waters. ("Silt grade" is size and "silt-charge" is proportion).

Silting in small channels can sometimes be remedied by widening the bed.

Silt Distribution at Various Depths of a Channel Section

The amount of silt suspended in various layers is different. It is about 60 to 70 per cent by weight near the surface, and about 130 per cent near the bed. At a depth of about 0.6 D from the surface where the mean velocity occurs, the quantity of silt suspended (or silt-charge intensity) represents a fair average of the total quantity of silt.

Distributary channels silt up most (due to defective designing) in the head reaches and berm in the tail reaches. The middle reaches are generally free from silt trouble. Therefore, it is desirable to have high ratio of bed to depth at the head of a reach, than otherwise required.

At a bend in an irrigation channel there is shallower depth on the inside and greater depth towards the outside of the curve. There is a constant flow of silt along the bed from the outside to the inside of the curve which tends to deposit silt and grow berms on the inside and errode the banks on the outside. Any off-take from the outside of a channel curve would therefore carry relatively low silt-charge by weight and grade.

Silt problem is practically non-existent in non-alluvial soils.

Silt is a great *fertilzing agent*, therefore, some silt must be carried in canal waters. It facilitates the formation of berms along canals and forms a water-tight coating on the canal section which reduces seepage of water. Channels have to be so designed that they are able to carry the maximum quantity of the useful silt without otherwise impairing the flow of the channel. The correct quantity and grade of silt that a particular channel should carry, which depends upon various factors, is a vexing problem for the engineer. From a look at the bed when the channel is dry an indication can be had whether the bed load is heavy, medium or light. Where the bed load is small, the silt particles form dunes in the bed. With increase of load the dunes disappear presenting a smooth bed surface. Further increase of load forms small hills called anti-dunes which travel upstream. The anti-dunes have the reverse shape of the dunes, their downstream faces are flat and the upstream steep. Therefore, a *regime* channel has to be designed, i.e., a channel which neither silts nor scours but carries a good amount of the silt.

Critical Velocity Ratio (CVR)—V/V_0

It is the ratio between the actual mean velocity (V) in a channel to the critical velocity (V_0) calculated from any of the standard formulae. For a non-silting channel the CVR should be one or a little more than one at the head or head reach, and about 0.8 towards the tail. This factor also takes into account the channel dimensions and does not show departure from *regime*.

It is advisable to maintain a constant velocity throughout the length of a canal so that the silt suspended in it may be carried on to the fields.

Weight or Density of Silt. Weight of dry silt is about 1280 to 1360 Kg/cu. m. In natural state silt has about 45 to 55 per cent voids. Silt swells when wetted. (See under "Bulking of Sand" in Section 8). Since volume of silt is very variable silt content should be determined by weight.

Desirable Velocities in Channels

Velocity varies considerably with the type of the channel and the nature of materials forming the bed and the sides. The velocity is least near the bed and banks and is greatest in a plane at about 0.3 depth below the water surface. In canals of over 15 m bed width the mean velocity in the central segment has been found to be just double the mean velocity in the slope segment. This ratio reduces to 1.5 on small channels. For changing a discharge the bed-width is generally changed.

An average velocity of 60 cm to 90 cm per sec., with 45 cm per sec. minimum, will generally prevent the deposition of silt or growth of weeds and avoid scour in an earthen channel.

The usual velocity of water in earthen channels is as follows :—

In main canals	110 cm/sec.
In branch canals	90 ,,
In large distributaries	75 ,,
In small distributaries	60 ,,
In very small distributaries	45 ,,
In water-courses and field channels	30 to 45 ,,

Since a canal in ordinary soil cannot be made with a greater velocity than about 110 cm per second and the critical velocity is greater than that in canals which have depth of over 2.7 m, it is not desirable to design canals in ordinary soil with a greater depth than about 2.7 m unless the water in silt laden.

In the case of channels fed by tube-wells or from reservoirs, or channels in non-alluvial soils non-scouring velocities have to be kept as the water is comparatively silt free and there is no danger of silting. On very flat slopes, such channels should be made deep in proportion to their width.

The velocity is also generally increased near cross drainage works such as aqueducts, to reduce the section of the channel and minimize the cost of such works. A velocity of 150 cm per sec. or even more (generally not exceeding twice that in the canal) is allowed.

Mean Velocities Safe Against Erosion or Scour in Channels of Different Materials

	cm per sec.
Soft earth or very fine clay	8 to 9
Soft clay or fine clay	15 to 23
Very fine or very light pure sand	23 to 30
Very light loose sand or silt	30 to 45
Coarse sand or light sandy soil	45 to 60
Average sandy soil and good-loam	60 to 75
Sandy loam	75 to 85
Light ordinary earth or sandy bed	75 to 90
Average loam or alluvial soil	85 to 90
Firm loam, clay loam	90 to 115
Firm gravel or clay	110
Stiff clay soil; ordinary gravel soil, or clay and gravel	120 to 150
Broken stone and clay	150
Grass	90 to 150
Coarse gravel, cobbles, shingles, shale	150 to 180
Conglomerates, cemented gravel, soft, state, tough hardpan, soft sedimentary rock	180 to 245
Soft rock	135 to 245
Hard rock	305 to 460
Very hard rock or cement concrete	460 to 760
For Lined Channels with	
Boulder lining	150
Burnt clay tile lining	180
Cement concrete lining	200

Bottom velocity may be about 4/5th to 3/4th of the above values which represent only the average conditions. Actually the safe values depend on the hydraulic mean radius.

A higher velocity can be given to a smaller channel because small body of water has less erosive power than a larger quantity. Quick flowing canals should be narrow and deep while slow flowing canals should be shallow and broad. Velocity needed to prevent silting or erosion varies with the depth of flow according to the relationship given in the table under "Kennedy's Theory" for fine sand-silt.

The principle of design of canals on alluvial and non-alluvial soils in quite different. Canals on alluvial soils carry silt and sand load.

The following velocities (V) of water, in meters per sec., in a river will move stones of diameter (d) :—

V—m/sec.	0.15	0.30	0 60	0.91	1.22	1.52	2.13
Dia.—mm	0.9	3.6	16	32	57	89	178
V—m/sec.	3.05	4.57	6.09	7.62	9.14	10.7	12.2
Dia.—mm	0.37	0.82	1.43	2.25	3.23	4.39	5.67

The following table gives the relation between mean velocity, hydraulic mean depth, and erosive or scouring power of a stream :—

There is no scour in a channel of hydraulic mean depth	Until a mean velocity is reached of—	
metres	*cm per sec.*	
0.30	12	Fine silt
0.76	21	
1.52	27	
3.05	45	
0.30	27	Heavy silt and fine sand
0.76	45	
1.52	55	
3.05	65	
0.30	55	Coarse sand
0.76	65	
1.52	90	
3.05	110	
0.30	65	Small pebbles (peas size) and gravel
0.76	90	
1.52	110	
3.05	135	
0.30	150	Large pebbles (hen's egg size) and coarse sand
0.76	180	
1.52	215	
3.05	275	
0.30	460	Large stones
3.05	700	

The following bottom velocities in a channel will just produce motion in the substances mentioned :—

Cm per second	Material
8	Soft earth, fine clay, river mud or silt
15	Common clay
21	Fine sand
24	Coarser sand
30	Fine gravel and coarse sand
60	Pebbles 25 mm diameter
90 to 100	Pebbles, egg size
150	Stones 75 mm diameter
200	Boulders 150 to 200 mm diameter
300	Boulders 300 to 450 mm diameter

If the velocity in a channel is very high, the water will erode its bed, if the velocity is very low, the sediment held in suspension will settle down.

Velocities to Move Stones :—Chailly's formula : (English formula)

$V=5.67\sqrt{G.d}$ or $d=\frac{V^2}{85}$ | d=dia. of stone in ft., G=specific gravity of stone (2.65)

Aging of Channels. Deposit of silt increases resistance to erosion and the beds can tolerate higher velocities when silt has been deposited. Velocities in new channels can be decreased by "check structures" (such as spurs and groynes) and deposition of silt encouraged.

Mean Velocities which will not Erode Channels after Aging :— (Am. Soc. C. Engrs. 1926).

Material of channel bed	Velocity in cm/sec.	
	Shallow ditch	Deep canal
Fine sand or silt	15 to 45	45 to 75
Coarse sand or sandy loam	30 to 45	55 to 75
Silty or sand loam	30 to 55	60 to 90
Clayey loam or sand loam	45 to 60	65 to 110
Fine gravel	60 to 75	75 to 150
Well graded gravel	65 to 110	120 to 200
Pebbles, broken stone ...	75 to 120	150 to 180
Stone masonry	230 to 460	—
Solid rock or concrete ..	460 to 760	—

Silting of Channels. All distributary channels have a tendency to silt up in their head reaches and to grow berms in their tail reaches. The reasons for silting up in head reaches are :—

Non-regime Section—It may sometimes occur that the regime slope is not available in the reach considered to transport silt of a coarse quality. The canal will drop its silt which it cannot carry in the head reach. The lower reaches will thus have to deal with less silt in water. If the slope is inadequate the canal will tend to increase its slope by silting at the beginning (below control points). There can be made a regulator at the canal head so as to admit finer silt only which can be carried by the slope available. If the *head regulator* is defective and it allows entry of excessive silt charge, the coarser part of the silt will drop in the head reach. Ordinary heads built at right angles automatically draw off the coarse silt from the parent channel. Falls create a natural break in regime and act as controls. They are, therefore, very suitable points for modifications in dimensions.

To prevent silting the discharge should be increased to enable the channel to carry higher charge, and slope (and consequently velocity) should be increased to enable the same discharge to carry the higher charge. In all cases when the slope is the controlling factor, the channel should be designed for the silt factor that the available slope indicates. In the case of a minor, if the silt of the same character is to be withdrawn as in the parent channel, a greater slope has to be given in the minor to avoid silting up.

If the *outlets* are defective and do not draw in their due share of silt, the channel will silt up in the head reach mostly, and in other reaches to a lesser extent. If the channel runs long periods with *lower supplies*, it will also silt up in head reaches to adjust the silt charge due to reduced depth and velocity. The reasons for berming of tail reaches are due to low velocities and growth of weeds and grass which necessitate frequent cutting of berms.

Kennedy's Silt Theory

Critical Velocity (V_0). Critical velocity is a velocity which causes neither silting nor scouring. This average velocity has a certain relation to the depth of water in a channel. Fine silt has a lower critical velocity than heavy silt. Critical velocity is the only velocity which will maintain the *regime* of the canal.

Kennedy's Fundamental Equation :

$$V_0 = K\, d^{0.64}$$

V_0—is the critical velocity in metres per sec.,

K—is a constant depending upon the type of the silt,

d—is the depth of water in metres over the bed portion of the channel (full supply depth.).

The value of K for various grades of material may be taken as:—
0.41 for very fine silt (as in Sind canals),
0.53 for light sandy silt,
0.55 for fine sand-silt (as in Punjab canals),
0.57 for coarser light sandy silt,
0.59 for fine sand silt (as in Burma rivers),
0.65 for sandy loam (as in Madras rivers),
0.70 for coarse silt and coarse sand,
0.78 to 0.97 for sand and small bajri,
1.62 to 1.95 for bajri and gravel,
1.95 to 2.27 for gravel and boulders.

The following equations have been evolved (based on the above Kennedy's theory) for the channels in Godavary and the Krishna delta systems :

Godavary $V_0 = 0.39\, d^{0.55}$

Krishna $V_0 = 0.53\, d^{0.59}$

V_0 is not constant but varies with the depth of the channel. In the case of small channels, the greater the water-depth, the steeper can be the bed-slope. When the slope is fixed, the bed can be widened and depth decreased.

The mean velocity of the channel should never be less than critical velocity.

Design Data for Earthern Channels based on Kennedy's Theory with 0.0225 Silt Rugosity Co-efficient

Discharge		Bed Width	Water Depth	Bed Slope	Critical Velocity	Mean Velocity
Cusecs	Cumecs	m	m	1 in ——	Ratio V/V_0	m
2	0.06	0.61	0.30	2500	1.00	0.24
4	0.11	0.82	0.37	2500	1.03	0.30
6	0.17	1.07	0.43	2860	1.00	0.31
8	0.23	1.22	0.46	2860	1.03	0.33
10	0.28	1.45	0.49	3330	0.92	0.34
12	0.34	1.60	0.53	3330	1.00	0.36
14	0.40	1.68	0.55	3330	1.01	0.37
16	0.45	1.83	0.58	3640	1.00	0.38
18	0.51	1.91	0.59	3640	1.00	0.39
20	0.57	2.01	0.61	3640	1.00	0.40
30	0.85	2.44	0.68	3640	1.02	0.44
40	1.13	2.82	0.78	4000	0.98	0.46
45	1.27	2.97	0.81	4000	0.99	0.47
50	1.42	3.13	0.84	4000	1.00	0.47
60	1.70	3.35	0.88	4000	1.00	0.51
60	1.70	1.22	1.31	2000	1.01	0.55
60	1.70	6.55	0.61	5000	1.01	0.53
70	1.98	3.66	0.91	4000	1.01	0.53
80	2.26	3.96	0.97	4000	0.97	0.54
90	2.55	4.11	1.02	4000	0.98	0.54
100	2.83	4.42	1.04	4440	1.00	0.55
100	2.83	4.00	1.08	5000	1.00	0.57
125	3.54	4.88	1.11	4440	1.00	0.58
150	4.25	5.18	1.12	4440	1.00	0.61
175	4.95	5.64	1.23	4440	1.01	0.63
200	5.66	5.94	1.31	4440	1.01	0.66
250	7.08	6.70	1.43	4440	1.01	0.71
300	8.50	7.31	1.46	4440	1.03	0.71
350	9.91	8.08	1.57	5000	0.97	0.71
400	11.3	8.69	1.61	5000	1.06	0.74
450	12.7	9.30	1.68	5000	1.00	0.76
500	14.2	9.75	1.72	5000	1.00	0.77
600	17.0	10.7	1.83	5000	1.00	0.79
700	19.8	11.9	1.86	5000	1.01	0.82
800	22.6	12.8	1.92	5000	1.01	0.86
900	25.5	14.0	1.95	5000	1.02	0.87
1000	28.3	15.3	1.98	5000	1.03	0.91
1060	30.0	13.9	2.30	5000	1.00	0.93
2000	56.6	25.4	2.26	5700	1.03	0.94
2120	60.0	26.9	2.30	5000	1.07	1.03
5000	142	56.5	2.50	6670	0.98	0.97
10,000	283	105	2.59	6670	1.02	1.03
10,000	283	110	2.68	8000	0.93	0.96

The following table gives Kennedy's critical velocities (V_0) for fine sand-silt (Punjab canals) for the values of K=0.55, for various depths. **Side slopes 1/2 to 1. (d is in metres and V_0 in metres/sec.)**

d	V_0	d	V_0	d	V_0	d	V_0
0.15	0.16	1.13	0.59	2.04	0.86	2.96	1.10
0.30	0.26	1.22	0.62	2.13	0.89	3.05	1.12
0.40	0.30	1.31	0.65	2.22	0.91	3.14	1.14
0.46	0.33	1.37	0.67	2.29	0.93	3.20	1.15
0.52	0.36	1.43	0.70	2.35	0.94	3.26	1.17
0.61	0.42	1.52	0.72	2.44	0.97	3.35	1.19
0.70	0.43	1.61	0.74	2.53	0.99	3.44	1.21
0.76	0.46	1.68	0.76	2.59	1.01	3.50	1.22
0.82	0.48	1.74	0.78	2.65	1.02	3.57	1.24
0.91	0.52	1.83	0.80	2.74	1.04	3.66	1.25
1.00	0.55	1.92	0.83	2.83	1.07	4.57	1.45
1.07	0.57	1.98	0.85	2.89	1.08	6.10	1.74

For Sind canals multiply V_0 by 3/4

To maintain Kennedy's relation of critical velocity to depth, the ratio of bed-width/depth should be between 1 and 5, and the lower value is preferred. A mean velocity of above 0.4 m will carry silt at a depth of 0.61 m but it will fail to carry silt at a depth of 1.52 m for which a velocity of over 0.7 m will be necessary

In alluvial soils, depth does not affect the value of the roughness co-efficient to any appreciable extent, but in the case of boulders and gravel, it varies greatly with depth.

Max:ratio of $\frac{\text{bed-width}}{\text{water-depth}}$	3.5	4	4.5	5	6	9
Discharge—cumecs ...	0.28	0.71	2.83	5.66	14.16	28.31

Lacey's Theory

According to Lacey, a channel flowing in its own silt will, if continued uninterfered with, reach final stability, and where the conditions of discharge and silt remain constant final regime will be obtained in time. Natural streams have a tendency to assume semi-elliptical shape; coarser the silt, the flatter and wider the semi-ellipse, while finer the material carried, the more the section approaches a semi-circle If a canal is designed with a section too small for a discharge and its slope is kept steeper than required, scour will occur till final regime is obtained. According to Lacey, the ratio of bed width to depth affects the silt bearing capacity of the channel, or in other words, the shape of the channel for a given discharge is a function of the silt grade; channel in finer material being narrower and deeper. There is only one section of a channel and only one slope at which the canal carrying a given discharge

will carry a particular grade of silt (silt factor). For constant silt grades, the ratios of the bed width to depth (or more accurately, that of the wetted perimeter to the hydraulic mean depth) steadily diminishes with reduction in discharge. Before a regime channel can be designed, it is necessary to select an appropriate ratio of bed-width to depth, and thereafter to assign the correct depth and water surface slope.

According to Lacey's theory there is only one value of the velocity, the cross sectional area, the wetted perimeter and the hydraulic mean depth.

Application of Lacey's Formulae

Lacey's formulae are regime formulae and hence depend on regime conditions, i.e., constant flow and constant silt charge. Final regime velocity is a function of the discharge and silt factor, for a given discharge and silt factor "f", regime velocity "V" can be worked out and from which A and R deduced. The bed width and depth can be calculated provided the shape of the channel is specified. Channels are usually excavated to 1 to 1 side slopes, it being assumed that after silting up they will have side slopes of approximately 1/2 to 1.

The most important factor in the use of Lacey's formulae is the fixing of correct values for the silt factor "f" which depends upon rugosity of channel silt grade. According to Lacey "f" is proportional to V^2/R. If a channel is said to be in regime in any given reach, it is better to observe the actual mean velocity, said to be the regime velocity and the hydraulic mean depth, and calculate the silt factor from the expression :

"f" $= 1.76\sqrt{M}$ Corresponds to a max: size of silt 0.01 mm to 0.257 mm.

M is mean diameter of silt in mm.

(Lacey's theory does not apply to Bombay Deccan canals because silt is non-coherent and soil is non-alluvial.)

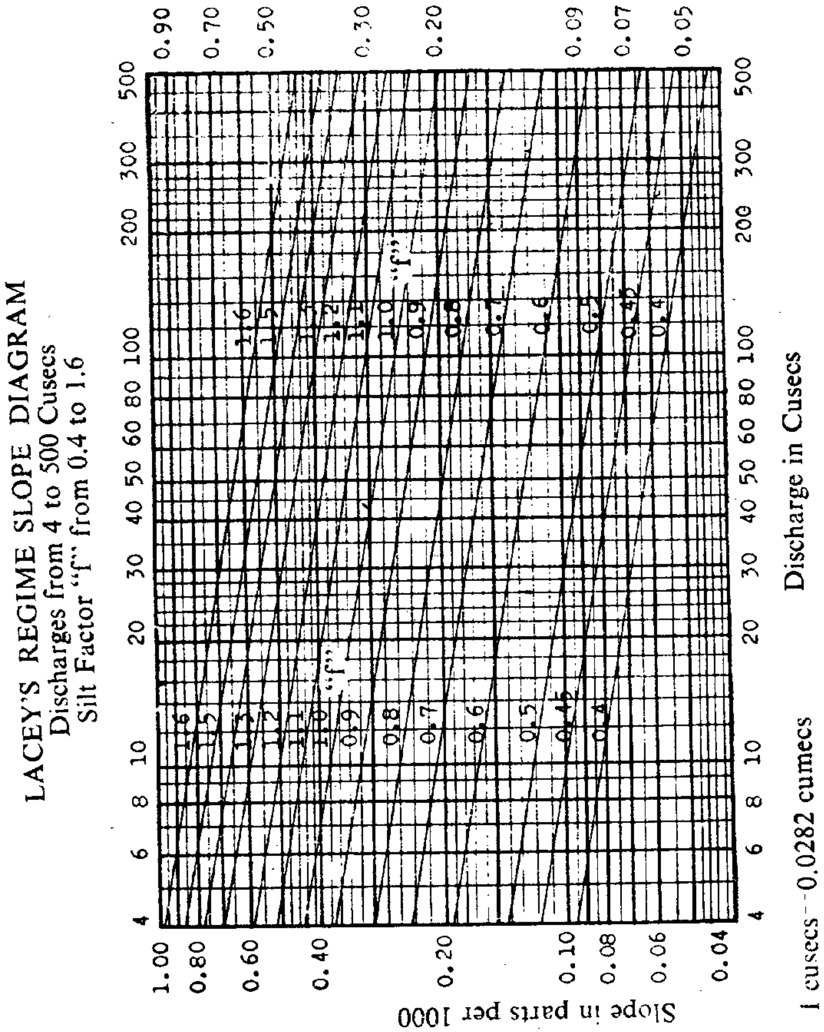
For canals the value of "f" is generally taken as follows: (as given by Lacey in his original paper)

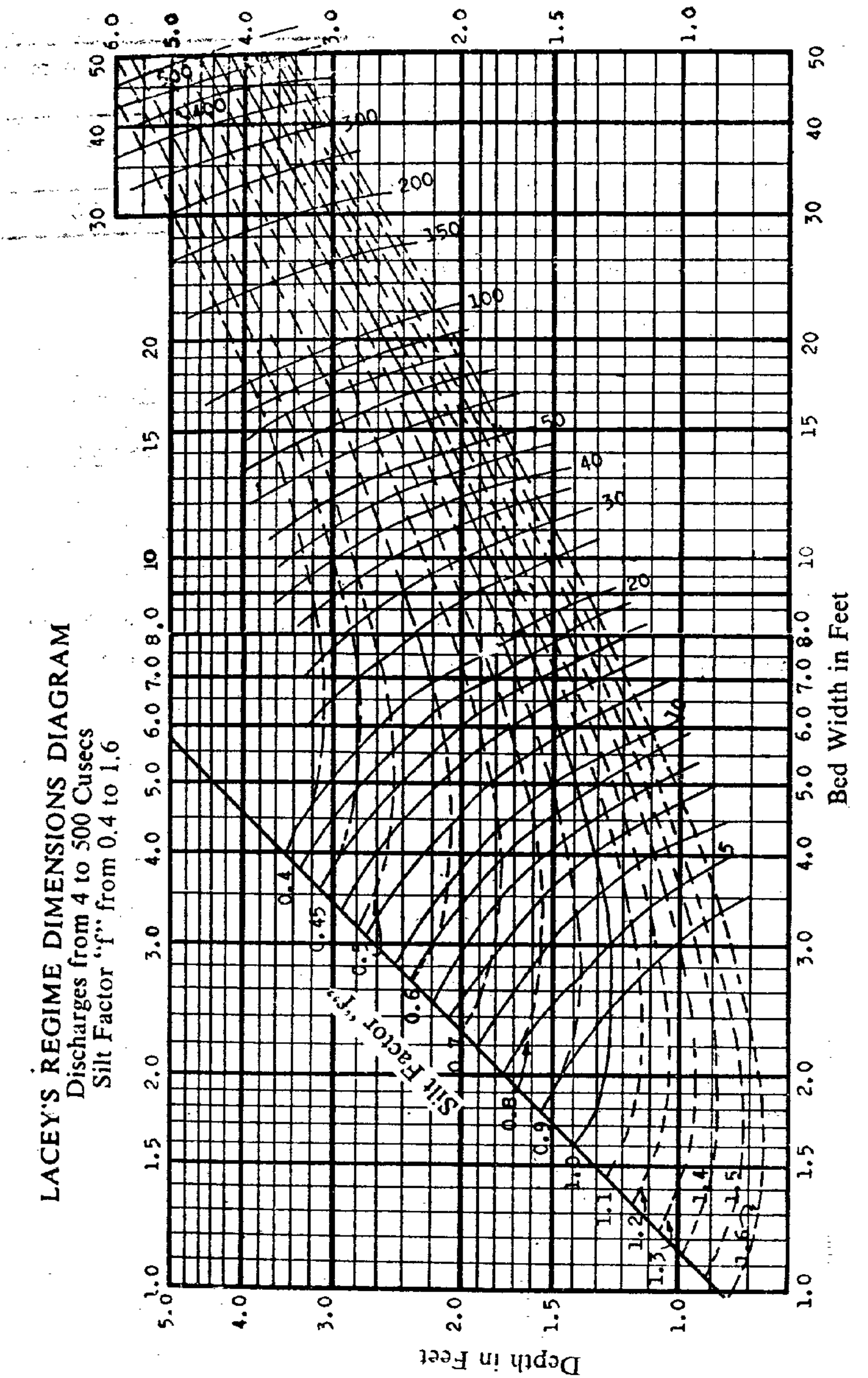
0 40—very fine silt as at Ismalia canal, Egypt,
0 50—fine silt as at Madras, Godavari delta,
0.60—fine silt as at Jamrao canal, Sindh,
0 60 to 0.70—for Sarda canal,
0.62—for Rohree canals taking off from the river Indus,
0.70—fine silt as at Krishna, Western delta type,
0.80—average for Punjab canals,
0.85—medium silt as at Ganges canal distributaries,
0.90—for Ganges at Sara.

(Take 0.90 f in head reaches and 0.80 f in tail reaches,) Channels desiged from Lacey's diagrams for f=0.8 will be smaller than those designed with Kennedy's diagrams with N=0.0225 for the same slope and discharge.

Use of Lacey's Diagrams—Reproduced from Lacey's original Paper in F.P.S. units.

For known values of discharge Q and silt factor "f" the wetted perimeter and the hydraulic mean depth are calculated. These are converted into bed widths and depths for a corresponding trapezoidal channel with horizontal bed and 1/2 : 1 side slope. This section is the one most commonly used in the condition of irrigation channels. From the two diagrams slope and dimensions of any channel can be determined if discharge and "f" are known. (Where channels are designed from Lacey's diagrams, there is no need to work out the critical veloci'' ratio).





Type of Soil	Fine silt	Sandy silt	Coarse sand	Gravel	Boulders
Grain size mm	0.12	0.32	0.72	7.28	72.5
"f"	0.60	1.0	1.5	4.75	15.0

Procedure for Design of Channels with Lacey's Theory :

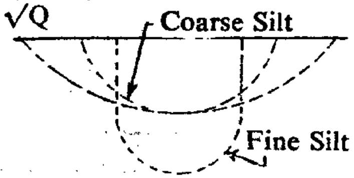
Velocity of flow $V=\left(\frac{Q f^2}{140}\right)^{\frac{1}{6}}$

Hydraulie mean depth $R=2.5\left(\frac{V^2}{f}\right)$—is affected by silt grade.

Wetted Perimeter P of the channel $= 4.75\sqrt{Q}$

Area A of the channel $=\frac{Q}{V}$

Longitudinal bed slope $S=\frac{f^{\frac{5}{3}}}{3340\ Q^{\frac{1}{6}}}$



Shape of Regime Channel

For a trapezoidal channel with 1/2 : 1 side slope $P=b+\sqrt{5}\times d$

$A=bd+\frac{d^2}{2}=\frac{Q}{V}$

From known values of A and P, bed width "b" and depth "d" can be calculated :

$b= P-2.236\ d$

3. GAUGING VELOCITY AND DISCHARGE OF RIVERS AND CHANNELS

For observing verlocity and discharge of rivers and channels a straight and uniform reach is selected and a cross-section in the middle of it is taken. This cross-section is divided into suitable compartments and the mean velocity of each compartment is found. If desired the velocities at different depths can be taken and the mean velocity worked out. A stop-watch is used for timing. Take velocity observations when there is least wind.

Selection of Site for Discharge Observations :

The site selected should comply with the following regulations as far as possible to minimize errors caused by irregularities in the motion of water. The site or the section under observation should :—

(a) Be on a straight run and not on a curve, or a fall, and must be clear of weeds, projections or any other obstructions interfering with smooth flow. This straight run length should be not less than 10 times

the mean width in the case of a canal, and at least for 150 m above the section line for rivers in hills and which will vary for rivers in the plains as described hereafter.

(b) Be regular and uniform, not showing much departure from its normal flow at different times of the year.

(c) Be at right angles to the direction of flow.

(d) Be easily accessible.

(e) Not be located where the river is too wide and shallow or too narrow and deep.

The site should not be located above a weir or barrage within the effect of the pond formed due to the heading up of the river, nor near a bridge or any snch masonry work which is likely to cause obstruction to the smooth flow of the water. For a permanent discharge site in the case of canals, it is desirable that the sides and bed be lined or pitched for a length of about 60 m.

Segmentation or Spacing of Sounding Points :

The distance between the sounding points depends upon the width of the stream, profile of the bed and the accuracy desired. Two ropes or cables (and sometimes a third between the two) are stretched across the channel marking the distance of the "run". A cable can be used only for channels up to about 300 m width. For gauging rivers where the distance is great and it is not possible to fix a rope across it, measurements are done by triangulation and observations taken with the help of a theodolite from the banks. Allowance must be made for the sag in the rope or cable. The width is divided into several segments or compartments of more or less of equal discharge, which may be up to 8 or 10 in the case of channels and 15 for wide rivers. The width of the segments usually vary between 1 and 30 m according to the size of the channel or the river and the accuracy desired. In the case of regular canals, the cross-section should be divided into separate slope and central segments which should be further sub-divided into smaller segments. For river widths of abour 250 to 500 m segments are 15 m apart and beyond that width 30 m apart over the portion of the section passing 75 per cent of the total discharge and 60 m apart for the remainder. A boat is useful for velocities of about 150 to 180 cm only; for higher velocities, anchors should be used for 90 m lengths, or a motor launch if available.

Tags or pendants are tied at all such intervals with one tag coming over the extreme water-edge of each bank. These tags mark the width of the section and facilitate in counting. The length of a float run is so fixed that a float should cross it in not less than 20 seconds and not more than 1-1/2 minutes. The length usually taken is 8 to 60 m for canals and 15 m for rivers. In the case of large rivers the upper gauge fixed two river widths above the discharge section and the lower gauge one river width below the discharge section.

Where gauges are fixed on a sloping wall, the markings should be elongated so as to give readings corresponding to vertical differences.

Measuring Depths. Depths are measured along the cross-section made out, at the start, the middle, and at the end of the run in the case where velocity rods or surface floats are used and along the central cross-sections only where a velocity meter is used. Depths are taken below the dividing marks and the middle of each compartment. Mean of the readings is taken.

Sounding Rod or Pole. An oval section for a sounding rod is considered to be the best as there is less heading up of the water and it gives more accurate readings. Flat iron of 50 x 6 mm size or a bamboo of 50 to 75 mm diameter instead of an oval wooden rod are also used. The rod is graduated in tenths of metre and bottom provided with a flat or round piece of wood or iron of size 10 to 15 cm to prevent its sinking in a soft bed. The depths should be measured at the downstream ends thus omitting effects of afflux due to velocity. Poles are generally used if the depth of water is less than 3 m, but bamboo rods have been used even up to 9 m depths in low velocities.

Log Line : The weight and cord attached to it used for determining depths at observation points where it is impossible to use a sounding rod.

Hook Gauge : A pointed hook attached to a graduated staff or vernier scale is used for measuring the elevation of the surface of still water. The hook is submerged, and then raised until the point makes a pimple on the water surface.

Gauge Line : The line across a channel, passing through the permanent gauge in a fixed direction.

Sounding Cable. Is used for depths below 3 m. It is usually wire or a hemp cord graduated into parts by tags of leather inserted between the strands of cords. A hemp cord is not very accurate as it will shrink when wet and stretch under weight, therefore, due precautions should be taken when using it. Copper cores covered with hemp (called log lines) if available, will not shrink or stretch. A weight of about 2 to 5 kg is required for canals with velocities up to 120 cm per sec. and in small depths. For higher velocities and greater depths, a weight up to about 23 kg may be necessary. The load sounding weight is generally of the shape of frustum of a cone. It requires experience to observe depths correctly as the weight after touching the bed trails down and and the rope has to be pulled up till the weight is vertically under the observer.

Observing Velocities

Variations in Velocity in a Cross Section of a Channel

The Surface and Mean Velocities : The velocities at different points of cross-sections of a channel differ widely and a mean velocity for the whole cross-section (or a compartment) has to be computed. The surface velocity of a stream is higher than the mean velocity; the

mean velocity is about 85 to 95 per cent of the surface velocity and the mean velocity is generally taken to occur at about 0.6 D from the surface. The velocity is least in the neighbourhood of the bed and the banks and greatest in the axis of the stream at about 0.15 D to 0.3 D below the surface The ratio of surface velocity to mean velocity is very variable and depends upon the form of the channel cross-section, the depth of water, and the roughness of the sides and bottom. The value is stated to be greater for sandy bed rivers and minimum in the beds of gravel. It has been observed by some engineers that mean of the velocities at 0.8 and 0.2 of the depth from the surface gives mean velocity involving error up to 2-1/2 per cent only.

In a branch canal of over 15 m bed-width the mean velocity in the central segment has been found to be just double the mean velocity of the sloped segment ; on small channels this ratio reduces to 1-1/2.

The following co-efficients are recommended by Buckley :—

Depth	Co-efficient
Surface	0.85
1/10 depth	0.89
2/10 ,,	0.91
3/10 ,,	0.92
4/10 ,,	0.95
5/10 ,,	0 98
6/10 ,,	1.002
7/10 ,,	1.046
8/10 ,,	1.102
9/10 ,,	1.174
Bottom	1.339

Observed velocity at certain depth multiplied by the co-efficient gives the mean velocity. (Buckley's Pocket Book.)

(Results of practical observations)

In a channel velocity varies from point to point, the mean velocity generally taken is as follows :—

Major distributary	0.70×central surface velocity
Minor ,,	0.65×central surface velocity
Water-course ,,	0.60×central surface velocity

Surface Floats. Wooden discs 75 to 150 mm or wooden blocks, hollow metal cylinders or even corked bottles, with small flags fixed on top for identification are used. Globular floats are better than flat discs. A double float is better and more accurate ; the lower float is larger and a little heavier than water ; the length of the cord between the two floats is kept equal to the depth of the point at which the velocity is required. A float registers the surface velocity which is higher than the mean velocity, and a reduction co-efficient has to be applied to get the mean velocity. Experiments on different rivers have shown the value to be between 0.79 to 0.91. The relation of mean, surface and bottom velocities is sometimes expressed by the relation :

$$V = 0.85V_s = 1.34V_b$$

The method of determining velocity by floats is not very accurate and is considered to err about 15 per cent in normal weather and which may be much more if the river flows with or against prevailing winds of considerable velocity.

A chord is tagged about 6 m above the starting point of the float run from which floats are released. When the line is great and it is not possible to stretch a cord without sagging, the line may be marked by another set of poles.

Velocities are observed at the middle of each section excepting in the case of end sections which are triangular and are measured at 2/3rd of the width of the triangle section from the edge. A length of 30 m will be sufficient for most of the channels.

Velocity Rods are straight wooden rods or hollow tin tubes 25 to 50 mm dia., of a uniform section and of varying lengths according to the depth of water, weighed at the bottom so as to float in a vertical position. A velocity rod should float 7.5 to 1.5 cm short of the bottom and about 4 to 5 cm above the water surface.

A rod is made of the same length as the depth of water and so weighed that the immersed length is 0.94 of the depth and is 0.06 above the water surface. A velocity rod of this length will give the mean velocity of the channel. But very often rods of shorter lengths have to be employed because of obstructions in the bed of the channel. The following correction factors are suggested by Francis to be applied in order to correct a rod velocity to mean velocity :—

l/d	0.75	0.80	0.85	0.90	0.93	0.95	0.96	0.97	0.98	0.99
$\frac{V\ mean}{V\ rod}$	0.954	0.961	0.968	0.975	0.981	0.986	0.989	0.992	0.996	1.00

l is the length of the submerged portion of the rod used and d is the mean depth of water : Multiply the rod velocity by the above factor to abtain mean velocity.

If after recording three results one of them differs by more than 5 per cent from the mean of the other two, it should be rejected and the run repeated. Laycey's telescopic discharee rod is sometimes used. This type of rod is a closed hollow tube made of tin and works inside another hollow tube which is weighed at its bottom by lead weights so as to make it run vertical in water with the top keeping about 5 cm above the water level. The length is adjustable and can be extended up to 4 m.

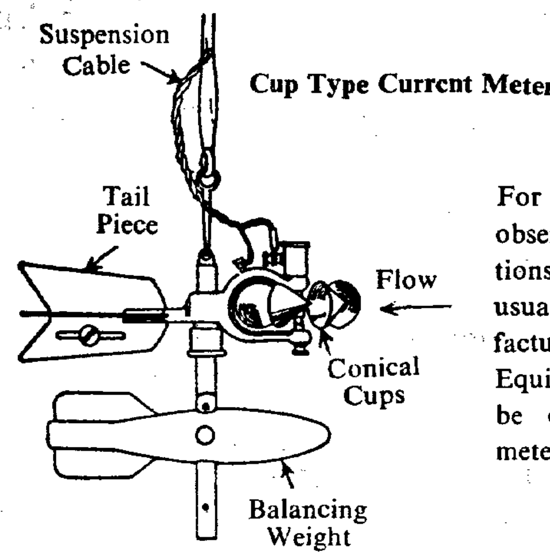
For dropping the rod into the stream (at some distance above the first rope) it is held horizontally in the hand with its bottom end (which is heavier) pointing upstream. The bottom end is lowered into the water and the rod is allowed to swing from horizontal position to vertical until the top portion (which is to stay above water level) comes above the surface of water. The rod should be released with a slight forward push so that it immediately gets into the vertical position and floats along steadily. The rod should have adopted a state of equilibrium and should float smoothly along with the current before reaching the first observation point. The rod travels with a speed equal to the mean velocity of the section. At least 3 to 5 rods must be run for each section and

their mean taken. A number of rods of different lengths are necessary. All time observations should preferably be taken by one man.

The rod method of observing velocity is very easy, simple, convenient, quite suitable and reliable for depths up to 1.5 m and for channels of regular sections. (Rods may be used for depths up to 4.5 m when any other more accurate instrument is not available). This method does not err more than about 5 per cent if carefully done. If there are weeds in the channel, rods will not give accurate results. The method of rod floats is applied only to canal observations. The advantage of velocity rods over surface floats is that the rods give approximately mean velocity and are not so much effected by wind.

Current Meters

There are two types—(i) Direct Action Meters (Screw or Propeller) and (ii) Differential Action (Cup Meters). A simple method of observing with a current meter is to lower the meter (fixed to a rod) from a boat or a launch in the centre of each compartment at the middle of gauge run and at a depth from the surface equal to 0.6 depth of the channel at that point, with its wheel facing the current of water—this involves errors up to 6 per cent. For more accurate results velocity is taken at intervals of 30 to 60 cm all along the depth at each section and averaged. Three or four observations should be taken of each section. For making velocity measurements with current meters the channel is divided into a number of sections varying from 0 6 m to 6 m according to its width. The number of revolutions of the wheel of the meter per second are worked out. This when multiplied by the factor given for (or rating) the instrument, will give the velocity. The factor can be worked out by moving the meter in still water at various known speeds.



For the use of current meters, observance of certain precautions are essential which are usually provided by the manufacturers with the instruments. Equipments listed should also be checked. Price's current meter is most commonly used.

Discharge through Contracted Rectangular Weir in Litres/Sec.

H cm	Width of Crest of Weir in Centimetres 25	30	40	50	60	75	100	135	150	175	200
1.0	0.5	—	—	0.9	—	1.4	1.8	2.2	2.7	3.2	3.6
1.5	0.8	—	—	1.7	—	2.5	3.3	4.2	5.0	5.9	6.7
2.0	1.3	—	—	2.6	—	3.9	5.1	6.4	7.7	9.0	10.0
3.0	2.3	—	—	4.7	—	7.1	9.5	12.0	14.0	17.0	19.0
4.0	3.6	—	—	7.2	—	11.0	14.5	18.0	22.0	26.0	29.0
5.0	4.9	6.0	8.0	10.1	12.2	15.0	20.0	25.0	31.0	36.0	41.0
5.5	—	6.9	9.3	11.6	14.0	—	—	—	—	—	—
6.0	6.4	7.8	10.5	13.1	15.9	20.0	27.0	33.0	40.0	47.0	54.0
6.5		8.4	11.8	14.9	17.9						
7.0		9.7	13.2	16.6	20.0						
7.5		10.7	14.5	18.3	22.1						
8.0	9.7	11.8	16.0	20.1	24.3	30.5	41.0	51.0	62.0	72.0	83.0
8.5		12.9	17.6	22.1	26.7						
9.0		14.0	19.0	24.0	28.9						
9.5		15.2	20.7	26.0	31.2						
10.0	13.0	16.3	22.2	28.0	33.8	42.0	57.0	72.0	86.0	100	115
10.5		17.5	23.7	30.0	36.2						
11.0		18.7	25.3	32.0	37.7						
11.5		19.9	27.1	34.3	41.4						
12.0	17.0	21.3	29.0	36.7	44.4	56.0	75.0	94.0	113	132	151
12.5		22.5	30.7	39.0	47.1						
13.0		23.7	32.3	40.9	49.5						
13.5		24.8	34.0	43.0	52.2						
14.0	21.0	26.2	35.8	45.4	52.2	70.0	94.0	118	142	166	190
14.5		27.7	37.9	48.2	58.5						
15.0		28.8	39.5	50.3	60.9						
16.0		31.6	43.3	55.2	67.0	85.0	114	143	173	202	232
17.0		34.3	47.2	60.1	73.0						
18.0		37.0	51.0	65.3	79.0	100	135	170	206	241	276
19.0		39.8	55.0	70.2	85.3						
20.0		42.8	59.3	76.0	88.8	117	158	199	240	281	322
21.0		45.7	63.3	81.0	99.0						
22.0		48.7	67.5	86.7	105.7						
23.0		51.3	71.7	92.2	112.3						
24.0		54.7	76.5	94.8	120.0						
25.0		57.0	79.8	102.7	125.8	161	218	276	333	391	448
26.0		60.3	84.6	109.2	133.3						
27.0		63.5	89.2	115.0	140.8						
28.0		66.5	93.7	122.2	148.3						
29.0		69.5	98.3	127.0	155.7						
30.0		72.5	103	133.0	163.3	208	284	359	435	511	586
35.0							354	449	544	640	735
40.0							428	544	660	777	893
45.0							505	644	783	921	
50.0							585	747	910		

H is head over Weir
See Section 14 "Hydraulics"

Discharge through Circular Orifice in Litres/Sec.

H cm	Diameter of Orifice-cm 2.5	5.0	7.5	H cm	Diameter of Orifice-cm 2.5	5.0	7.5
1.0	0.13	0.53	1.2	16.0	0.53	2.10	4.80
1.5	0.16	0.64	1.4	16.5	0.54	2.15	4.83
2.0	0.19	0.74	1.7	17.0	0.54	2.20	4.92
2.5	0.21	0.81	1.8	17.5	0.55	2.22	5.00
3.0	0.23	0.91	2.1	18.0	0.56	2.25	5.13
3.5	0.25	0.99	2.2	18.5	0.57	2.30	5.15
4.0	0.26	1.15	2.4	19.0	0.58	2.31	5.20
4.5	0.28	1.20	2.5	19.5	0.58	2.33	5.30
5.0	0.30	1.21	2.7	20.0	0.59	2.37	5.32
5.5	0.31	1.23	2.8	20.5	0.60	2.40	5.40
6.0	0.32	1.30	2.9	21.0	0.60	2.42	5.45
6.5	0.33	1.34	3.0	21.5	0.61	2.45	5.53
7.0	0.35	1.39	3.1	22.0	0.62	2.50	5.60
7.5	0.36	1.45	3.3	22.5	0.63	2.51	5.65
8.0	0.38	1.50	3.4	23.0	0.63	2.53	5.70
8.5	0.39	1.53	3.5	23.5	0.64	2.57	5.77
9.0	0.40	1.60	3.6	24.0	0.65	2.60	5.83
9.5	0.41	1.62	3.7	24.5	0.66	2.62	5.92
10.0	0.42	1.70	3.8	25.0	0.66	2.63	5.95
10.5	0.43	1.72	3.9	25.5	0.66	2.65	6.00
11.0	0.44	1.75	3.9	26.0	0.67	2.67	6.10
11.5	0.45	1.80	4.0	26.5	0.68	2.70	6.12
12.0	0.46	1.83	4.1	27.0	0.69	2.75	6.18
12.5	0.47	1.87	4.2	27.5	0.69	2.80	6.23
13.0	0.48	1.90	4.3	28.0	0.70	2.81	6.30
13.5	0.49	1.92	4.4	28.5	0.71	2.82	6.37
14.0	0.50	1.94	4.5	29.0	0.71	2.83	6.40
14.5	0.50	1.96	4.5	29.5	0.72	2.87	6.47
15.0	0.51	1.98	4.6	30.0	0.72	2.90	6.53

H is head of water over centre of orifice. See Section 14 "Hydraulics"

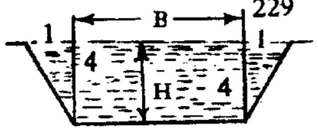
Discharge through 90 deg. V-Notch in Litres/Sec.

H cm	Dis. L/S	H cm	Dis. L/S	H cm	Dis. L/S	H cm	Dis. L/S	H cm	Dis. L/S	H cm	Dis. L/S
7.0	1.8	11.0	5.7	15.0	12.3	19.0	22.3	23.0	35.7	27.0	53.8
7.5	2.2	11.5	6.3	15.5	13.3	19.5	23.5	23.5	38.2	27.5	56.3
8.0	2.5	12.0	7.1	16.0	14.5	20.0	25.5	24.0	40.0	28.0	58.7
8.5	2.8	12.5	7.8	16.5	15.6	20.5	27.0	24.5	42.7	28.5	61.5
9.0	3.4	13.0	8.6	17.0	16.7	21.0	28.3	25.0	44.5	29.0	64.5
9.5	3.9	13.5	9.5	17.5	18.3	21.5	30.3	25.5	46.7	29.5	66.8
10.0	4.5	14.0	10.5	18.0	19.4	22.0	31.0	26.0	48.8	30.0	69.4
10.5	5.1	14.5	11.3	18.5	21.7	22.5	34.0	26.5	51.0		

H is height of water.

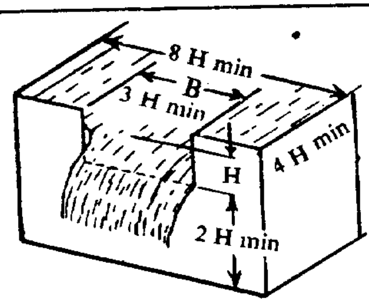
Discharge through Standard Cipolletti Trapezoidal Weir in Litres/Sec.

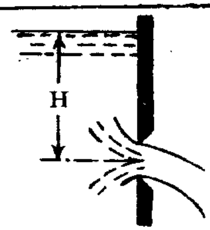
H cm	Width of Crest of Weir (B) in Centimetres 25	50	75	100	125	150	175	200
1	0.5	0.9	1.4	1.8	2.3	2.7	3.2	3.7
2	1.3	2.6	3.9	5.2	6.5	7.8	9.2	11
3	2.4	4.8	7.2	9.6	12	14	17	19
4	3.7	7.4	11	15	19	22	26	30
5	5.2	10	16	21	26	31	36	42
6	6.8	14	21	27	34	41	48	55
8	10.5	21	32	42	53	63	74	84
10	15.0	29	44	59	73	88	103	118
12	19.0	39	58	70	97	116	135	155
14	24	49	73	97	122	146	170	195
16		60	89	119	149	178	208	238
18		71	106	142	177	212	248	284
20		83	125	166	208	249	291	333
25			174	232	290	349	407	465
30			229	305	382	458	535	611
35				391	481	577	674	770
40				477	588	705	823	941
45				571	701	842	982	1122
50					822	986	1150	1314



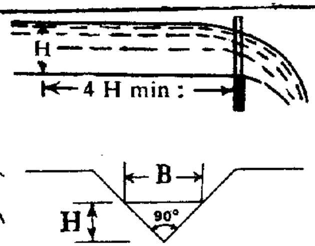
The conditions for measurement for a rectangular weir also apply to a cipolletti weir. The standard side slope is 1 horizontal to 4 vertical. Discharge is calculated from the formula : H is head over Weir.

$Q=1.86\ BH^{3/2}$ in cu.m/sec. See Section 14—"Hydraulies" and page 102

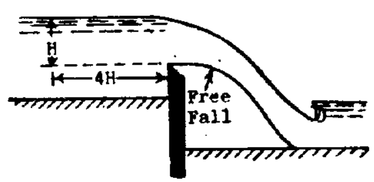




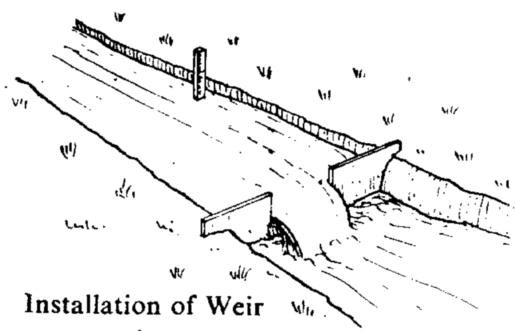
Circular Orifice



90° V-Notch



Rectangular Weir with End Contractions



Installation of Weir

Most of the Irrigation Departments have their own standardized and officially approved discharge tables for outlets, but which may not give correct results in all cases.

The above will indicate that if the outlet was just working as a free fall its discharge can be increased by allowing the water-course to silt up and thus drown the outlet. After a certain stage, however, the increase in the value of the co-efficient would be compensated by a decrease in the working head and further silting up would reduce the discharge through the outlet.

For fixing of discharge from a pipe outlet, as the depth of water in the supply channel is variable, two-third or even one-half of the full supply discharge is taken for calculating the diameter of the pipe.

Where short pipes are used to build up the barrel, the sockets should preferably be laid facing the downstream end against the usual convention of sockets facing against the flow, otherwise the front socket will act as a bell mouth. The projection of the upstream end of the pipe in front of the masonry has no appreciable effect on the co-efficients of discharge. (Also see Page 17/93)

5. EARTHEN EMBANKMENTS & DAMS

Before commencement of work on all large embankments, the centre line should be marked by pegs at every chain, curves properly laid out, the top and bottom edges of the excavations and the toe of all the embankments and spoil banks clearly lock-spitted. A complete profile should be set up on every 150 m of distance and at every change of section. This profile should be a 3 m length of the actual completed channel or embankment with the correct heights, width and all slopes dressed to true form. The seat of the embankment should be prepared to receive the new earth and for which the whole site must be cleared of all trees, roots, shrubs, grass which might decay and form dangerous pockets. All loose surface or soft soil should be removed to about 15 cm depth and the surface roughed by ploughing or digging all over. Small key trenches should be dug out in the bed to unite the body of the new embankment to the sub-soil. Or, the land may be prepared by cutting V—shaped benchings at intervals running parralel to the centre line. A *key trench* is very essential where the ground is porous, sandy or is cracked. If only one trench is made, its bottom width may vary from 1.2 to 1.8 m depth 0.9 to 1.5 m with usual side slopes. If a number of small trenches are made, their sides may be vertical, or in saw-tooth shape. All soft soil should be removed as far as possible, especially soil containing salts. Also see Page 101, 102

For an embankment or a dam to be stable, its foundations must be strong enough to stand the enormous weight of the dam; slopes should be made as flat as practicable, so that the shear stress produced in the foundation is less than the shear strength of the foundation material. The lower portions of high earthen dams are highly compressed, therefore need special care. To increase cohesion and friction at the bottom of such dams for stability, moisture content should be reduced. Coarse sands and gravels in the foundations of earthen dams give no trouble with regard to stability of the foundations because even though they may not be consolidated, they will promptly consolidate as the load is applied. Caution is necessary where the foundation material consists of silt, fine uniform sand, or clay-type materials. A plastic clay foundation is the worst type.

For all important dams and reservoirs it is very essential to explore the foundation strata to ascertain the exact properties of the soil materials underneath and, not only its pressure bearing capacity but also the watertightness of the floor of the reservoir under the weight of water that the dam will impound. Soil should be tested for cohesion, moisture content, liquid limit, permeability, shear and compression. A mattress of cement concrete may have to be provided all over the base of the reservoir or the dam in case of unstable soils.

The permeability of the bed can be tested by digging small pits about 90 cm deep and filling them with water. If the surface soil is pervious and the sub-soil is relied on to retain the water in the tank, the pits should be excavated down the sub-soil level and their sides lined with clay so that all percolation loss will take place through the bottom of the pits. In all cases the pits should be kept full of water for not less than 24 hours before commencing to observe the rate of loss, to saturate the surrounding soil.

The soil of which the embankment or the dam is to be built is another most important factor for the stability of the structure which depends upon the shear strength of the particles of the soil and this in turn is due to either cohesion on friction of the particles, or both. Although sand and gravel are good for resisting friction and shear they are non-adhesive and cannot be used independently for an embankment, therefore, clay must be added to bind them together; if the soil is clayey, sand must be added in sufficient proportion. Clay swells when wetted and shrinks or cracks in drying. Too much sand results in too high a permeability. To be stable, the embankment must be water-tight and non-slipping. A soil containing proper proportions of sand, silt and clay will, of caurse, form a most stable structure, but such an ideal soil is seldom available. A dam of homogeneous material throghout should be built, as far as possible.

Testing Soils for Embankments. The quality of a soil to resist saturation by water may be tested by making balls of the soil about 10 to 12 cm in diameter and immersing them in still water 45 to 60 cm deep. A good soil will resist saturation for days while an inferior soil will fall to pieces in a few hours. The soil should be damped and kneaded up in the hands until it becomes a stiff plastic mass before making into balls. It is useless testing a dry lump of plastic soil by

immersion in water as it will readily absorb water and fall to pieces. For important works, the qualities of any soil may be tested by making small square tank say of a about 3 m sides with the proposed side slopes and filling it with water which is let to remain for about a week, any percolation losses being made good at intervals. The actual slope of the percolation plane in the banks can be determined and by gradually cutting back the outer slope the qualities of the soil as regards slipping when saturated may be observed.

Stabilized Soils for Embankments

If a sufficient quantity of retentive earth or gravel containing enough clay (25 to 30 per cent), or moorum which will effect required water tightness when consolidated, is available, the dam can be made without a core wall. 25 to 35 per cent of gravel or sand and the remaining of earth or soil, or one part of pure black cotton soil or other such clayey soil to one part of gravel will also make a good embankment. Black earth is generally impervious and plastic while yellow earth is generally sandy.

The following proportion of materials will make a waterproof dam :—

Sand (0.02 to 2.0 mm)	...	60 to 80 per cent by weight
Silt (0.002 to 0.02 mm)	...	12 to 25 ,,
Clay (below 0.002 mm)	...	8 to 15 ,,

The above proportions will "stabilize soil". "If natural soils confirming to the above limits are not available and blending of soils has to be resorted to the following specifications may be used :—

Plasticity Index—8.5 to 12.0

Sand content not less than 35 per cent by weight (sand being the fraction retained on 500 micron sieve). If 5 per cent cement is added, blocks can be made (See under "Stabilized Soils for Building Construction in Section 7.)

It will usually make a good stabilized soil with 70 per cent sand and 30 per cent clay (or silt and clay together). If aggregate is used, the proportions are :—

Sand	50 per cent	The aggregate should be 12 mm downwards well graded.
Clay	17 per cent	
Aggregate	33 per cent	

From 45 to 60 per cent sand should be retained on 250 micron sieve. The clay content should be more in dry areas and less in wet areas.

Coarse gravel	60 per cent	To test the mixture : Ram it moist into a bucket and then ascertain if it will remain in the bucket when turned upside down.
Fine gravel	20 per cent	
Sand	8 per cent	
Clay	12 per cent	

If the whole of the embankment cannot be made with the above stabilizing mixtures, at least the core wall should be so made.

This subject (Stabilized Soil) has been treated in detail under "Soil Mechanics" and "Roads.")

The more and the less pervious materials must not be so distributed generally and locally, in layers that water will be trapped so to develop uplift or bursting pressure or seepage.

Clay soil in a confined state is not only highly impermeable but is also very retentive of the water absorbed and pure clay in the heart of a big embankment when once in a suitable plastic condition will retain moisture and remain plastic in a very dry climate for a year or two even if no water has been standing against the bank for such a period.

Earth for embankments and dams should not be taken from salt affected areas; peat or other soils which become soft and fluffy during winter must not be used. Material containing organic matter, humus, tree roots, etc., is not to be used. Avoid grassless areas for making borrow pits. The earth selected for hearting, (central 1/3 to 1/2 of the cross section) and more especially for the water side, should be the best earth available suitable for tile making, and sufficiently rich in clay so as to be highly impervious but not so rich as to crack on exposure to the sun. As far as possible all the earth should be of the same quality of material throughout. The more sandy or stony soil should be used for outer side of the embankment.

Compaction of Embankments. The object of compacting soils is to improve their properties as regards strength, liability to settlement and resistance to weathering. All earthen embankments should be thrown up in layers not exceeding 23 to 30 cm (depending on the hardness of the soil and the weight of rollers available for consolidation) stretching right across the whole section. On no account must an embankment be originally made of less than the full width with a view to widening subsequently. When commencing work it is desirable to take earth first from the more distant pits gradually lessening the lead as the work rises, so that all earth is thrown into the slope and not tipped over. Side slopes should be carried up simultaneously with the rest of the work and not filled in afterwards.

All large clods should be broken up in the borrow pits and no clods larger than a man's fist should be brought on the banks. Ramming is not enough for crushing the large clods completely, which can be done effectively only by heavy rollers. Each layer should be rolled well until all clods are flattened. Any roots, grass, jungle or other rubbish should not be buried in the banks along with the earth. The width of each layer should be a little in excess of the width required by the cross-section of the bank and the slopes are then dressed off to the final section. Each layer should be laid with a slight slope of about 1 in 12 towards the centre making a concave curve. This allows for some rain-water to to be held up and help the bank to settle down quicker, and it also prevents the side slopes from being washed down. If the ramming is to be done by manual labour, the layers should not exceed 75 mm in thickness. Compaction of banks can also be done by camels or bullocks in which case the layers of earth may be 150 to 200 mm.

Suitability of Rollers, and Rolling

Sheepsfoot rollers are suitable only for compacting dry cohesive soils at low moisture contents. Pneumatic tyred rollers are most suitable machines for compacting soils in embankments. Smooth-wheeled rollers are satisfactory for most cases of sub-grade and base compaction. In confined areas, rammers for clay soils and vibrating machines for granular soils are most convenient.

The organization of filling, spreading and rolling should be such that newly deposited fill is spread and rolled smooth immediately in order to minimize the loss of moisture. Rolling on a dry earth layer is unless. earth can be compacted by sheepsfoot rollers which break down and spread the large soil lumps. The fill should then be completed by smooth rollers, giving about eight complete coverage passes per layer. It is sometimes advisable to pass a light roller over a newly-spread layer in order to bring it to a level surface, before working a heavy roller. To prevent the materials sticking to the rollers, dry earth should be sprinkled on the surface before or during consolidation, if necessary. No watering should be allowed until the layer has been completely rolled. Flooding with water to effect compaction of the fill is a bad practice. Water should be sprinkled over the rammed layer before the next is spread for the two layers to adhere. Consolidation is done at "optimum moisture content." "Rolling" has been described in detail in the Section "Roads and Highways".

The following allowance per metre (of loose material) in height should be made for shrinkage or settlement of earthwork :—

Soft or loose rock, laterite or gravel	8 cm
Firm compact earth	12 cm
Ordinary loose earth	16 cm
Black cotton soil	24 cm

Some engineers prescribe the following specifications for settlement of channel banks and earthen dams :—

Make an allowance per metre of vertical height of 7.5 cm on all works rolled and watered, 15 cm on all works rolled but not watered, and 23 cm on all. works neither rolled not watered provided works which have passed through one monsoon shall be considered to have attained to 50 per cent, through two monsoons to 75 per cent, and through three monsoons to 8.5 per cent, of its final settlement.

No matter how well an embankment has been consolidated it will keep on settling for some years due to its enormous weight and the rainfalls. The total vertical settlement of a well consolidated embankment is about 1/30 of its height.

The process of compacting soil involves the packing of the soil particles, and the degree of compaction is measured by the bulk density (see under "Soil Mechanics") of the solids in the soil. With a given amount of ramming or rolling, there exists for every soil an optimum moisture content at which this compaction will produce a maximum density.

Suggested Method of Embankment Compaction with Various Plants

Plant	Silt and heavy clays	Sandy soil	Gravel, shingle or loose sand	Fragmental rock (not over 10 cm size
	Thickness of layers			
8-ton sheepsfoot roller or tractor, dumper, etc.	15 to 20 cm	...	Loose sand should be damped	
5-ton smooth roller	...	20 to 30 cm		
Tractor equipment	...	...	30 cm	
Heavy smooth rollers	...	...	...	30 to 60 cm

(Also see similar table in the Section "Roads and Highways" under Rolling.)

On all important embankments it is desirable to ascertain by compaction tests the densities of the soil that should be obtained in construction.

In banks over 1m in height in cotton soils a topping of moorum or other suitable water-proofing soil must be laid to at least a third of the height and continued over the side slopes. Before the commencement of monsoons continuous longitudinal earth bunds about 23 cm in height and 30 cm wide on the top with side slopes of 2 to 1 may be made on the outer edges of the top embankment, also cross bunds of the same dimensions at every 7.5 m so as to impound rain water to expedite consolidation.

Stabilizing Side Slopes against Slipping. Side slopes of embankments depend upon the nature of the material used, stability of the material under conditions of saturation and its resistance to percolation of water and to slipping ; bearing power of the foundations is also considered. The stability of earth depends on the case and rapidity with which it can be drained of all superfluous water ; slightly damp earth is stable while saturated earth is very unstable. Sometimes shallow herringbone drains are provided to trap surface water running down the slope and lead it to the deep counterfort drains.

Low dams can be constructed with much steeper slopes than high dams ; for heights above 2.5 m slopes should be flatter. Slopes are made flatter than the natural angle of repose of the material and the upstream portion, unless protected by pavement, being laid flatter than the angle of repose of the material when wet (under water). (Saturated earth assumes much flatter slope than when dry). Cohesive soils yield and slip under pressure when saturated. A flatter slope is necessary on the rear side to keep the earthwork above the "line of saturation" (described hereafter). A bank is considered to be safe if the saturation line is covered by at least 60 cm of soil depending upon the nature of the material. Some engineers consider that tank-bunds should be designed of such dimensions as to ensure not less than 90 to 150 mm of material on the outer slope vertically above the hydraulic gradient line. (See under "Hydraulic Gradient" and the Illustration showing "Typical Cross-sections for Channels") described in the pages following.

Stability of slopes can be increased by the construction of berms of hard material, or hand-packed stone filling along the toe of the slope. The removal of the soil from the top of the slope, in the form of a benching, or lessening the gradient of the slope will also improve stability. Sheet piling driven into the ground near the toe of the slope will also increase the factor of safety against slipping. Vegetation has been successfully employed in stabilizing slopes; apart from reducing the moisture content of the soil, the network of roots acts as a reinforcement in the soil. See under "Stability of Bank Slopes"—Retaining Walls in Section 7.

Suitable Side Slopes in Various Kinds of Soils : see page 62

If a layer of stiff or hard soil (of appropriate thickness) is put on a soft or loose soil, a steeper slope can be managed. Slopes can also be made steeper by providing stone pitching. Where it is proposed to protect the slopes with turfing, a top layer of suitable soil (favourable for the growth of grass) should be laid. In most of the soils, bank slopes of 3 : 1 for the lower slopes and 2 : 1 for the upper slopes will be found suitable. Under favourable conditions of saturation and draw-down 2 : 1 slope for low embankments should suffice but for sandy soils, even 4 : 1 may not be sufficiently flat.

Hydraulic Gradient ; Saturation Gradient ; Percolation Gradient ; Line of Saturation or Seepage and Percolation Line are all synonymous terms. It is a line inside an embankment marking the boundary between wet earth and damp or dry earth.

In the case of earthen embankments which hold up a depth of water against one face, the bank becomes gradually saturated by percolation up to a certain level constituting a gradient or an inclined line falling from the point where the water touches the embankments on the upstream face. This inclined line is called the Hydraulic Gradient for that soil and below which the embankment portion is saturated. This is due to the pressure of water, and the more the soil is porous the less is the resistance to percolation, and the flatter the hydraulic gradient (or, more water-tight the materia e steeper will be the line of satura-tion).

The plane of the surface of colation water is called the *plane of saturation or percolation* and if cuts the outer face of the bank, visible flow will appear along and ow the line of intersection. The hydraulic grade line must fall with the toe of the bank and be covered by at least 60 cm of soil and whic hould be much more in the case of river embankments. Rear berms o .5 m wide may be provided to give adequate cover to the seepage ne. The hydraulic gradient can be observed by installing a small pipe with strainer, of say, 50 mm dia., vertically in the embankment, it w have water rising up to this line. The relative permeability of differ kinds of soils may be compared by observing the respective percolati gradients under the same conditions. The velocity of percolation is very all when compared with surface flow and varies with the permeabi of the soil being proportionately greater in a coarse soil than in a one.

Core walls of puddle-clay etc. duce the seepage water and prevent the flow lines from cutting the do stream face of the embankment. Sometimes filter materials are pla on the downstream toe to provide free drainage, which will force the epage lines down. Filter materials can be placed in several layers, ea layer coarser than the last.

The hydraulic grade line is erally as follows :—

For good clay ... ...1 in 3
,, good compacted soils ... ...1 in 4
,, average soils (sandy lo) ...1 in 5
,, bad soils ... ...1 in 6
,, fine silt ... ...1 in 6
,, fine sand ... ...1 in 8
,, coarse sand ... ...1 in 10

For soils in the Punjab : (f anal banks)

Where full supply level is up to m above natural surface : 1 in 4
,, 1.2 to 1.5 m ,, : 1 in 5
,, 1.5 m or more ,, : 1 in 6

If the foundation of a dam an embankment is permeable, water will percolate under the structur hich will be steeper as the head is greater. To remedy this, the ba idth of the work is made wider or a "cut-off" wall is built in the ce up to a hard bed where available, or a blanket of water-tight mate is provided on the upstream side.

Percolation through or unc a work does not effect its stability unless it is of sufficient pressure velocity so as to disturb particles of any portion of the work.

Free-board. Free-board canals is given at page 63. This is a margin between the water level the top of embankment or bank. The height of free board varies the importance of the embankment and the provisions of safety ess and upon the height of water and

wind conditions: Violent winds may cause wave dashing to a height of 3 to 4.5 m.

The free-board is generally kept 1.5 m for heights of dams 4.5 m and under, and 1.8 m for heights 4.5 to 7.5 m. Above 7.5 m upto 15 m it may be 2.1 m.

For embankments of rivers the height of free-board is the same as found adequate for dams, and is generally kept about 1.5 m (and in exceptional cases up to 3 m), above the designed high flood level. Free-board should be able to provide for earthwork in emergency to close small leaks. It also allows for any accidental settlement of the dam.

Top Width of Dams and Embankments. The minimum top width of an earth dam is 1.8 m up to 4.5 m height, increased to 3.7 m up to 15 m height of dam. If top width is kept narrow, the rear slope will have to be correspondingly flattened to accommodate the line of saturation.

If top width is kept narrow, the rear slope will have to be correspondingly flattened to accommodate the line of saturation.

The following dimensions may be taken for bunds of minor tanks :—

Depth of water (m)	Breadth of water spread (m)	Top width of bund (m)	Free-board (m)
1.5 to 3.0	under 90	1.22	0.91
3.0 to 4.6	90 to 270	1.52	1.22
4.6 to 6.0	270 to 450	1.83	1.52
over 6.0	450 and above	2.74	1.83

A wide top has the advantage of being raised up during emergencies and also to provide material for closing of breaches or filling up scour holes.

The top of the bank should be given a slight slope for drainage to the land side.

Junctions in Embankments. When adding new earthwork to old (raising in height or widening), the old bank must first be cut or "benched" into steps with the treads sloping slightly towards the centre of the embankment and when throwing on the new soil the surface of the old work should be wetted so that the new earth may adhere to the old. Similary, junctions should also be made by cutting grips or "forks" into the sides of the old embankments. The new earthwork should preferably be added on the upstream or river side of the embankment where it will be pressed on to the old bank by water pressure.

Choice of Silt for Alignment of River Embankments. The most suitable alignment is a matter of judgrment and is fixed from various considerations depending largely on the behaviour of the stream as regards shifting its course. A flood embankment very near the river is prone to attack from river esosion which will shorten its life, and where it cannot be helped, stone pitching should be considered. Embankments made some distance away from the river cause less interference with the natural operation of the silt distribution by the river over the country and raising its level. Such embankments, by providing a wider waterways enable the high flood water level to be lower than with closer banks, create artificial storage for sometime. Wider banks are more expensive and require longer lengths of open canal heads.

The alignment should, as far as possible, follow straight lines avoiding sharp corners. The highest land practicable should be selected but care should be taken not to traverse, if possible, ground which is sandy, friable, cracked, impregnated with salts or otherwise unreliable. It will sometimes be necessary to have a double line of flood embankments in places where the first line is in any sort of danger, either from river erosion or from ordinary breaching owing to low ground or bad soil. A second bund is also necessary where the breaching or destruction of the first bund will cause wide spread damage. No embankment is safe against river scour and a bund does not aim at holding the river permanently to a particular course. Therefore, schemes should be at hand to construct new rear embankments at a safer distance the moment river threatens the front embankment.

Breaching Sections are sometimes provided in dams to guard against overflowing and breaching at dangerous places; they also serve as a warning. The dam is made smaller in cross-section at such places and the top level is also kept lower by about 30 cm, but it is made strong enough otherwise to be safe under ordinary circumstances, and which can be cut away rapidly when required. The water from a breaching section is led away into a channel made for the purpose.

Failure of Earthen Embankments and Closing of Breaches. Failure of an earthen embankment can be due to various causes such as :

(i) Erosion due to velocity of water, action of waves, rain and wind. Erosion causes slipping. Stone revetment is made or pitching is done, as explained in the following pages.

(ii) Overtopping due to insufficient height of freeboard. Maximum failures occur due to this cause.

(iii) Percolation and leakage due to insufficient ramming of the embankment and porosity of the material. The leakage water washes away the soil and caves are formed in the bund. The percolation may be under the foundation or through the bund proper.

(iv) Slipping due to steeper slopes than the materials can stand. It is generally due to the oversaturation of the downstream side of the bund which has insufficient cover. The bund must stay within the line of saturation as explained earlier. Proper drainage should be provided by putting in granular material on the land side toe to drain out the surplus water. When a slip has occurred, all the slipped portion and the loose and slushy stuff must be removed and replaced by fresh dry material. The site of the slipped portion should be stepped back or benched and fresh soil added layer by layer, well rammed and brought to the proper slope.

(v) Leakage due to cavities or holes formed by the burrowing animals or insects and rats. Hollows are also formed due to roots of trees which have decayed. leaking outlet pipes or conduits. Efficient patrolling of the banks should detect these before they develop into breaches. Slopes and tops of embankments should be provided with a layer of hard material which the burrowing animals cannot penetrate. If a sand-core is provided the sand collapses and fills the rat or ant holes and the leakage stops. Breaches also occur due to intentional cuts by cultivators.

(vi) Due to excess supply the hydraulic grade line rises up wetting the portion of the bank which was never wet before and which settles down the dry earth of the bank above causing a breach.

(vii) General defective construction and maintenance can of course, always be a cause for failure.

Closing Leakages. If the water flowing through a leak is sluggish and clear, it may be seepage water and there is no immediate danger but a muddy water flowing with some force shows that the soil particles of the bank are also being washed away and need immediate attention. Correct location of the hole on both sides of the bank is essential, which may not always be perpendicular to the bank. There is a whirling action in the water just above the leakage hole if it is of a big size. If the hole is small and some heavy turf sods are thrown on the surface of water near the approximate location of the leak, they are attracted towards the leak there, and may come out at the rear.

Leakage can be closed by throwing sawdust, bran, powered dung, etc., just upstream of the leaks. The stuff is carried by water into the leaks where it swells and stops the leaks. Holes can also be filled in from the front side with balls of clay and turf which can be pushed inside into the holes. A method for closing big leaks is to cut an inverted T-shaped trench a little above the water line outside the bank and the entire leak is then opened out starting from the exit side, and all is filled with best material available (loam is ideal for the purpose), softened with water. The trench side should be made in steps for good bonding,

Closing of Breaches. Before starting closing of a breach, labour and material (such as earth, sand, gunny bags, stakes, brushwood) should be collected at site in sufficient quantity. If earth is not available at site it can be obtained by cutting the outer slope of the existing bank. Enough earth should be collected on both sides of the breach on the existing bank. The ends of the banks should be protected first to prevent further widening. The process starts from both ends by slipping the earth from the heaps and protecting channel sides by grassy clods usually available from the berms. Earth baskets should never be thrown in the water. A semi-circular bund (ring bund) may be constructed on the water side with stakes, brushwood, mats and earth, etc., and water baled out. The sides and bottom of the existing bund at the breach site should be cut into steps to remove all loose material and to form good bond with the new material. If good soil is not available, provide a core wall.

Closing Breaches in Big Canals. This is usually done by driving a double line of stakes and filling jungle in between the stakes pressing it down with bags filled with sand and by men walking over them, a temporary bank of gunny bags is thus raised in the position of stakes and bushing. Straight closure in the large channels is not possible. No earthwork should progress before the flood through the breach has been arrested to some extent in this way. The closing of the breach is done by constructing a ring bund behind the line of stakes. Earth is slipped from both sides to fo.m the ring bund.

Protection from Wave Action in Floods. When banks dry up during non-rainy days, the soil material of the banks becomes friable and cannot stand the action of waves. Where plantation is possible, pilchi, sarkanda, kikar or willow should be planted for a width of about 30 m in front of the toe of embankment as the existence of such plantation breaks the force of the waves. Where practicable, grass should be allowed to grow on the side slopes below high flood level on the water side. The provision of brick pitching or loose stone protection on the upstream face will ensure safety of an earthen dam against wave action. Wave wash can be prevented by inserting a line of stakes at the water line along the inner slope of the bund and inter-twining twigs of bushes As a temporary measure, gnnny bags full of sand or earth ean be used.

Borrow Pits. Borrow pits should be sited well away from the embankments and should be so located as not to cut the hydraulic grade line but to leave some cover above it No pit in the bed of a water tank should be excavated nearer than twice the height of the dam from the front toe of the bund. Some engineers recommend that borrow pits should not be nearer than 15 m or even 30 m, from the toe of an embankment, and not nearer than 9 m of the toe of a big canal bank, and 3 m of the toe of a small channel bank, but if the depth of the borrow pits exceeds 0.6 m the distance should not be less than 4.6 m. As far as possible borrow pits should not be excavated on the land side as there they increase the infiltration head-acting on the embankment to the extent of their depth and tend to cause the embankment to leak. Borrow-pits should be as shallow as possible and not more than 30 cm deep in land acquired temporarily in cultivated areas, otherwise 1 m max., and no pit should be excavated more than 1.5 m deep within a distance of 90 m from the front toe of an embankment. Borrow pits should not be continuous as otherwise they will form a channel. At least 3 m wide strip should be left unexcavated in every 30 m or so. A space of about 1.8 m should be left around all pits for the workmen to pass. Borrow pits should be in multiples of 3 m lengths. No pits should be dug in the centre portion of a channel berm nor in a canal bed below bed level, except as detailed below. Where earth must be obtained from near a canal bank, the pits should not be more than 15 cm deep.

Borrow pits in beds of channels : In the case of large channels borrow pits can also be put in the bed leaving 1.5 m berm from the inner toe of the banks on either side, and a width equal to half the length of the pits between each pit, The width of such pits should not exceed half the bed width of the channel and depth 30 to 60 cm below the bed. These pits get silted up in 3 or 4 weeks' running and they form a partly water-tight bed. No pits should be dug in beds of channels in which no silt is ordinarily deposited.

For Bhakra canals, borrow pits in the beds of unlined branches, to get earth for banks, were specified to be dug up to a max. depth of 120 cm; length not more than 12 m, leaving 3 m ridge between two adjacent borrows Width 1.2 m less than the bed width of the canal.

Pits should not be dug near any masonry works or within 6 m of where footpaths or cattle trecks cross a channel as they tend to cause the inner slopes to the channel to slip down.

Borrow pits in berms : Borrow pits may be dug in the berms where they are too wide and likely to silt up rapidly. The earth should ordinarily be obtained by cutting vertical pockets, long lengths of which should never be dug down to below water level. The length of pockets should not exceed the bed width of the channel or 3 m whichever is less. Spaces left between the pockets should not be of less than 1.5 m width.

Pitching. Pitching is a covering of some hard material such as, stones, kankar blocks, concrete blocks or bricks, laid over slopes of earthen embankments. If possible, one rainy season should be allowed to elapse and bank given time to settle after it has been built, before pitching or any kind of stonework is commenced. Slopes of embankments should not be steeper that 1 : 1 but 1.5 : 1 should usually be adopted. Rough stones are most generally used for pitching with a thickness varying from 23 to 60 cm according to the velocity or wave action of the water. The stones should preferably be packed and firmly embedded over a bedding or backing of 75 to 150 mm thick layer of small broken stone, quarry rubbish, moorum, gravel, ballast or small kankar, well consolidated over the earthen slope to prevent the earth from being sucked out from between the stones by wave action. Pitching should be constructed at right angles to the slope to be safe against sliding.

The pitching stones should be the heaviest available that can be handled, and roughly cut to fit in properly. Stones should be tightly hand packed and laid with their broadest face downwards, with as large a proportion of through stones as possible, giving due regard to bond. All interstices, hollows and inequalities between stones should be filled up with smaller pieces and wedged up tight with spalls driven in with slight hammering The outer face of the pitching should be made as smooth as possible so as not to set up eddies that may cause scour lower down. The toe of the pitching should generally be carried 60 to

90 cm below the foot of the slope (into the ground), or a small retaining wall built, in order to give it a footing below saturated and soft top soil of the tank bed, for the stability of the pitching and security of the slope against slipping. The pitching should be widened out at the toe (near and below ground) so as to distribute the pressure over a wider area. Where the bank is of a soft and erodible character, the foot of the slope may be secured by piling instead of a small retaining wall suggested above and the thickness of the stone pitching also increased downwards at the rate of 1 : 12. The topmost course should be horizontal and laid in one level line throughout the length of the embankment, preferably in mortar, and rounded off at the corners in side pitching. Pitching should be at least 1 m higher than the High Flood Level and if possible should not be carried up to a greater height than 3 m without giving a berm somewhere.

When concrete or kankar blocks are used they should not be less than 30 cm cube in size. Where brick pitching is used, only one brick should be placed for each course either as a header or as a stretcher to prevent sliding. In reinforced brickwork pitching, care should be taken to leave expansion joints vertically at suitable intervals; the bricks are laid with frog downwards. Where stone pitching is to be pointed or grouted, the voids should be filled up with small chips or gravel and then pointed, or concrete grouting poured in.

Revetment

Revetment is a facing of dry stone pitching or other material laid on a sloping face of earth to maintain the slope in position or to protect it from erosion. Revetment is generally constructed with a slope of 1.5 to 1 or or 2 to 1 for ordinary soils which may be increased to 3 to 1 for sandy soils. Thickness of revetment generally varies from 45 to 75 cm according to the height. Other details given under "Pitching" should be followed.

Where stones are not procurable, mattress formed of brushwood may be used, which are bundles of branches and twigs from 20 to 30 cm diameter and about 3.6 m long bound with tarred ropes at intervals of 1.2 m laid side by side and tied together. These brushwood bundles should be secured by stakes or short piles to the bank on which they are deposited.

Core-Walls in Dams and Embankments

The object of a core wall is to provide a barrier to the passage to seepage water from the water side to the rear of the dam and also to the passage of burrowing animals who cause dangerous breaches in embankments. A core-wall may be of compact clay puddle, masonry (called a diaphragm wall), concrete, or planks driven as sheet piling for small or temporary dams, taken down to impervious strata. The core-wall may be located either in the centre of the embankment or on the water side of the slope, Both the methods have their own merits and demerits depending upon the materials used and other conditions. Although the outer core-wall prevents percolation of water into the dam but it is open to cracking due to alternate wetting and drying as a result of fluctuations in water level, and injury due to settlement of the slope.

(Also see under "Stabilized Soils for Embankment".)

The puddle core wall is generally 1.2 to 2.4 m wide at the top and both sides batter outwards about 1 in 12 or 1 in 10 to the gound level below which thickness is quickly reduced to about 60 cm wider than the top width and carried down at this as far as necessary. The thickness must be increased if the puddle clay is of poor quality. The top of the core wall is kept 30 cm above the highest flood level and 60 to 90 cm below the top of the embankment.

It is always preferable to make the whole embankment of one homogeneous watertight material and do away with the core wall which is liably to produce cracks and other defects in the body of the dam due to unequal settlements for non-homogeneity of the materials constituting the embankment and the core wall. The earthwork of the dam near the puddle needs to be specially selected and well consolidated to minimize unequal settlement of the earthwork and the puddle core. The dry soil around the puddle core extracts the noisture in course of time. The construction of the puddle wall should be carried up simultaneously with the earthwork of the bank. At ground level a suitable groove or nose is constructed into which the puddle core is keyed. A covering of 1 to 1.2 of ordinary earth must be placed over the top of the puddle core to prevent shrinkage and swelling due to exposure to atmoshere changes. Where the height of the dam exceeds 18 m a masonrv core wall should be preferred to a clay wall. It is the compacted clay core which gives real strength and impermeability to the dams.

The water tight barrier is known as *Core-Wall* where it is above ground and as *Cut-off* when below ground.

Sand Core. A sand core is sometimes provided where the embankment has to be built on an unrealiable *kalrish* soil. It is keyed 1 m into the ground and carried up to the high flood level line, giving 1.2 to 1.8 width at the top with side slopes the sand will naturally stand. The core wall is usually provided in the centre of the bank, but where it is to be added later on, it may be provided on the upstream slope with sufficient cover of earth over it. If any holes or cavities are formed by burrowing animals or ants, the sand collapses and fills the holes and breaches are avoided.

Clay Puddle

A pure clay does not make a good puddle although it may be sufficiently impermeable to water, as it is liable to crack. An admixture of about 1/2 to 1 part of sand with 2 parts of clay (exact proportion depending upon the nature of the clay) will reduce shrinkage considerably. Clay containing sodium carbonate is considered to be the best and the clay suitable for making roofing tiles quite good. Where sand is not easily available moorum should be tried but the mixture must be free from stones. Where black cotton soil is found it should be mixed with moorum in the proportion of not less than 1 to 1, prefer 2 to 1. Puddle core of such materials should be thoroughly tested before attempting any important construction,

The clay should be dug up and left expossed to the air in layers not more than 30 cm thick for at least 2 to 3 day and watered a few times a day. The materials for making puddle should preferably be passed between a pair of rollers placed not more than 3 mm apart so as to crush any stones or gravels present, before being mixed with water. The scoured clay should de passed through a pug mill or otherwise well worked up by men's feet into a smooth homogeneous plastic mass, only just sufficient water being added. The correct consistency for a good puddle is that at which it can be squeezed in hand without any appreciable quantity adhering to the hands when the pressure is released. A piece of clay puddle when dried should not shrink more than 40 mm (preferably 25 mm) and not less than 20 mm, per 30 cm length or it will probably not be sufficiently impermeable to water.

The clay puddle should be consolidated compact and deposited in layers not exceeding 150 mm in thickness and each layer should be well moistened before new layer is laid and must be well incorporated with the layer below by making cuts or "keys". Special precautions are taken to prevent the puddle becoming dry as it will otherwise crack. All puddle which has become dry or has cracked must be replaced. There should not be any right angles in the cross-section of the puddle wall or trench, as these might produce fissures or cracks in the puddle. In building a clay core, the clay should be contained within boards which can be raised as the dam is built up. Ideally, each layer of puddle should be continued over the whole length of the core-wall before another layer is placed, but in practice this is not always possible.

Cut-off Trench. In order to render the foundation of the dam impervious to seepage water a "cut-off" trench is made in the bed under the dam up to such a depth that will prevent water from the reservoir percolating underneath the dam. The 'cut-off' trench is made in the centre of the dam, over which the core wall is built. Holes may be drilled all along the bed of the trench and thorougly grouted with cement so as to provide a deep curtain below the bed, which is impervious to water (described below). The trench is filled with puddled clay or concrete which is well bonded into the bottom of the trench by keys or gooves to ensure water-tightness. Puddling in the trench is carried out by heeling by feet by workmen. The usual depth of a "cut off" trench is 6 to 9 m (even 30 m below the surface is not uncommon and still deeper walls have been built in England) and width 1.8 to 3 m according to the depth.

Key-Trench. A trench made under river banks which has the same functions as a cut-off trench and increases the path of percolation of the water. A key trench is very essential where the ground is porous, sandy or fissured. Usual section is : depth 90 to 150 cm, bottom width

120 to 180 cm, side slopes 1/2 to 1 or 1 to 1.

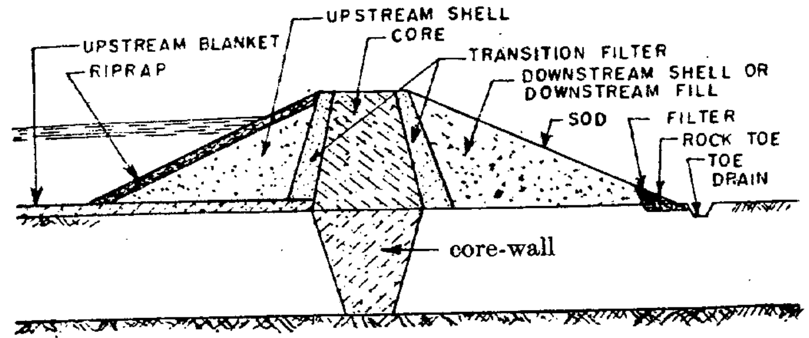
Where the cut-off trench is filled with concrete and puddle core-wall built over it, suitable gooves should be made for the core-wall to key into the concrete below.

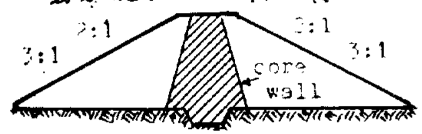
Strata which are not wholly water-tight can be made impervious by the injection of cement grout. The process consists of drilling small holes 5 to 12 cm dia into the strata and forcing in, under pressure, liquid cement either with or without sand or other fine aggregate. The cement enters and sets in the cracks and fissures in the soil, thus sealing them against the passage of water. If the trench is filled with concrete before grouting, it will provide an adequate weight to prevent undue wastage of cement. Pipes are brought up through the concrete for grouting.

It is interesting to note that sodium silicate has been used to seal strata into which it would have been difficult to inject cement grout.

Under certain circumstances of the soil use of cement concrete or grouting will be obligatory.

Earthen Dam





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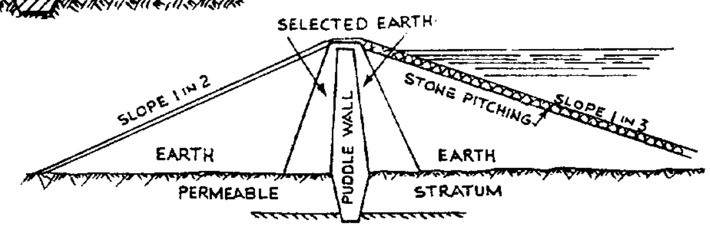


Drainage of Dams

Drainage is necessary to remove water which may get into heart of the dam due to percolation, leakage or rains. With good drainage a well built dam is rendered quite safe but undrained clay dams are liable to failure due to percolation through them. If the upstream portion of a dam is built of water-tight matcrial and downstream of permeable moterial, it will help draining out the excess water. A stone toe on the downstream side or a slope of about 1 in 10 outwards from the toe for some distance with alongitudinal collecting drain will facilitate draining of rainwater away from the bank.

A mattress of rubble stones about 60 cm thick all over the base (or only at the rear of the puddle trench) of the embankment with a system of under-drains will keep the embankment dry. Cross drains at intervals under the dam and longitudinal surface drains parallel to and near the toe, with a longitudinal drain in the bottom alongside of the puddle core to drain it of any superfluous water that gets into it while seeping through the bottom, should be provided. If a drain is provided on the downstream side below the hydraulic grade line, it will make the saturation gradient steeper. (Also see under "Stabilizing Side Slopes Against Slipping", described earlier).

Dry Stone Bund

Thickness at base of bund with river bed	...	0.50 H
Width at top of bund	...	0.25 H
Width at foundation bed	...	0.75 H
Foundation depth below river bed	...	(h/10+3/10)

H=total height of bund=height of water in metres impounded (h)+foundation depth. Also see Page 56,101

Spurs and Groynes. Are obstructions built across a channel projecting from the banks, generally from both sides opposite to each other for training the flow and for the formation of berms. They are constructed of 50 to 80 mm diameter stakes or kilas driven in the river bed 60 to 120 cm apart centres, single for short depths & two rows of stakes intertwined for deep canals. The height is usually up to half the height of the bank for temporary spurs and up to full height of the bank for permanent groynes. For best results, these structures should be placed two to three times their length apart. Brushwood and twigs are filled in between the stakes in alternate layers with stones or sand bags. The spurs break up the current and stop erosion by causing silting between the spurs and protect the embankment from damage by flow. They are quite successful in the case of small streams and where the soil of the bed is firm but in soft soils will cause scouring. For slow moving overflow floods in flat country, pitching, combined where necessary with spur bunds, are generally suitable. These obstructions are built either projecting at right angles with the banks or sloping downstream.

Hanging Spurs. Are branches of trees suspended at the edge from stakes driven into the berm or bank at some distance which induce silt deposit and formation of berms. New bushes are added where required and old allowed to rot. This method is very successful and is also called *bushing*.

Another method is to drive a line of stakes about 60 to 120 cm. apart on one or both the banks parallel to the length of channel with branches of trees intertwined behind the stakes. This is called longitudinal bushing.

Groynes. Spurs of permanent nature and obstructions going right across into a channel are generally called groynes. The word spur is generally restricted to a short protrusion. Earthen embankments are also made covered with stones, projecting into the river in order to head up water. These embankments are generally 3 to 3.5 m wide at the top with side slopes of 3 : 1 on the upstream side and 2 : 1 on the downstream side and section so designed as to cover a hydraulic gradient of 1 in 5 to 1 in 7 depending on the soil. Groynes are also made for the protection of canal massonry structures especially where the bed is sandy.

When a fixed obstruction is placed in a channel, there is scour upstream and silting up down-stream side of the obstruction; as the berm growth progresses the bed scours.

Detailed descriptions of spurs and groins for rivers along with sketches are given hereunder.

A Trunger is a cage or meshed casing made of about 5 mm steel wire, or coir rope 12 to 25 mm dia. which in the case of ordinary size spurs, in made about 7.5 m wide and height about 1 m more than the depth of water. The length of the spur depends upon the width of the river and the scour to be controlled. The filling in the trunger casing may be of anything like small trees, brushwood, dry grass weighed with stone, or it may consist wholly of stones, but in that case the trunger should be made of wires and not of coir rope. (Also see "Wire Crates and Mattresses for Training Works" in Section 19.)

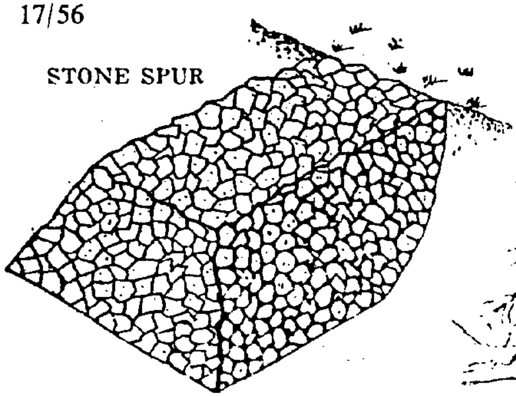
Stone spur consists of solid stones filled throughout. Stones weigh about 2.5 to 5 kg each laid in the form of a trapezoidal bank protruding into the river.

Brushwood Boulder Spur This consists of alternate layers of stone and brushwood. The brushwood is intertwined on the sides, so that the bottom and the top of each layer of stone forms a continuous lining round the layer.

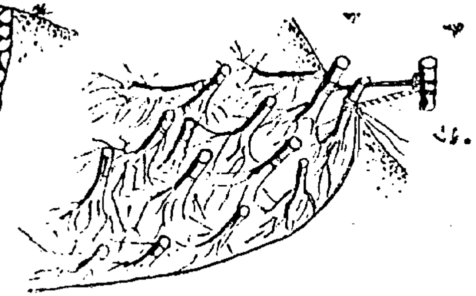
Brushwood Spur. Wooden poles (bullies) 100 mm to 230 mm dia. and of length of about 2.5 m more than the depth of water are driven 1 m to 1.2 m in the bed of the river to form an enclosure protruding from the bank. The space in-between is filled with freshly cut shrubs. The wooden poles are held together by means of coir ropes 25 mm to 40 mm thick.

Tree Spur. This spur is made of trees alone. Trees 6 m to 12 m in height and trunk varying between 45 cm to 120 cm girth may be cut and four or five of them tied to a stout iron or wooden pole driven about 12 m to 15 m away from the edge of sour, and are hang upside down along the bank of the river. More trees are then tied in a similar manner to these trees by means of coir ropes or steel wire.

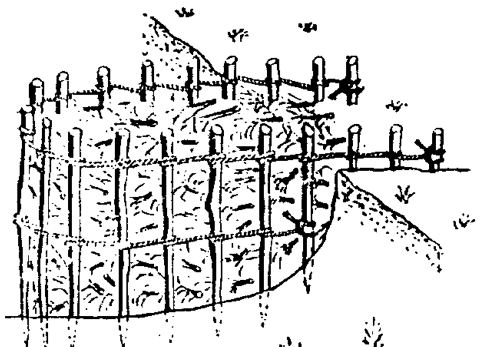
Log Spur Logs of wood are laid alternatively in longitudinal and cross direction in layers, one above the other and secured by means of



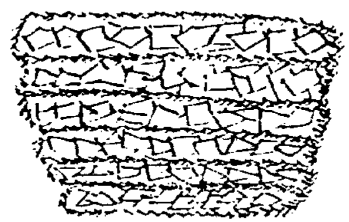
STONE SPUR



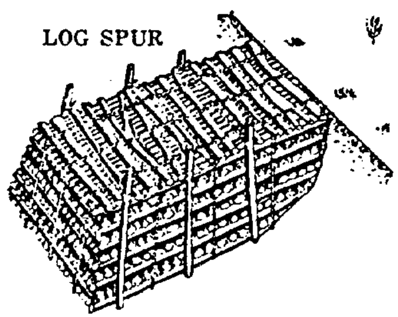
TREE SPUR



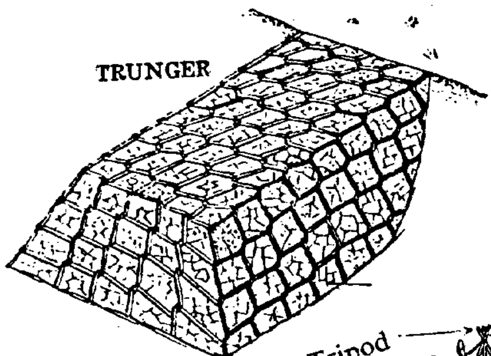
BRUSH WOOD SPUR



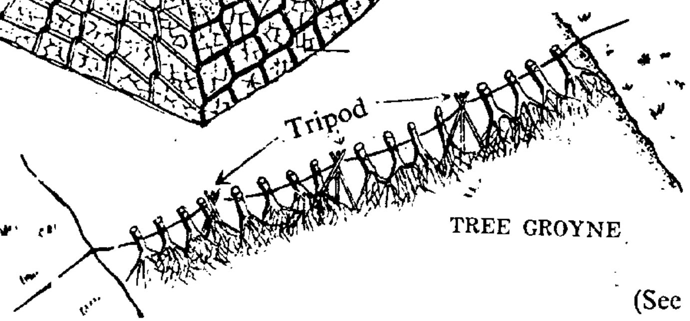
STONE FILLING IN BRUSHWOOD SPUR IN SECTION.



LOG SPUR



TRUNGER



TREE GROYNE

(See Page 17/101,102

ropes passed round them and fixed to the wooden poles on the sides. Wooden poles are driven into the ground.

Tree Groyne is a simple device. A steel wire rope 12 mm to 30 mm dia., depending on the length of the river is stretched across the river and anchored at its ends. At intermediate points it is supported on tripods formed of wooden poles. The anchors may be of iron rails embedded in concrete or rough logs laid in a pits of 2.5 to 3 m deep with earth filled over the decking. Usually holes are drilled through the trunks of the trees, and the tying wire in passing through the trunks fixes the trees to the wire rope. The trees begin to collect rubbish floating in the water and form permeable barrier which then collects silt and becomes impermeable.

6. CROSS-SECTIONS FOR MAXIMUM DISCHARGE, OR BEST FORMS, IN SMALL CHANNELS

The best form of cross-section of a channel is the one which is most economical, that is, a section which gives maximum discharge for a minimum cross-section, to reduce its cost, and a section which will have the least loss of water from absorption (i.e., with minimum of wetted perimeter). Such a section will have maximum sectional area with minimum perimeter or in other words, the best perimeter for a given area.

Theoretically, a circle for a closed channel, a semi-circle for open channel, half a square (i.e., depth equal to half the width) for rectangular channel and semi-hexagon for trapezoidal channel, are the best discharging channels.

The velocity of water in an open channel varies with the hydraulic mean depth (HMD) for channels of same area and slope, those which have the smallest wetted perimeter will have the largest HMD and the higest velocity. Flow will be greatest when friction is least, that is when the wetted perimeter is least for given area. Other factors remaining constant, the velocity varies as 1/2 to 2/3 powers of the HMD

For a trapezoidal channel the most economical section will be obtained when a semi-circle touches all its three sides and the diameter coincides with the water surface. Such sections are used in the case of lined channels with rounded corners or channels taking off from reservoirs where there is no silt in water. When the discharge from an open channel of a given area and with given side slopes is maximum, the hydraulic mean depth must be half of the central or greatest depth. But channels have to be designed according to the circumstances of the soil and the silt.

In a trapezoidal channel, if b be the bottom width and d the depth with given side slopes n, the channel of max. discharge has the following characteristics :

$$b = 2d\left[\sqrt{n^2+1} - n\right] \qquad d = \sqrt{\frac{A}{2\sqrt{n^2+1} - n}}$$

The side slope is equal to half the top width. The border is equal to the sum of the top and bottom widths. Unless there are special circumstances which make it more economical to do so, it is not usual to design canal sections with depths greater than their bottom widths.

The section for maximum discharge thus determined is suitable for small channels on land with an even surface and where the declivities are small. If the depth is great in proportion to the width, the deposits of silt will be heavy if the water they carry is silt-laden. Therefore, channels of any given depth which are broad in proportion to their depth, (i.e., those of which the hydraulic mean depth is large) will be "non-silting" on flatter slopes than those which are narrow and deep, (i.e., those of which the hydraulic mean depth is small). The loss from absorption, if the soil is porous, will be greater in a deep and narrow channel than in a wide and shallow channel.

Table for "Best Discharging" Channels

I. When area of cross-section is fixed or discharge is fixed :

Side slopes	Depth	Width of base	Width of top	Hydraulic mean depth	
	Square root of the area of the water-way of the channel multiplied by—				
Semi-circle	.798	0	1.596	.399	Area of water way is— $\frac{\text{discharge}}{\text{velocity}}$
0 to 1	.707	1.414	1.414	.354	
1/2 to 1	.759	.938	1.697	.379	
3/4 to 1	.748	.675	1.996	.374	
1 to 1	.740	.613	2.093	.370	
1-1/2 to 1	.698	.417	2.484	.345	
2 to 1	.636	.300	2.844	.318	

II. When breadth of the channel bottom or depth is fixed :

Side slope	Vertical	1/2 to 1	1 to 1	1-1/2to1	2 to 1	3 to 1
Bottom width b	2d	1.237d	0.828d	0.606d	0.472d	0.325d
Depth ..	0.5 b	0.809b	1.208b	1.65b	2.112b	3.077b
Top width ...			3.416b	5.95b	9.476b	19.46b
Area ...			$2.667b^2$	$5.734b^2$	$11.11b^2$	$31.55b^2$
R ...			0.604b	0.825b	1.059b	1.504b
Sides ...			1.414d	1.803d	2.236d	3.163d

If the section of a channel is too small for a given discharge and its slope is kept steeper than required, scour of the banks will take place till final regime is obtained. If section is too large for the discharge and the slope is kept flatter than required, silting will occur till final regime is obtained.

Silt concentration of the canal water affects the velocity considerably. When the canal water carries excess silt load, silting takes place and when the water is silt free, it picks up silt from the bed and sides. This results in erosion of the canal section. Hence velocity of flow of the water is the most important factor in a canal design with respect to its soil and slope.

7. DESIGN OF CHANNELS

Alignment

As far as possible, the line of canal should follow the water-shed or the highest line of the irrigable lands. Subject to this, more central the alignment is with respect to the commanded area the better will it be from the consideration of cost of distributaries and minors. Branches are so arrange as to command the greatest area of land and to supply the laterals (distributaries and water-courses) in the most direct manner. Where natural fall of the country through which a canal runs is greater than the slope of the canal, falls are provided.

It is not however, always essential to select the very highest ground as it may sometimes place the whole channel in heavy digging and thus add to the cost of the canal. The ideal design and alignment is that in which the earth obtained from digging in the bed is equal to the earth required for the formation of banks. This is called "economical digging" and the channel is said to be designed with the "balancing depth". If however the digging is more than the earth required for the banks, then either the bank dimensions are increased or another bank is built, called a "spoil bank", behind the service road bank and about 15 cm lower.

The most economical condition is when the canal water runs partly within and partly above the ground level so that digging may balance embankment and there is also sufficient command of level for irrigation.

It should not also be forgotten that carrying of canals in high embankments involves greater percolation and danger of breaches. The full supply level at the head of the off-taking channel are marked on the "Longitudinal" section and the full supply line proposed for the main canal or branch is kept high enough to provide a good fall into off-taking channel. After the alignment of the channels has been settled the area proposed to be irrigated by each channel is marked on the plan. It is essential that the boundaries of commanded area of various channels should follow the natural drainage line. The capacity of the different parts of the system must be based on the full supply capacity. Falls and bridges combined are very economical. For large distributaries a drop of at least 60 cm is desirable between the full supply level of the distributary and the main canal or branch, and 30 cm between major distributary and the minor.

Design of Distributaries

The length of distribution channels and their number depends upon the length of water-courses fixed. As far as possible the water surface should be keep at a level where flow irrigation is possible. The water level in a distributary channel should be designed so that the whole area commanded can be irrigated when 3/4th of the full supply is carried. Some engineers recommend that a channel should command its entire area at 1/2 to 5/8 of full supply discharge. Channels should be kept on the ridges, where this is not possible, they should be at right angles to the contours. Deep cuttings and high embankments should be avoided and the channels excavated in straight lines and short curves on the most direct course available. In order to avoid high banks and to ensure the surface of water being above that of the country, the slope of the distributary, should be made as nearly parallel as possible to that of the land it traverses.

Bed-slopes of canals are now generally fixed from Lacey's diagrams. The usual slopes for Northern india canals are : 1 in 10000 to 1 in 7800 for main canals and branches and 1 in 6000 to 1 in 3000 for distributaries and minors. The flatter the slope, the shallower is the channel. After the alignment of a channel is marked at site, the natural surface along the centre line is double-levelled.

The capacity is proportional to the duty performed and cross sectional area is diminished with steady draw-off from the outlets. As the water-courses do not run at right angles to the parent channel but always have an inclination downstream, in general the width of the strip of land between two distributaries, for about 3 km length of water course, may not exceed 5 km. Small size of distributaries are not economical as regards the maintenance cost ; a bed width smaller than 0.91 m should be avoided.

Design of Water-Courses or Field Channels

As regards levels, the same principles are followed as described for the design of distributaries The length of a water course depends upon the nature of holdings and is usually limited to 3 km and they are often branched to reduce the length and to gain command. Therefore, a distributary should be so designed that the length of a water-course does not exceed 3 km (max. 5 km) subject to min: command which gives a discharge of 0.084 cumecs, As far as possible separate water courses should be provided for high and low lands. Slope of a water-course generally conforms to the slope of the country. A min. slope of 1 in 5000 is ordinarily adopted, but 1 in 3000 to 4000 should be preferred especially for field channels. The usual practice is to provide a "field command" of 15 cm (min. 7.5 cm) and the working head at the outlet is kept 23 to 30 cm (min. 15 cm). A water-course occasionally may have to be carried in filling to serve a high area situated at the tail. In such cases the min. gradient of 1 in 10030 may be adopted, The full sapply level required at the tail therefore, will be the natural surface level plus 7.5 cm. A free-board of 15 cm is usually sufficient. As far as possible the water-courses should be aligned along the field boundaries. In a irrigation system for undeveloped tracts the area is generally divided into square or oblong blocks of 4 to 6 hectares according to the working capacity of the cultivators. The discharge of an outlet depends upon the size of holding and the interval between waterings

It has been found in practice that a 0.056 cumecs outlet is generally the best when the cultivator irrigates flat fields of about 0.2 hectare area. A discharge of less than 0.028 cumecs or more than 0.084 cumecs is not generally adopted. For optimum conditions the discharge of an outlet should be about 0.35 cumecs times the area in hectares of the field it irrigates.

Water Losses in Field Channels and Water Courses :

The proportion of water lost in a water-course steadily carrying one cumec will be higher than in the case of another carrying a discharge of two cumecs. In addition to this loss in the water-course there is the "spread" loss which occurs in the field. Actual loss is greater in the case of small discharges and the proportional loss is very much higher. Canals should run full whenever upon, and kept closed between waterings. This system reduces losses and gives "full head" to every outlet-pipe which draws its authorized discharge and the silt.

The following dimensions for the design of minors will be found useful :

Discharge cumecs	Bed width m	Dkpth m	Slope per thousand	
0.142	0.61	0.52	0.275	The cnanges in bed-width should occur as near to outlet heads as is feasible.
0.165	0.76	0.49	0.275	
0.170	0.91	0.47	0.275	
0.198	1.07	0.47	0.275	
0.226	1.22	0.47	0.275	
0.255	1.37	0.49	0.275	
0.283	1.52	0.49	0.275	

Curves in Channels

Minimum radii of curves on unlined channels :

Capacity of channel	Min. radii
Below 0 3 cumecs	100 m
0.3 to 3 ,,	150 m
3 to 15 ,,	300 m
15 to 30 ,,	600 m
30 to 85 ,,	900 m
above 85 ,,	1500 m

In general the min. allowable radius is 20 times the bed width of the canal in non-alluvial soils and 30 to 50 times (lesser radius for lesser capacity) in alluvial soils, with max. up to 100 times in non-alluvial soils and 200 times the bed width in alluvial soils.

Curves on Lined Channels

Capacity of channel	Radius	Capacity of channel	Radius
42 to 70 cumecs	300 m	210 to 280 cumecs	760 m
70 to 140 ,,	450 m	280 and above	900 m
140 to 210 ,,	600 m		

Largest radius possible should be given and channels should be super-elevated on the outer side of the curve so that a velocity higher than the safe velocity is not developed. Water has a tendency to erode the outside and deposit silt on the inner side, this is especially so in soft and alluvial soils. If, however, the channel is lined at the curve, the velocity can be maintained due to the reduced co-efficient of rugosity of brickwork. Earthen channels can have non-silting super-elevated bends with depths less than 2 m. Where the developed max: velocity exceeds the safe-scouring velocity of the soil of the bed and the sides, the channel section should be protected by pitching or paving.

Suitable Side Slopes in Various Kinds of Soils

For Canals in Cutting : Horizontal to Vertical.

Nature of bank	Up to 2.5 m depth	2.5 to 4.5 m depth
Firm rock	1/8 : 1	1/8 to 1/4 : 1
Soft or disintegrated rock	1/4 : 1	1/4 to 1/2 : 1
Alluvial soil, firm gravel, hard compacted earth, hard moorum	1/2 : 1	1/2 to 3/4 : 1
Tough hard pan	1/2 : 1	3/4 : 1
Stiff earth or clay well drained, soft moorum	1 : 1	1-1/2 : 1
Ordinary gravel	1-1/4 : 1	1-1/2 : 1
Ordinary earth, soft clay, dry sand, sandy loam, gravelly loam or loam	1-1/2 : 1	2 : 1
Loose earth, loose sandy loam	2 : 1	3 : 1
Wet sand	2-1/2 : 1	4 : 1
Light sand, wet clay	3 : 1	3 to 4 : 1

For Embankments slopes are flatter. See under "Earthen Embankments".

In ordinary soils, the usual practice is to excavate the channel at 1 : 1 slope, which after running for sometime, is gradually converted to 1/2 : 1 by the deposition of silt. Excavations can only be done to slopes safe for the soil. (See under Lacey's theory). Spoil banks are, however, made with 1-1/2 : 1 slope or the one with safe slope for the material. Estimates for excavation are made for 1 : 1 side slopes but discharge is calculated at 1/2 : 1 (hor. to ver.) See Illustrations—"Typical Cross Sections for Channels." Lined channels are generally given slopes of 1 : 1 to 1-1/2 : 1. Inside berms of channels in non-alluvial soils are made to the final shape at the time of construction, they cannot be formed by inducing the deposit of fine silt. The excavation should be done by first cutting a centre trench with vertical sides and then trimming the slopes.

Essential conditions are to have a good wide berm, sufficient free-board and a bank strong enough to keep the hydraulic gradient line for the particular soil well below the top of the bank.

Berms are made between the channel section and the bank which are formed by silting up where the canal is in filling and left at the time of excavation where the canal is in digging. Berms strengthen the banks bring in the saturation line and reduce possibility of leaks and breaches; they have maximum utility where the canal is in high embankments. It has been described earlier that the channels are usually excavated with side slopes of 1 : 1 which silt up to 1/2 : 1 slope, this itself gives a minimum berm width of 1/2 the full supply depth of the channel.

The berms width prescribed by U.P. Irrigation is : 0.6 + 1/4th of the width of combined side slopes of cutting and embankments as minimum, and 0.6 + 1/2 the width of the combined slopes as maximum.

Also : Dimensions in Metres

Up to 4.25 cumecs discharge = 0.6 + 1/2 full supply depth (D)
4.25 to 28 " " = 1.25 + 1/2 ditto "
28 cumecs and above " = 1.25 + 1/2 ditto + full supply discharge/28

For Punjab canal when the N.S. (natural surface) is above full supply level, the berm width is 1.5 D on the road side and 2D on the other side, when the N.S. is between full supply level and the bed, it is 2D and when the N.S. is below bed, it is 3D

The berm should be 15 to 30 cm higher than the full supply level and slightly sloping towards the channel.

Banks. Canal banks have to retain water and withstand the pressure. The adequacy of banks is usually checked by drawing the "saturation line". For safety of banks there should be a cover of at least 50 cm on the saturation line. Top widths and slopes of banks are generally governed by the hydraulic gradient and the nature of the soil.

Free-board. The height of the top of the channel bank above full supply level is called free-board and depends upon the nature of the soil, conditions of the banks and the abnormal fluctuations expected in the channel. Wide berms may have less free-board.

	Free-board	Top width of bank
Water courses	15 cm	0.4 m
Minor distributeries with 1.0 m bed width and below 0.3 cumecs discharge	30 cm	1.0 m
Ditto. with 1.0 to 1.5 m bed width	35 cm	1.25 m
Ditto. with 0 3 to 1.0 m cumecs discharge	40 cm	1.5 m
Major distributaries with over 1.5 m bed width and up to 3 cumecs discharge	45 cm	1.75 m
Ditto. up to 5 cumecs discharge	50 cm	2 0 m
Ditto. with 5 to 10 cumecs discharge	60 cm	2.25 m
Main and branch canals with 10 to 30 cumecs discharge	75 cm	2 5 m
Ditto. with over 30 cumecs discharge	90 cm	3.0 m

The following free-boards have been generally adopted for lined Punjab Bhakra canals :—

Lined main canals and branches	75 cm	to top of lining
Unlined branches	75 cm	"
Unlined distributaries	45 cm	"

The hight of free-board should not be less than 30 cm plus 1/10th of the supply depth, for fluctuations.

Service Roads and Inspection Banks. A service road to be motor-able should have a minimum width of 3.7 m, exclusive of the dowel, and that may be for canals up to 7 cumecs discharge. For higher discharge canals up to 28 cumecs, the width of the service road bank (in filling) may be 4.5 m to 7.6 m according to the cross-section, and which may be increased up to 9 m for canals of over 28 cumecs discharge. On distributaries and canal in cutting inspection roads are made on the natural surface, and in canals in filling the roads are made on the banks (top of pushta). Left banks are made inspection banks.

The width of the roadway is less when at N.S. level and more when on the banks in filling. Sometimes an "inspection path" is provided next to the berms in addition to service road, when there is not much of filling. When there is a spoil bank outside of and higher than the roadway, there should be a continuous drain along the outer edge of the road and cross drains through the spoil bank. A canal roadway should have an outward cross-slope of 1 in 30 to 1 in 40. Sometimes the word "Service Road" is restricted to tne road made during construction as an interim means of communication. The ins-pection bank is also called a "Petrol Bank". A *Boundary Trench* or a *Boundary Ditch* is made to mark the boundary of she canal land and to drain out rain water.

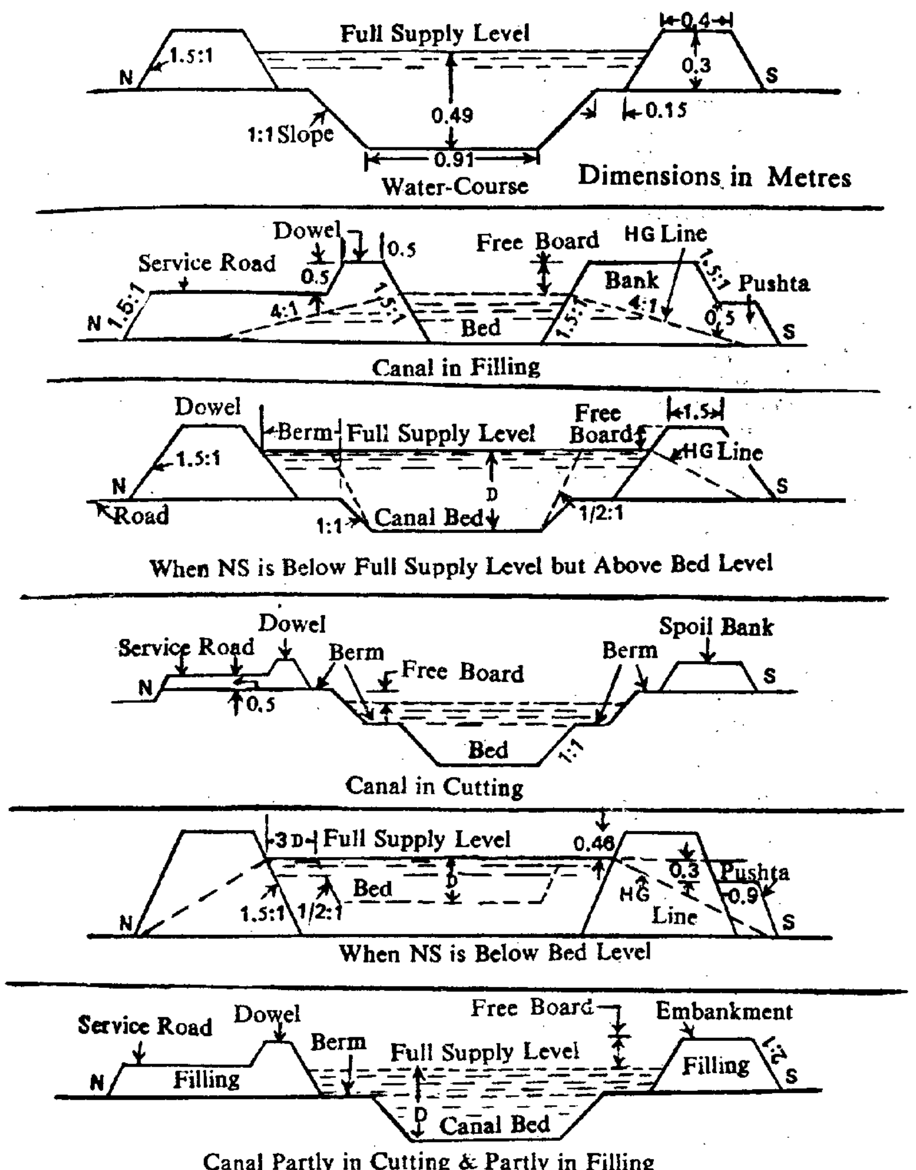
Dowels are short projections over a canal bank at its either edge constructed primarily to prevent cutting up of the bank slopes due to rain. Gaps are left (on the other side of canal) at intervals to drain off the water. Dowels are provided on the canal side only of the inspection road on N.S. level. Dowels provide additional safety so far a free-board is concerned and also ensure greater safety for wheeled traffic in driving. The size of a dowel is 30 cm to 45 cm wide on top and at least 30 cm high with 1-1/2 : 1 side slopes,

The dowel and bank should slope away from the channel; a slope of 1 in 60 to 1 in 80 in dry climates and 1 in 40 in heavy rainfall regions for the bank, and 1 in 20 for the dowel will be desirable for well maintained works.

Pushta, Banquette or Cover over Saturation Line

A cover of 50 cm (min) : to 1.22 m of earth is provided over the hydraulic grade line where it cuts the bank above the ground level. The minimum top width of the pushta should be 2 m for canals and branches and 1 m for distributaries (inclusive of the banks). This has been explained in detail under "Hydraulic Grade Line". The top level of pushta should be 30 cm below the full supply level.

The boundary ditch along distributaries and minors should be 30 cm wide and not more than 45 cm deep; care being taken that it is actually on the boundary.



Capacity of Channels	Main & Branch Canals		Major Distribu-tories		Minor Distribu-tories	
	30 to 150 cumecs	10 to 30 cumecs	5 to 10 cumecs	1 to 5 cumecs	0.3 to 1.0 cumecs	Below 0.3 cumecs
Max: Crest Width of Bank	3 m	2.5 m	2.25 m	2 m	1.5 m	1.01 m

HG Line is Hydraulic Grade Line or Saturation Line.
Slope in Cutting is 1:1 and in Filling 1/2:1.

Lining of Canals

Advantages of Lining :

(i) There is considerable saving in weter losses due to seepage in unlined channels which is considered about 20 to 40% through main canals, branches, distributaries, water courses and field channels, of the water delivered at the head. It is on the lesser side in alluvial soils and higher in predominantly sandy soils. The seepage is reduced to about 1/100th by lining. See under "Seepage and Evaporation Losses". Since the co-efficient of friction is decreased, velocity can be increased and size of the channel reduced to minimize travel losses. By lining a canal, its discharge capacity can be almost doubled; or conversely, the cross-section of lined canals can be only half as large as that of unlined canals. This also enables flattening of the bed slopes and increase of command for which greater areas can be commanded with more quantity of water.

(ii) Since the section of the canal is considerably reduced there is a saving in cost of excavation and land acquisition. (But this saving has to be adjusted against the cost of lining).

(iii) Lower maintenance cost.

(iv) Pilfering of water by cultivators is stopped.

(v) Prevents water-logging and efflorescence of adjacent lands due to seepage of canal waters. Large areas of irrigable land have turned to waste due to the rising of the water-table and the rise of alkaline salts to the surface caused by seepage through unlined canals.

(vi) Reduces bank erosion and breaches.

(viii) Lining prevents water absorbing salts where passing through *Kalarish* tracts.

Various types of lining are used which have their own advantages and disadvantages. The most commonly used are described below. The type selected should be cheapest combined with its stability and usefulness, or efficiency. Availability of the various materials at work-site is a very important factor.

(i) *Clay Puddle* : A layer of 7.5 cm to 15 cm is spread on the bed and sides and covered with 25 cm to 30 cm of silt. Puddle lining is quite satisfactory but can only be used if good clay is available; is considered to reduce seepage by about 80 per cent. As it is liable to develop cracks on drying it is suitable only for perennial flows. (Clay puddle has been described in detail elsewhere).

Permeability of soil can be considerably reduced by treating it with a bituminous material. The technique consists in adding the bituminous mixture to the ordinary mud plaster and then applied to the irrigation channel in thickness of about 12 mm which should be allowed to dry.

(ii) *Brick Lining* : For a successful job it is very essential that the bricks and the brickwork must be of the best possible quality. The earth to be used in the manufacture of bricks should not have a salt content of more than 0.3 per cent, calcium carbonate not more than 2 per cent, and clay content between 12 to 20 per cent. The bricks may be laid in single layers flat, or on edge or flat in two layers, (preferably tiles). The bricks are usually laid in "herringbone" pattern and are bedded on 12 mm layer of 1 : 5 cement mortar laid on the consolidated and damped soil in 1 : 3 cement mortar. A layer of 12 mm cement plaster 1 : 3 is sandwiched in between the two layers of bricks (or tiles). In floors of canal works the top brick-on-edge should be laid diagonally to the centre line of the channel, or in herringbone pattern. The lining may be plastered or pointed on the face, and may also be reinforced.

Brick lining has the advantage that no expansion or contraction cracks are formed as with concrete lining, repairs can be done easily, and has lower cost than concrete lining. Brick lining gives a saving of about 75 per cent in losses by absorption and percolation as compared with the earthen channel section. As bricks are porous the lining on the whole is less efficient in preventing seepage, except when sand-wiched with a layer of cement mortar. It has been found that tiles themselves are useless so far as prevention of seepage is concerned. They are extremely porous.

Extracts from some of the General Specifications adopted for the Lining of Bhakra Canals

Two layers of tiles 30 x 15 x 5 cm have been used, the bottom layer resting on 10 or 12 mm thick 1 : 5 cement-sand plaster. The sand-wich layer consists of 10 mm (preferably 15 mm) thick 1 : 3 cement-sand plaster. The bottom layer of tiles is laid in 1 : 5 and the top layer in 1 : 3 cement-sand mortar. A 6 mm layer of 1 : 3 mortar being laid on the sand-wich layer before the tiles are laid (of the second layer). This makes a total thickness of about 13 cm. One horizontal tile has been allowed in the top of the lining throughout the reach, which may be increased to 1 m width in heavy cutting reaches. 20 per cent of the cement used in the cement plaster and the tile masonry can be replaced by finally ground surkhi (used as puzzolana, see Section 12). Cement : Kankar lime : Surkhi in the proportions of 1 : 5 : 12 may be used to replace cement-sand mortar where the use of this mortar is found to be cheaper.

Specifications adopted on Thal Canal :

On a compacted and dressed soil is laid 12 mm layer of 1 : 10 cement-sand mortar and after curing for two days the first layer of 50 mm thick tiles is laid in 1 : 6 cement-sand mortar. The thickness of the mortar under the tiles is 3 mm on the average. Joints between the two tiles are usually 6 mm thick filled with cement mortar 1 : 6. 1 : 3 cement-sand mortar 10 mm thick is laid over the tiles. The second layer of tiles is laid on the 1 : 3 mortar and the tiles are also laid with 1 : 3 mortar.

Where salts are found in canal reaches within 3 m depth, a mortar consisting of 1 : 2.5 (cement-sand) with 20 per cent surkhi as puzzolana should be used on the sub-grade instead of 1 : 5 cement-sand mortar.

Where salts are in excess, these reaches should be lined after inter-posing a layer of 1.5 mm thick 30/40 penetration bitumen in between the sub-grade and the lining. Crude oil should be sprayed over the clay sub-grade at the rate of 50 litres per 10 sq. m. before spraying the bitumen layer so as to provide a bond between the sub-grade and the bitumen. (Fuller details of this process can be obtained from the Director, Irrigation Research Institute, Amritsar).

Low density polythene film 250 micron— 1000 gauge thick can also be used.

Method of Construction : Success of linining is mainly dependent on the care with which the back-fill has been compacted. The com-pacted section should extend not less than 60 cm inside the final section of the canal. The length to be lined should be thoroughly soaked with water, without making it slushy, to ensure that water panetrates to a depth of 30 cm in sandy soils and 15 cm in other soils. Alterna-tively, the sub-grade may be water-proofed by use of oil paper or by spreading of crude oil, linseed oil or such proprietary materials as Shell Primer, etc.

Tiles with their top ends at correct formation levels are first placed at 7.5 m centres in the bed of the canal alcng the centre line, and similarly on the side slopes. To get the correct profile in the side slopes, three templates are erected 7.5 m apart. For lining the side slopes the masons work standing on 60 cm wide planks supported on ladders leaning on the side slopes.

The frogs of the tiles should be kept upwards in order to give a good grip to the sand-wiched plaster. The direction of the tiles in the bed will be at right angles and on the side slopes parrallel to the centre line of the canal. The tiles should be well soaked in water a few hours before laying and they should still be quite wet when laid. The plaster should be well pressed so that the excess water or air locked into the pores is driven out. The finished surface of the plaster should be made rough for proper jointing by means of fibre brush or broom. Full and complete curing should be done. It is important to see that the thick-ness of the sand-wich layer is uniform.

If the formation is sandy and soaks in water from the cement mortar, the slump of the mortar should be increased to suit the condi-tions at site and tiles should be quite wet. The first layer of tiles is allowed to cure for 3 to 4 days before the sandwich cement sand mortar is put on. The sandwich layer is kept wet for a full day before the top layer of tiles is put on.

The sand-wiched mortar 1 : 3 has been found to be impermeable for all practical purposes. A pacca lining with seepage losses of less than 0.15 cumec per million sq. m of wetted perimeter may be regarded as more or less impermeable for all practical purposes.

General Design Data :

Surface slope	0.15 to 0.20 per thousand
Lacey's "f"	1.54 to 2.01

Velocity	1.25 to 1.61 m/sec.
Full supply depth	4.35 to 5.37 m
Free-board	0.76 m

(iii) **Concrete Lining**. This type is generally considered most for big canals if cost is not prohibitive. Usual thickness is 6 cm to 15 cm according to the design. Gunite and shot-crete linings are superior to ordinary work. A thin coat of plaster is applied to give a smooth surface. Longitudinal and transverse joints are provided according to the thickness of the concrete, thinner linings having joints at closer intervals Normally plain vertical butt joints at 2.5 m to 3.5 m intervals will be found suitable. To make the joints water-tight, copper strips or steel plates may be put in and the joints filled with bitumen. Bitumen cannot be filled in properly in a joint less than 6 mm wide. The concrete should be made waterproof by any of the methods described in Section 8. Kerosene oil 10 per cent by weight of cement, is known to water-proof the concrete and also to retard the evil effects of alkaline soils, for which otherwise high alumina cement is required. The concrete may be reinforced but reinforcement is not much in favour, although it will assist in preventing failure of the lining due to settling of the sub-grade, and the spacing of joints can be increased. Concrete blocks with joints filled with asphalt will probably be better.

Guniting is not considered suitable for channels in soil sub-grade. For the use of precast cement blocks flyash may be used in proper proportion as replacement of cement (See Section 12)

The use of polythene to replace the impervious layer of cement-sand mortar of 1:3 is feasible. For combination type of lining, though the durability of 400 gauge polythene is adequate, use of thicker gauge will definitely improve the durability yet for its large scale adoption limitations such as short closures, compaction difficulties, problem of weed growth in clear water channels, high water tables scour below structures, etc. be kept in view.

Soil Cement Lining. Stabilized soil with 5 per cent of cement to be compacted in a 75 mm layer and topped with 6 mm to 12 mm thick cement-sand plaster. Proportions of clay, silt and sand, may be 8 to 15 per cent, 12 to 25 per cent and 60 to 80 per cent by weight respectively. (For fuller details see under "Soil Mechanics" and "Roads".)

General Considerations for Lining :

(i) The sides of the channel to be lined should preferably be kept at the natural slope of the soil as then there will be no earth pressure against the lining. Where the side slopes are made steeper the lining will have to be designed as sloping retaining walls which under worst conditions will be subject to pressure due to saturated back-fill and the differential water head across it. Arragements should be made so that no water gets behind the lining from any external sources; it is necessary to provide adequate facilities for artificial drainage of rain water to keep the backing in proper form. There should be no over-topping of the canal water due to wave action for which sufficient free-board and a dowel should be provided. The dowel and the bank should slope away from the channel.

(ii) The backfill soil must be thoroughly compacted at the optimum moisture content, and if the channel is allowed to run for sometime before the lining is laid it will further settle the soil especially where a channel runs in embankment.

(iii) The bed and sides should preferably be constructed independent of each other with the lining of the sides resting on the walls if practicable.

(iv) In deep canals life saving devices should be provided which may consist of steps or flat slopes at intervals for cattle and some ladders for men.

The safeguard the lining against any uplift pressure, pressure release arragements have to be provided by making provision for filter drains (perforated pipes) and pressure release valves.

Forms of Cross-Section. Since a lined channel can allow a higher velocity it would seem more economical to use narrow and deep sections with sides at the same slope as the angle of repose of the soil and a circular bed. The arc in the bed should be tangential to the side slopes with the centre of the arc at F.S. (full supply) line and the radius equal to depth, with the central angle equal to twice the angle of side slopes. This will make a triangular section with circular bottom. For wider channels a trapezoidal section with rounded corners can be made by giving arcs at the bottom two corners as for the above stated section the each central angle in this case will be equal to the angle of the side slopes with the horizontal.

For lined canals side slopes of 1:1 up to a depth of 3.6 m is generally adopted but it is considered a little unsafe. Flatter slopes depending on the soil conditions should be considered. The side slopes of the sub-grade may be 1 : 1, 1.25 : 1, and 1.5 : 1 respectively for depths up to 1.5 m; 2.5 m and more than 2.5 m.

Surveys and Plans for Canal Projects.

For canals, contour surveys may be available from the Survey of India Deptt. Perliminary canal line may be surveyed by running a rapid contour falling at the rate of 15 to 40 cm per kilometre according to the nature of the land. Detailed levelling can then be conducted on the final line by taking levels at every 30 to 90 m longitudinally according to the nature of the site and the work, and at every 6 or 9 m cross-wise. *Reconnaissance* is a preliminary field examination of a proposed project. For preliminary estimating for canal exacavations and embankments the depth or height should be taken to the nearest 15 cm and the distances at 60 m to 120 m apart.

For the lay-out of water-courses and in the irrigation selection of a *chak* (field) the first requirement is an accurate closely contoured map of the area. In the Punjab Irrigation a 20 cm contour map to a scale of 5 to 10 cm (which may be up to 15 cm) to a kilometre is usual. Water-courses are designed on contour plans.

Scale for longitudinal sections of a distributary is generally 3 cm a km is used. Vertical scale generally adopted is 1/100. For small channels or where more detailed information is required a vertical scale of 1/50 can be adopted. All cross-sections should be 150 metres apart in case of minors and distributaries and 75 metres apart in the case of minor branches.

Trial pits should be made in the canal line, about 6 in a kilometre, to ascertain the nature of the soil and also at the site of the masonry works and canal crossings,

9. SEEPAGE AND EVAPORATION LOSSES IN CANALS AND RESERVOIRS

Loss of water by absorption, percolation or seepage and evaporation depends upon various conditions such as :

(i) Permeability of the strata through which the canal passes. In sandy or porous soil, loss from percolation is very great until the soil between the ground surface and the sub-soil water level has been fully saturated. In black soils the losses by percolation are much less.

(ii) The sub-soil water level and the drainage conditions of the sub-soil.

(iii) The age of the canal, the loss by absorption is much greater in a new canal which is reduced by age as the silt deposits The usual method to stop excessive percolation is to excavate the channel to a bigger section and get it artificially silted up with light and fine silt, to the required cross-section. If the distributaries are kept alternatively wet and dry for short periods, the soakage loss is increased.

(iv) Loss by absorption is greater when a canal is in cutting than when it is in a bank.

(v) Amount of silt carried by the canal; the purer the water the more the loss.

(vi) Velocity of water in the canal; the more the velocity the less will be the percentage of loss.

(vii) Cross-section of the canal, and this is the most important factor. Other conditions being constant, the loss by absorption varies directly as the wetted perimeter and the depth of the channel. Therefore, economy lies in having a section which has the least perimeter for a given area. Percentage of loss by seepage is greater in small channels than in large canals.

The loss due to seepage in earthen channels is much more than loss by evaporation. Kennedy found that the rate of seepage in Punjab canals from April to September was 40 cm depth of water per day, and from October to March it was 27 cm per day. In full scale experiments carried out in Northern India, the loss by seepage in an unlined channel was reduced to 0.45 in six weeks of the cubic measures of water that was on the date of filling. The percentage losses which include evaporation are usually taken as follows, at different stages :—

	Per cent
In main canals and branches up to heads of distribution channels	15 to 20
In distribution channels	6 to 7
In field channel or water-courses	17.5 to 21
In cultivation over fields (due to un-economical methods)	8.5 to 25

Useful percentage of water received by plants may be as low as 30 per cent. The total loss of irrigation water in transit through unlined channels usually varies from 15 to 70 per cent, which is reduced to almost negligible with a good lining.

Absorption loss in an unlined earthen channel (for Punjab soils) may be worked out from the formula :

$K=1.9\,Q^{\frac{1}{16}}$ K=absorption loss per million sq. m of wetted perimeter; Q=discharge in cumecs.

On some channel it is usual to add a further 75 mm as a margin to cover silting in the head reach of the water-course. This is a valuable precaution because in the absence of such margin any silting in the water-course must effect the modularity of the outlet.

The following percentages are generally taken of (full supply) discharge for Bombay Deccan canals for loss by seepage :—

Full supply Discharge in cumecs	Over. 2.8	2.8 −1.4	1.41—0.7	0.71—0.42	Less than 0.42
	Per cent	Per cent	Per cent	Per cent	Per cent
Allowance for transit loss	0.25	0.5	1.5	2.0	3.0

(non-alluvial soils)

In U.P. the following empirical formula is used to determine the losses in cumecs per km length of channel : $Q=1/200\,(B+D)^{\frac{2}{3}}$

Where B is bed width and D is depth of the channel in metres, Q is discharge in cumecs. In general, the losses are taken as 4.80 cumecs per million sq. m of wetted perimeter for ordinary clay loam to 9.60 cumecs for sandy loam.

In Madras the allowance per million sq. metres of wetted surface is as follows :

(i) for rock ... ... 0.90 cumecs
(ii) for black cotton soil ... 1.51 „
(iii) for alluvial or red soil ... 2.42 „
(iv) for decayed rock or gravel ... 3.00 „

For channels unaffected by fluctuations in ground water table the following average transportation losses are recommended by Khushalani in cumecs per million sq. metres of wetted perimeter :—

Nature of Soil	New Canals	Old Canals
Impervious clay loam	1.20	0.90
Medium clay loam with a hard pan layer at 60 to 90 cm depth	1.51	1.20
Medium clay loam	2.10	1.51
Ordinary clay loam	2.50	1.80
Silty clay loam	2.85	2.42
Silt loam	3.16	3.70
Loam	3.30	3.00
Sandy clay loam, gravelly clay loam	3.46	3.16
Sandy loam	5.12	3.62
Loose sand	5.73	5.12
Gritty soil	6.93	5.73
Gravelly sand	8.74	6.93
Porous gravelly soil	10.6	8.74
Gravel	21.1	10.6

Evaporation Losses

Water will evaporate if left open. The quantity of evaporation depends upon climatic factors of temperature, humidity and wind; it varies directly as the area of the water-surface and inversely as the depth of water. Evaporation takes place all the 24 hours. Evaporation however, is never a considerable percentage of the total flow of a canal and is hardly about 1.5 to 2.0 per cent of the seepage loss of an unlined canal. But rarely exceeds 10 mm in a day in the hottest and driest parts of India.

There is more of evaporation from an open water surface than from land. Evaporation from distributaries is less than from main canals.

Loss of Water in Storage Reservoirs and Earthen Dams

In reservoirs on natural ground the loss from absorption is considerably reduced after the bed of the reservoir has become water-logged and silt has been deposited. Pressure due to depth of water increases seepage losses. In an earthen dam, there is much loss by seepage and absorption. Total loss may amount to 15 to 25 per cent of the storage capacity in very dry tracts with small rain-fall. Loss of storage by silting in the case of earthen dams is assumed at 15 per cent to 20 per cent of the gross storage capacity. It is usual to keep the outlet level higher than bed and at a point below which the reservoir capacity will be 10 per cent of the total as in course of time silt is deposited at the bed of the reservoir. There is no loss of storage by silting in the case of masonry dams as scouring sluices are provided at the bottom.

Where water is stored in masonry tanks, absorption in the tank may be as much as 3.5 mm per day. This loss by absorption is generally taken half of the loss due to evaporation. Loss by leakage or seepage is negligible unless structure is faulty.

For preparation of projects the following figures for evaporation may be taken :

for 4	cold	months— 75 mm	per month	300 mm
for 4	hot	months—250 mm	per month	1000 mm
for 4	monsoon	months—125 mm	per month	500 mm
			Total	1.8 m

Loss of water in tanks from evaporation and absorption in Southern India is about 1.8 m per year and from lakes in Bombay about 1.2 m at full lake level.

10. SUITABILITY OF SOILS AND WATER FOR IRRIGATION AND LAND RECLAMATION

Types of Soils

Alluvial Soils : These soils are formed by materials like sand, silt or clay transporated and deposited by flowing water or as a result of river floods in the course of time, and are found in flood plains and even country (the area of alluvial soil has very flat or gentle surface slope), deltas of rivers and near the coasts Alluvial soils are soft and generally of great thickness and can absorb a fair percentage of rainfall and retain in the sub-stratum which make the soil highly productive. These soils are mostly alkaline Alluvial soils cover large portion of land areas in the Punjab, U.P., Bihar, Bengal, Orissa, Assam, Sind, Malabar coast and other north-eastern parts of India. *Incoherent alluvium* is pure sand.

Non-alluvial Soils : Soils formed as a result of disintegration of rocks, carried over a long time. The area of non-alluvial soil has usually an uneven topography and hard beds. There is no silt problem in the channels built through this soil and the rivers passing through such area have tendency to shift their courses. The major portion of Bombay district is an area of non-alluvial soils.

Red Soils : These soils chiefly occur in Madras, Hyderabad Deccan Bihar and Orissa provinces. Are light textured, crystalline, porous and friable and of sandstone formations differing in depth and colour from place to place. The sub-stratum is *moorum* or decayed rock. The colour ranges from red to black with an intermediate shade of dark brown ; cultivation changes the colour of the soil. These soils are generally acidic or neutral but not alkaline, contain some lime and phosphate and no free calcium (kankar), content of soluble salts is very low and vary in fertility from place to place. The ground with this soil is broken, fissured and undulated. As such no big irrigation works are feasible but they are good for cultivation with tanks. But some of the better classes of these soils are more suitable than alluvial soils.

Black Soil : Is a heavy soil varying from clay to loam and made of similar materials as the red soils. This soil is found in Bombay, Madhya Pradesh, Madras and Hyderabad Deccan provinces and vary from shallow depths of 30 to 60 cm to deep layers of 3.5 m and over. The sub-strata consists of *moorum* or heavy clay. (See "Black Cotton Soils in Section 6.) The soil contains uncombined calcium corbonate, is rich and productive, except for the deficiency of nitrogen, and are very favourable for the growth of sugar cane and cotton.

The kind of crops that can be grown on a soil depend upon the texture of the soil which is the size and gradation of the particles, and that is determined by its clay contents. Soil containing less than 2 per cent clay is useless for crops other than barani grams. (An average soil in the Punjab contains clay from 12 to 15 per cent.) The clay content of a soil mostly determines the kind of crops that can be grown on it. Soils with very heavy proportions of clays are not suitable for artificial irrigation.

***Sandy** soil is known as light soil, loam as medium or normal soil and clay as heavy soil.*

Heavy Soils or retentive clay soils are unsuitable for crops requiring large quantities of water. But soils containing 20 to 40 per cent of clay are suitable for sugarcane, rice, cotton and wheat. (Produce of the last two below normal).

Light Soils : Sandy soils are light soils and are suitable for crops requiring small quantities of water. Soils containing 2 to 10 per cent of clay can grow wheat, gram and fodder crops. With less clay the yield is below normal. (Yield of wheat below normal).

Medium Soils : Loam is known as medium soil with clay content of 10 to 20 per cent. Are suitable for crops requiring only normal quantity of water. Generally cotton, wheat, maize, vegetables, oil seeds and fodder crops are grown ; the best yield is from such soils.

Composite soils are sandy-clay, clayey-sand having sand and clay in certain proportions In sandy-clay, the clay content is more and in clayey-sand, the sand content is more. In nature, we have mostly composite soils.

Salts in Soil Crust. The presence of certain salts change the entire physical and chemical conditions of the soil. The salts commonly met with in the soil crust are the sulphates, chlorides, carbonates and nitrates of sodium, potassium and magnesium, etc. and also chlorides and nitrates of calcium Salts of sodium chloride, sodium sulphate, sodium carbonate and calcium chloride are harmful salts. Potassium are beneficial. Harmful salts are usually called alkali salts. The calcium carbonate and sulphate are not sufficiently soluble in water and are thus not harmful to the crops. Salts of sodium are undesirable in any appreciable quantity and especially sodium carbonate which is most harmful and this salt is known as black alkali. The yield from the soil is not generally affected up to about 0.18 per cent of the total harmful salts, but if the salts exceed 0.25 per cent the soil becomes infertile. The harmful sodium salts should not be more than 0.10 per cent if sodium carbonate, 0.25 per cent if sodium chloride, and 0.50 per cent if sodium sulphate is present. Deficiency of salts can be made good by manures. Sodium and potassium nitrate are beneficial as manure if present in small quantities. Nitrogen is the principal material which helps growth of plants and Indian soils generally lack in nitrogen.

Generally speaking, clayey soils are alkaline (as in the Punjab) and sandy soils acidic, especially where rainfall is high and the country slopes too rapidly allowing the finer particles of sand to be washed away. In general, salts are found in tracts of poor slope, high sub-soil water table or water logging, relatively impervious crust, and low rainfall. They are almost absent in regions of over 63 cm of rainfall; a rainfall of over 75 mm at a time in a day will wash away lots of surface salts. The application of irrigation waterings may keep salts from damaging crops but the reduction of waterings may bring the salts up again, especially where the areas were fairly salty before irrigation.

The alkalinity of a soil is a very important factor. A very alkaline soil is impermeable to water and is unsuitable for the growth of normal crops. Provided a salt content is low (less than 0.2 per cent), the yield is not affected until the pH value rises above 8.5 (pH value has been explained under "Water Supply".) In such a case there are usually no white salts on the surface.

Suitability of Water for Irrigation

The same salts present in the soils are also found in well waters and most of the mineral salts (which are alkaline) are injurious for agriculture although some of the salts are also beneficial. If the dissolved salts in water are those of calcium and magnesium only they are not harmful, but if sodium salts are present, the suitability of water will depend upon the nature of the soil to which the water is to be applied. In the case of a well drained (clayey) soil containing calcium carbonate, small quantities of sodium salts in water will not do any harm. Water containing appreciable quantities of sodium carbonate and sodium bicarbonate when applied to low-lying clayey areas will harden the soil with likelihood of causing water-logging. But where the soil contains a large proportion of coarse and fine sand this tendency will be checked; and if there is a high percentage of lime in these soils this will further tend to counteract any bad effects due to the presence of sodium bicarbonate. Calcium salts retard the evil effects of sodium salts. As a general rule the quantity of potassium nitrate in well waters is very small, the greater its amount the better the water for agricultural purposes.

Ordinarily the total soluble salts should not exceed about 300 parts in 100,000 parts of water beyond which the yield of crops drops. Water containing total solids up to about 100 parts per 100,000 of water with high percentage of calcium salts (40 to 60 parts) and magnesium salt (20 to 40 parts) and percentage of sodium salts (10 parts) can be used for irrigation purposes without injury to the soil or the crops.

If the value of salt index is negative the water is suitable for irrigation purposes and vice versa. The pH value of irrigation water should be between 7 and 9; between 6 (acidic) or 7 to 8·5 gives normal yield, between 8.5 to 9 yield decreases and when the value rises to 11, the soil becomes infertile. Tube-well waters which come from deep soils generally have very little fertilizing qualities and more manure has to be applied to the land.

Soil Efflorescence. Free salts in the soil when near the surface usually concentrate during dry periods as a white crytalline deposit known locally as *Kallar* if of long standing and *thur* if brought up since the start of irrigation. Other local names for salt efflorescence are *shora usar*, *lona* and *reh*. Salts are also deposited an the surface when the land is irrigated with alkaline water for sometime. Sodium chloride (common salt) and sodium sulphate (Glauber salt) are the principal ingredients of white alkali. Over irrigation is also a cause for soil efflorescence where the sub-soil water-table is high or the soil is non-porous. Efflorescence is destructive to crops and natural vegetation and attacks even masonry works nearabouts.

It has been observed that in areas of high water-table the sub-grade is frequently impregnated with sodium sulphate which has its deterimental action even if present in small quantities. In canal irrigation much larger quantities of water are used which bring out salts in the soil crust, as due to over-irrigation the water-table in the locality generally rises up. Basin irrigation, where a good depth of water is stored for considerable periods, is also a cause for rise in water-table and efflorescence. Well irrigation (unless the well water is salty) does not bring salts on the surface. Too much watering should be avoided and only just sufficient quantity of water required for raising the crops should be given. Water logging, bad sub-soil drainage and scanty rain-fall with long periods of hot and dry weather cause efflorescence.

Water-logging. Is the result of rise of water-table due to infiltration from rivers, canals, tanks, and inadequate sub-soil drainage. Water-logging does not occur in porous sub-soils but it occurs in impervious substratum. For most of the plants to grow it is necessary that the water-table is below the root zone of the plants and at least 1.5 m below the surface. In a water-logged area water-table can be lowered by providing surface and sub-surface drains. (These have been described in detail in Section 18). Tube-wells lower the water-table to a great extent. Lining of canals and water-courses prevents water-logging.

Land Reclamation. Land reclamation is a process of making an unculturable land (such as a waste-land under thick jungle, alkaline, water-logged or badly eroded land) fit for cultivation. It is essential to know the limits of both salts and alkalinity in the soil at which yields of crops begin to decline, the limits at which crop growth becomes impossible and also the limits when reclamation cannot be economically carried out.

Reclamation of Salt Affected Land : (a) By artificial drainage to lower the ground water-table below the limits of capillary action; both the surface and under-ground drains are provided. The limit of "capillary action" depends upon the kind of soil. In clayey soils the ground water-table has to be lowered to a greater extent than in sandy soils.

(b) **By Leaching :** Excess salts are leached from the top 90 to 120 cm of the soil by flooding it with some 15to23 cm depth of water which will dissolve the deposited salts, and the salts in solution percolate down and join the water-table, when the clay content of the soil is low. As large quantity of water is required and the process may have to be repeated, this method may not be practicable. Land may be divided in areas of not more than 2000 sq metres each so that heavy flooding may be done frequently without undue waste of water. The interval between flooding should not be so long as to let the soil to dry. Once the soil dries up without having cleared all the salts, salts will come up on the surface again by the action of capillarity. This method may be worked with sub-soil drainage or coupled with suitable cropping (such as rice). Very alkaline soils are impermeable and leaching is difficult.

Washing down of salts is likely only when the water is within 1.5 to 1.8 m of the ground surface and the clay content is less than 10 to 15 per cent; but there is a possibility of the salts appearing again unless the salt zone can be depressed at 3 metres.

(c) Soils with pH value higher than 9.5 are not economical to reclaim. Soils with salt content below 0.2 per cent and pH value between 8.5 and 9.0 can be reclaimed with one rice crop only after which gram, barseem, sugar cane, cotton or even wheat may be grown. Soils with salt content less than 0.5 per cent and pH value between 9.0 and 9.5 generally require two rice crops. Rice and barseem crops can tolerate the alkaline salts to a great extent and also reduce its quantity from the soil where grown.

(d) Some chemicals such as gypsum (calcium sulphate) has been used but this process is very expensive. Gypsum does not act in the presence of excess of sodium salts.

Soil Erosion. The washing away of top soil by the action of floods, rains or winds whereby the soil loses its agricultural productive qualities. The method adopted for prevention of soil erosion is called *soil conservation.* The productive qualities of a soil may also be impaired by excessive use of artificial fertilizing manures; using wrong methods

for the rotation of crops and removal of the natural cover of grass and forests from the ground. Methods adopted for controlling soil erosion or for improving the soil are : Holding irrigation or rain water over the land for long periods or making it flow at a very slow velocity by constructing small bunds or terraces; growing small plants on the fields at all seasons to hold water ; making temporary dams of brushwood against streams and by making Detention Basins.

11. STORAGE OF RAIN-WATER FOR IRRIGATION

Study of Rainfall. The maximum rainfall of a year and the minimum rainfall of another year of a place has been found very variable. According to Binnie, the maximum annual rainfall of a place is 1.51 times the average annual rainfall, and the minimum annual rainfall is 0.60 of the average annual rainfall. Average annual rainfall is the mean of 35 years. For two consecutive years, the maximum rainfall of both the years together will be 2 × 1.35 × average annual rainfall ; and the minimum rainfall will be 2 × 0.69 × average annual rainfall. For three consecutive years, the maximum rainfall of all the three years together will be 3 × 1.27 × average annual rainfall ; and the minimum rainfall will be 3 × 0.75 × average annual rainfall.

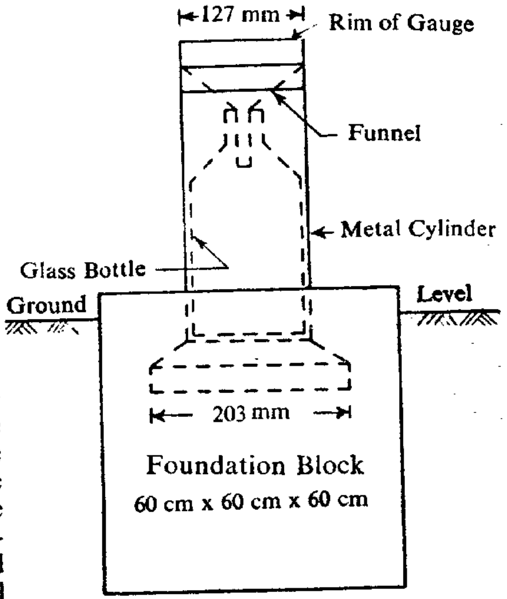
Blandford, however, found that the maximum annual rainfall at a place may be between 1.24 to 2.54 times the average annual rainfall of that place and the minimum annual rainfall may be 0.27 to 0.78 times the average annual rain fall. This is called the driest year. The mean of yearly rainfall of about 10 to 12 consecutive bad years (average bad year) may be 2/3 to 3/4 of the average rainfall of the place (based on records of 35 years). The year in which the rainfall is less than the average annual rainfall, is called a *bad year*, and the year in which the rainfall is more than the average annual rainfall, is called a *good year.*

For irrigation projects the rainfall of an "average bad year" is taken which is mean of the lowest yearly rainfall of a number of consecutive years Out of the total rainwater, the run-off in an average catchment is only about 30 to 40 per cent and out of this total run-off the storage possible may be as low as 10 per cent (and as high as 80 per cent) with about 40 to 50 per cent as average. Out of the water stored in a reservoir, the losses due to draw off, evaporation and leakage may come to about 50 per cent. Thus, only about 10 per cent (av.) of the total rain-water is available for irrigation. (Also see under "Seepage and Evaporation Losses").

Determination of Annual Run-off. or Flow off of a Catchment :

There are a number of formulae and methods such as : Binnie's percentages, Barlow's tables, Strange's tables, Inglis's formulae, Khosla's formula, Vermuele's formula, which are more or less based on the area of catchment and the annual rainfall, and are derived as a result of their observations from certain particular catchments in different parts of the country and their suitability is limited only to those catchments. There are so many variable factors affecting run-offs from different catchments that it is almost impossible to get correct results from any of the formulae. The results of one catchment cannot be applied to another. Therefore these formulae and tables must be used with caution if at all applied. The rational method given under "Drainage" may be used. Also see under "Waterways for Bridges".

Rain Gauge. The standard rain gauge used in India is called the Simon's rain gauge as shown in the illustration. It consists of a metal cylinder 5" (127 mm) in dia with its base enlarged to 8". (203 mm). A funnel 5" (127 mm) in dia. is fixed over the cylinder. The funnel shank is inserted in a glass bottle which receives the rain water. The water of the bottle is poured into a measuring glass supplied with the rain gauge, which gives the number of centimeters of rain that has fallen over the funnel. The rain gauge is fixed in a concrete block in such a position that the rim is 12" (300 mm) above the ground. A rain gauge should be fixed in an open place far removed from trees, buildings and other obstructions. The rainfall is measured every day at 8 a.m.



Rain gauge stations should be evenly distributed over the area of which the rainfall is to be measured. In plain country, one rain gauge may be fixed for an area of about 130 sq. km and in hilly country according to the nature of the site. Automatic rain gauges are available which record all the rainfalls on a graph paper fixed on a rotating drum, by a pencil point. Such rain gauges are useful for recording intensity of rainfall.

12. MAINTENANCE OF CANALS

Silt Clearance and Berm Cutting

In view of the Lacey's theory of regime flow in alluvial channels it may not be necessary to clear silt to the theoretical bed level and cross-section when a channel is taking its full supply and is irrigating its allotted area. If a channel is not doing its work properly, it may be sufficient merely to clear a portion of the silt to get it into efficient working order, or it may be necessary to clear to the full theoretical cross-section. Where it is proposed to do general silt clearance it should only be done after preparing a long-section of the silted bed and marking a proper-bed slope. Bed silt may be thrown on the back slopes

of the banks or spread out evenly in the neighbouring borrow pits but should not be heaped on the top of the bank or thrown in lumps on the outside. It should generally be thrown on the back slope of the weaker bank to strengthen it; if both banks are equally strong, it should be thrown on each side alternatively. The silt should, never be thrown on the inner slopes of banks or where it is likely to be washed back into the bed or blown back into the canal by winds. Advantage should, however, be taken to utilize all good excavated silt in repairing and improving the banks. Berm silt is an excellent material for bank repairs of all kinds. Where silt is used for raising banks it should be covered with 15 cm of good earth. Where general clearances are in hand it should be seen that silt and rubbish are cleared from under the bridges. Silt should not be cleared below falls but if outlets in such places are drawing more water due to a raise in water surface, they should be raised.

Overhanging berms must be cut off at a slope of 1/2 to 1, else they will fall in when the channel is running. If lowered continuously 7.5 cm below "full supply level" they will become uniform next year. Such earth will be useful for repairing banks which are low or narrow.

Before starting work on either the bed or the berms of channel they must be lined out with flags and string. Berm cutting should not be started until profiles have been cut and the lines carefully lock-spitted All irrgularities and kinks in the alignment should be straightened and all curves eased off where scouring or silting takes place.

When a channel is first opened after clearance a low supply should be run for a few hours at least and the gauge then gradually raised according to requirements. For regulating supplies to them, distributing heads should be planked up from the bottom so as to keep out silt. If the head has two more bays, an equal number of planks should be kept in each.

Repairs to Banks and Roadways. All holes and ravines should invariably be opened out to the bottom with stepped slopes and cleaned of all earth etc.; wet earth should then be rammed with wooden rammers in layers 30 cm thick, allowance being made for settlement. Silt from canal berms may be used where such berms exist, or from the spoil bank. Where silt is taken by cutting away berms, care should be taken that a layer of at least 15 cm thick of silt next to the bank is left untouched and also cross dowels at close intervals to permit the berm silting up again quickly. Any bank which is to be widened should be cut into steps as has been explained in detail under "Earthen Embankments". Driving banks should not be raised with silt or sandy earth from silt berms but earth should be taken from spoil banks; borrow pits should not be made on the top of spoil banks as in wet weather they form into tanks and may lead to damage by breaching. The outer edge of the roadway should never be dag-bailed as it will stop flow of water. A suitable drain should be provided at the toe of the bank to drain away the storm water. (Also see under "Banks": "Service Roads", "Dowels".)

Water Weeds and their Eradication

Aquatic plants are a great menace to agriculture and they spread throughout the irrigation system, in canals, reservoirs and tanks, etc. They hinder the free flow of water; weeds have been known to grow to such an extent that the discharge reduces to less than 15 per cent and canals cannot be run without restoring to clearance. Weeds render the water of the ponds and streams dirty and in some instances contribute an excess of organic matter which is very harmful if the water is used for domestic consumption. They increase the evaporation losses from reservoirs to a great extent. Weed growth is not uniform throughout the year.

Canals fed from storage reservoirs or artificial lakes grow more weeds than those fed directly from rivers, and trouble also abates during monsoon periods. Where the stagnant water cannot be drained off completely when the canal is closed, the weeds will not die off and may even increase. Aquatic weeds can grow in clear water up to depths of about 5.5 m below the water surface. Exclusion of light reduces weed growth, which can be due to turbidity in a canal water; there are no cases in which the weeds grow where the water is turbid throughout the year. Where natural turbidity in the parent river is inadequate, it can be created by providing a pick-up-weir designed to hold about 3 days' supply, for range to 60 cm to 1.8 m of water.

Fairly high velocities should be maintained which are not likely to deposit silt and make the water clear creating favourable conditions for weed growth. Weeds grow luxuriantly where silt is depositing and the water is also relativley clear. Velocities below 60 cm/sec. should not be allowed, but channels should be designed and maintained with velocities higher than the regime velocity as far as possible so as to keep silt in suspension and not allow it to settle and for which also deposition of seeds and cuttings of the weeds does not occur. When canals are newly opened they are generally run with low discharges and water has to be headed up at the regulators. This reduces the velocity and causes deposition of silt especially along the banks rendering the water clear, so that all conditions are then favourable for weeds growth. Thereafter the weeds tend to persist.

Temperature between 20 deg. C and 30 deg. C has been found to be highly favourable for weed growth but extremes of temperatures have a depressing effect. There are many varieties of weeds which are formed above and also under the water surface which may have to be removed either with silt clearance during the off season when the canal is closed or more frequently, which may be even after 45 to 60 days during summer, and the canal also will have to be shut off. No mechanical method of weed clearance has been found which is as satisfactory as doing the work by hand. Chemical methods are also used as described under "Water Supply". Sometimes "Rush Rotations" method is adopted. This term is applied to running a canal at full supply for sometimes and all dry at other times. Long interval closures also destroy weeds and have adverse affect on their growth.

Remodelling of Channels

A detailed hydraulic survey of the channel is necessary from which longitudinal and cross-sections are plotted. The sections should show: Existing full supply and bed levels, crest levels and working heads of outlets, water levels in water courses prevailing when high level fields are irrigated, dimensioned details of banks. Proposed bed levels, working heads and water levels in water courses and banks should be shown in different colours on the same section paper. The proposed channel sections may be within 20 per cent of the Lacey's regime sections. For longitudinal sections the horizontal scale may be 3 cm to a kilometre and vertical scale 1/50.

13. WATER ALLOWANCE AND DUTY

For preparation of an irrigation scheme the amount of water required is based on the following considerations :—

(i) *Crops*: Some crops need more water than others and this quantity is very variable; (ii) *Soil*: Sandy porous soils will take about 2 to 3 times more water than clay. (The light textured soils require less water per watering than the heavy textured soils, but the total amount required by heavy soils will be smaller than light textured soils.) (iii) *Seasons of the year*; (iv) *Sub-soil water level*; (v) *Method of water distribution*; (vi) *Condition and Design of the channels and the distance the water has to travel*; (vii) *Rainfall*; *etc.*

Water Allowance is defined as the number of cusecs (or cumecs) of outlet capacity, authorised per 1000 acres (or hectares) of culturable irrigable area. The water allowance, therefore, not only determines the size of an outlet for any area but also forms the basis for the design of the distributing channels in successive reaches.

Too much of watering is harmful and no useful purpose is served by supplying water beyond the quantity required for maximum yield for the particular crop and which can saturate the soil. In a porous soil the extra water will go down to the sub-soil watertable carrying with it the useful salts and shall be a waste. If the soil has impervious substratum the extra quantity of water is likely to create water-logging which would affect the yield of the crops or might bring up harmful alkaline salts to the surface.

The Gross Area Commanded by the canal is fixed from which the Culturable Area Commanded - CAC (Irrigable Area Commanded minus the unculturable area within it) is made out, and then the percentage of the area to be irrigated during different crop seasons and the water required for the same is computed.

The gross requirement represents the total supply of water which is to be arranged for a system and is the nett requirement for plant growth, percolation and evaporation losses and surface waste, etc.

The area to be irrigated per kilometre of a channel is not ordinarily in excess of 150 hectares and the percentage of Rabi to be irrigated is between 4 to 6 per cent of the CAC per week for max. period in which one watering is assumed to be completed. Depth of each watering is usually from 6 to 10 cm depending on the kind of crop.

Water-turn — Means any particular crop or area will have its turn for water after so many days in accordance with a programme.

Crop Ratio—This is the ratio of the area under different crops of a particular channel to the entire project area to be irrigated.

Duty of canal water—Duty denotes the irrigating capacity of a unit of water and is the relation between the area of land served and the quantity of irrigation water used. Duty varies according to the kind of crop, nature of soil, climatic conditions, canal conditions and the method of cultivation, etc. As the water requirement of a crop varies greatly at different times of a year, the duty worked out for one crop period is an average duty.

Base period or Base is the period during which water is supplied to the crop—whole crop season. It is usually reckoned in days.

Delta is the total depth of water required by a crop to come to maturity. It is generally denoted by —.

From duty or delta for a crop we get an idea of the amount of water required by the crop.

Relationship between Duty Delta & Base Period

In metric units :

One hectare-metre is the volume of water standing one metre deep over an area of one hectare. (One hectare is 10,000 sq. metres).

One cu. m/sec. running for base period B, provides a volume of water$=1\times60\times60\times24\times B$ cu. m/sec.

The number of hectares that can be irrigated by one cu. m/sec. running for base period B

$$\frac{1\times60\times60\times24\times B}{1\times10{,}000\times \text{delta}}=\frac{8.64\text{ B}}{\text{delta}}=\text{Duty}; \qquad \text{Delta}=\frac{8.64\text{ B}}{\text{Duty}}$$

one cumec—day=8.64 hectare-metre.

1 acre=0.4 hectare , acre-ft. x 1233.48=cu. metres

cusecs x 0.028=cu. m/sec.

Duty is said to be "low" when large amount of water is applied to irrigate a small area of land, and it is said to be "high" when small volume of water irrigates a large area of land. Since different kinds of crops require different amounts of water therefore duty is different for each crop.

Duty is generally reckoned at the head of the water course (or the field channels). Duty reckoned at the head of the distributary will be less (than at the head of the water course) and that of the head of a canal will be lesser still. Duty goes on decreasing as we go higher up along the distribution system due to the loss of water in transit.

A knowledge of duty for various crops is helpful in the design of irrigation channels in a project. The area that can be irrigated is worked out from the quantity of water available at the head of canal and the over-all duty for all crops. It also helps to check the efficiency of working of a canal system. The duties and deltas adopted in various parts of India are very variable. Exact figures should be obtained locally.

Methods of Irrigation

Basin Irrigation—It consists in flooding the area with a large quantity of water at a time. A greater quantity of water is required and the duty is less.

Flood Irrigation—The plots of land are flooded with water for a small depth. This requires more water than furrow irrigation but less than basin irrigation.

Furrow Irrigation—Water is applied to a field by means of small, narrow field channel, and is suitable for crops grown in rows. Since the water is not applied to the entire area, the consumption of water is less.

Perennial Irrigation System—In this system of irrigation, the water required for irrigation is supplied in accordance with the crop requirements throughout the crop period. Here some storage works sueh as dams or barrages are required to store the excess water during floods and release is to the crops as and when required.

Advantages and Disadvantages of Tube Well Irrigation :—

Advantages: Isolated areas can be irrigated; water is availabie all the times and is under control; tube wells lower the sub-soil water level of water-logged areas.

Disadvantages : Tube well waters have no fertilizing qualities of silt. Initial cost, working and parts replacement expenses are higher than those of the flow irrigation. There are more possibilities of breakdowns.

The water of a well should be tested for its salt contents.

Classifications of Crops :

(a) Heavy crops (perennial—12 months)
(b) Medium crops (seasonal and two seasons)
(c) Light crops (hot weather and seasonal)

(i) *Bases of crops* (or Base periods) *in the Punjnb and Northern India* :
Kharif crops : April to Sept.—183 days (summer crop)
Rabi crop : Oct. to March—182 days (winter crop)
Zaid Kharif : Is late summer crop – Sept. to Jan.
Zaid Rabi : Is late winter crop—Jan. to April.
Perennial : Is twelve months crop.
Usual ratio of Kharif area is one-half of Rabi area.

(ii) *Bases of crops in the Bombay Deccan* :

Kharif (monsoon) 15th June to 14th Oct.—122 days
Rabi (winter) 15th Oct. to 14th Feb.— 123 days
Hot weather 15th Feb. to 14th June— 120 days
This is also sometimes known as "Crop Periods".

In South India, Rabi and Kharif seasons are not quite distinct, and there is no marked distinction between the various seasons.

In Rajasthan and the areas round about, six seasons are reckoned each of two months.

In the remaining parts of India three seasons are generally observed where monsoon lasts for about 4 months.

The usual depth of first watering from a well is 75 cm in a ploughed field and the subsequent waterings are of 5 cm depth each.

Kharif crops require about twice to three times the quantity of water required by Rabi crop.

In designing a distributary, an allowance of 25 per cent should be added to the above, for the requirement of water is not spread evenly over the whole period, and to allow for the time factor this is multiplied by about 10/7 to arrive at the final figure of discharge for which the distributary should be designed.

14. IRRIGATION OUTLETS

Explanation of Terms

An *outlet* is a device (fixed or regulating) built at the head of a water-course and which connects the water course with the distributing channel and controls the flow of water in the water-course It is a connecting link between the irrigator's channel and the Govt. irrigation system. Ordinarily, outlets are not built in the main line or branches, when built, such outlets are termed "direct outlets" and are usually rigid modules. Distribution of water is carried out by means of outlets and they provide a measure of discharge passing through them. Most of the irrigation departments have their own standardized and officially approved discharge tables for outlets. An irrigation outlet may be :—

A Modular outlet whose discharge is either independent of the water levels in the distributary and the water-course and which ensures constant or fixed supply within certain working limits, or the discharge depends on water level in the distributary only. The former outlet is called a *rigid module* or *absolute module* while the latter is called a *semi-module* or *flexible module*. These outlets are used where charges for water are made on the basis of quantity supplied.

A modular outlets is necessary for equitable, proportionate and economical distribution of irrigation water. This type of outlet entails greater loss of head than non-modular outlet and is also more costly.

A *Semi-Moduler outlet* is that outlet whose discharge varies according to the water level in the supply channel but is independent of the fluctuations of water level in the water-course (or downstream channel), so long as the minimum working head required for the working of the module is available. In irrigation channels mostly semi modules are used so that all the outlets (in the head or tail reaches) draw proportional discharge according to the water level in the parent channel. A semi-module may be a weir or an orifice having a free fall. Pipes working under free fall conditions are semi-modules.

A non-adjustable proportional semi-module outlet can work in non-alluvial soils as there is no silt problem.

A *Non-Modular outlet* is an outlet whose discharge is dependent upon (and varies with) the water levels in the supply distributary on its upstream side and the water-course on its downstream side. Such outlets usually consist of a rectangular opening and a pavement. The discharge is not constant also due to silting, and high lands get less water. They are usually used for channels serving lift areas.

A non-modular outlet may be an orifice (pipe or barrel-submerged) type. It is controlled by a shutter on its upstream end ; arrangement is provided to lock the shutter in any required position so as to have a given discharge through the outlet. The main disadvantage of a non-modular outlet is the ease with which the cultivators can increase the discharge passing through it by silt clearing of the water-course and thus increasing the head.

Rating Flume—An open conduit built in a channel to maintain a consistent regimen for the purpose of measuring the flow and developing gauge discharge relation.

A flume is based on the principle of a Venturi Meter, but it is in the form of a taper in an open channel, which increases the velocity, resulting in a difference in head between the upstream side and the throat of the flume.

Flume—A flume is a narrowed waterway (or a narrowed channel), usually built in masonry for measurement of the discharge. *Fluming* may, therefore, be defined as reduction of waterway below the normal, and this is accompanied by increase in velocity (and loss of head). Flumes are divided into two main classes : flumes with "free" water surface or open flumes and covered or roofed flumes as *venturi flumes*. The purpose of fluming is to effect reduction of section without causing extra loss of energy, as at a sudden change in the cross-section eddies are formed which entail loss of head. The exit of a flume (outlet) is designed in such a manner (by gradually expanding) that no standing wave is formed and there is no loss of head. Fluming if properly done may enable the water to regain nearly the entire head lost at the entry. In a venturi flume the inlet contracts gradually and the outlet expands with a (depressed) throat in the middle. An outlet in which a standing wave is formed, is semi-modular. When the head available to use a standing wave flume as a semi-module is not sufficient, venturi flumes are used. In the case of a standing wave flume, the discharge is a function of the upstream depth only, in venturi flumes the discharge varies with the difference of water levels upsteam and in the throat. Open flumes are suitable for the tail reaches of channels and preferably in the form of distributors and clusters.

Control or Rating Flume. A flume for measuring the discharge in a channel.

Meter—A meter is a measuring device and in irrigation engineering its use is restricted to structures installed for measuring discharge of channels.

Meter Flume—The device for measuring discharge from the direct measurement of the depth of water flowing over it.

Surface Outlet—Is a weir or flush escape which allows the surplus water to escape above the full supply level of the channel.

Module—A device for ensuring a constant discharge of water passing from one channel into another irrespective of the water level in each, within certain specified limits. (ii) A device for measuring or controlling the flow of water.

A P M (Adjustable Proportionate Module)—A semi-modular submerged orifice outlet with an adjustable roof block designed to distribute small variation in supply proportionately when set at the correct level. The discharge varies as the depression of the roof block and not as the head measured from the distributary water surface to the centre of the orifice.

Outlet Setting—Adjustment of the discharge of an outlet by altering the working head. The setting is the ratio of the depth below FSL of the sill or bottom of pipe of an outlet to the full supply depth of the channel at the point.

Minimum Modular Head (Modular Limits)—The minimum working head required for the modular working of an outlet. It is the difference of water level between supply and delivery sides which is the minimum necessary to enable a module or semi-module to work as designed. All modules, whether rigid or flexible, require a certain minimnm head to ensure modularity (i e. its working as a modular outlet). In the case of rigid modules there is also an upper limit (max. modular head) beyond which consistancy of discharge fails. Minimum working head normally required for an outlet is 0.2 × the full supply depth of the distributary. This is what would be required for an open flume outlet set at bed level.

Working Range or Range of Modularity—Modules work as modular outlets within certain limits of water levels in the distributary and the water course; the range over which each module works as a modular outlet, is called its working range or range of modularity.

Depression or Depression head—The depth of a point below the supply level at a semi-module in terms of which the discharge can be expressed in the form of a hydraulic formula. In the case of an open type module the depression head is measured up to the sill level and in the case of an orifice module up to centre of the orifice.

Depression Ratio—The ratio between the depression head and the height of the opening. The depression ratio is considered only for an orifice type module as it is unity for an open type module.

Outlet Area or Chaks—An outlet chak is the area included in the irrigation boundary of an outlet. The discharge of an outlet is worked from CCA in the said chak based on the permissible intensity of the irrigation and the water allowance. The maximum discharge of an outlets is 85 litres/sec. and the minimum 28 litres/sec. The length of the water-course irrigating the chaks should not be more than 3 (max:5) km.

Proportional Moduling—The fitting of semi-modules on a supplying channel in such a manner that when supply fluctuates each off-take draws always a constant proportion of the supply.

Rateable—An outlet is said to be rateable when it can be rated or set to give a particular fixed discharge under a given set of conditions.

Modular Limits—The upper and the lower limits of any factors beyond which a module or semi-module ceases to be acting as such.

Min: Modular Loss—The minimum loss of head or the difference between the supply and delivery water levels which is absolutely necessary to be maintained to enable the module to pass its designed discharge.

Working Head—The difference between upstream and downstream water levels, or those of the supply channel and water-course.

Flexibility—Is the ratio which the rate of change of discharge of the outlet bears to the rate of change of discharge of the distributary.

Sensitivity—The ratio that the rate of change of discharge of an outlet bears to the rate of change of water level in the supply channel when referred to the normal depth of water in it.

(In the case of 'flexibility', while taking the ratio, the discharge in the supply channel and the outlet is considered, while in the case of 'sensitivity', the level in the supply channel and the discharge in the outlet are considered.)

Sensitiveness—The variation (per cumec) of discharge of a semi-module for a tenth of a metre variation of supply level.

Partially Rigid Module—An outlet which works as a semi-module up to a certain level of water (usually full supply level in the parent channel) after which its discharge remains constant.

Rigid Module—Module passing a fixed supply A rigid module without moving parts is used.

Drowning Ratio—The ratio between the depth over the crest of water level downstream to that of water level upstream of the outlet. (Also see page 17/6).

Submerged Orifice—An orifice which in use is drowned by having tail water higher than all parts of the opening.

Efficiency of an outlet is the ratio of the head recovered to the head put in. In the case of the weir type of outlet, efficiency is the same as the 'drowning ratio'.

Tail Cluster—Outlets (two, three or four) made at the tail of a supply channel. The sill level of side modules is sometimes built 6 mm lower than the central module and gauges are fixed with their zeros at the crest level of the central module to compensate the side modules for reduced velocity of approach. The crest of all the modules (at the tail) should be at the same level so that any change of water level in the supply channel may affect them all equally, as there is low depth of water in the tail of a channel. The outlets at the tail cluster should be of the open flume type.

Gibb's Module—Is a device for obtaining constant discharge even if the water level varies in the supply channel up to a certain limits. The water is led through a bell-mouth inlet pipe into a curved rectangular trough called eddy chamber. A number of baffles are inserted in the eddy chamber with their lower edges sloping at the required height above bottom, to prevent increased discharge passing through the module. It is a rigid module and has no moving parts. It is a costly structure built of RCC or masonry, and it draws silt in alluvial soils from the distributary which affects its working. The Poona Research Station have standardized the design of Gibb's module.

Portable Flumes were made of gal, iron sheet according to certain prescribed designs and were considered to measure accurately discharges up to 85 litres/sec. The gauge was provided on one side in feet and on other side in cusecs. This flume is used for measuring the discharge of water-courses and can be carried and fixed easily.

The Optimum Capacity of an outlet is the discharge which the cultivator can handle efficiently and should be such that the absorption losses in the water-course and in the field are a minimum. The optimum capacity of an outlet is fixed in relation to the size of the field, depth of watering, the rate of evaporation and absorption into the soil and the time taken to irrigate the field. Outlets along distributaries spaced at about 200 to 400 metres apart are found to be adequate and suitable.

The Requirements of a good Outlet:

(i) It should be simple in design, construction and working, having no moving parts liable to derangement by rubbish, weeds or silt, and should require no periodical attention. (ii) It should be strong enough so as not to be easily tampered with by the cultivators, and any interference should be easily detectable. (iii) The outlet should be able to draw its due share of silt carried by the parent channel. (iv) It should take proportionately more or less discharge with the varying supply in the parent channel and should work efficiently with a small working head. (v) It should take steady automatic discharge of the designed valume, not withstanding variation of head, within fixed limits.

Semi-modules satisfy most of these requirements and are generally preferred to rigid modules but have comparatively greater loss of head.

Design of Outlets:

The outlets should be fixed at right angles to the channel and at correct levels. The required loss of head should be less than the available difference of level between the water-course and the channel. To prevent their being lowered by cultivators, outlets should rest on concrete or masonry pillars at each end. Iron frames should be used to make the tampering with the openings difficult No direct outlets should be given from the main canal or branches but should take off from distributaries and minors, as regulations of supplies is very difficult from the main canals. The outlets should be able to work with only a small loss of head

The modules most commonly used are open flume and orifice types. The clear overfall outlets are the best but they require more loss of head. The orifice type of modules require more loss of head than the open type and are better located in the head reaches where the depth of channel is considerable. Minimum working head normally required is 0.2 × the full supply depth of the distributary; this is what would be required for an open flume outlets set at bed level. To safeguard against the heavy fluctuations, a minimum depression head of 1.5 is usually fixed.

Silt Drawing Capacity of Outlets Capacity to draw silt depends upon the setting and design of the outlet.

Only that much quantity of silt should be admitted in an outlet as it can carry; but on outlet must draw some silt. Water-courses having steep gradients can take more silt and the outlets that have to feed a high command should take less silt. An outlet with its sill level at the bed of the parent channel (or lower) will draw maximum quantity of silt, a higher setting of the outlet will draw lesser silt. Modules with narrow and deep openings will draw more silt than outlets which are shallow and wide. A silt prevention device can also be provided at the head of the distributary. Distribution of silt and setting of outlets also depends upon the condition of the channel. If the channel is in regime, the setting of the outlets should be such as to make them proportional. Where a channel is silting, an open type module with its sill near the bed should be preferred but where the water-course has to feed a high command, pipe-cum-open flume should be used. It is generally considered that pipe outlets fitted at bed levels of distributaries seldom give any silt trouble.

Open Flume Outlets—General principles :

It is a smooth weir with a long (constricted) throat and a gradually expanding flume at the out fall. There are various types and their modifications. These outlets require very low working heads and are most suited to tail reaches or tail clusters and in the form of proportional distributors. The main disadvantage is that in many cases it is deep and narrow which is easily blocked, or is shallow and wide, which fails to draw its fair share of silt. Except in small channels it is seldom possible to place the crest of an open flume outlet with a normal discharge of less than 57 litres/sec at the bed level of the channel. The width of the outlet is limited to a minimum of 6 cm and, as such, it often becomes necessary to raise the crest of the outlet much above the bed level. The discharge is given by the formula :

$$Q = CBH^{\frac{3}{2}}$$

B is width of the throat; H is depth of water over crest;
C is a constant depending upon the width of the flume :

For B—6 to 9 cm	C = 1.60	C is in metric units
9 to 12 cm	= 1.64	Q is in cumecs,
over 12 cm	= 1.65	B, H in metres

Standing Wave Flume consists of a converging transitionally rising flume, an horizontal and level throat and a sloping and gradually expanding flume. The bed of the throat is called the hump. The throat on account of its large length acts as a broad crested weir. (See "Flume" in the preceding pages.) Depth of water above the hump is taken for calculating discharge.

For a constant discharge over a weir the stream lines in the flow at the critical section should be parallel. This condition in a flume is generally attained when the length of the throat is not less than 2H. For a narrow, long flume or with a comparatively large H (as in an outlet) the friction losses from the sides and the crest may be appreciable. In that case, to attain parallel flow, the crest should have a slope equal to the loss of head due to friction in the flume. In the formula given under "Hydraulics" for calculating discharge over a broad-crested weir frictional losses on the crest have been ignored.

Orifice Semi-Module (OSM)

Consists essentially of an orifice provided with a gradually expanding flume on the downstream side. There are various forms and modifications, the earliest of these is the *Crump's Adjustable Proportional Module* (APM) which was subsequently modified and named *Adjustable Orifice Semi-Module* (AOSM). It is a long throated flume in design more or less like the open flume outlet with slight changes in the shape in the upstream and downstream approaches and the throat. A cast iron adjustable roof block with bed plate is fixed on the upstream end of the throat, and size adjusted according to the desired discharge. The base plates and roof blocks are manufactured in eight standard widths. The silt drawing capacity is also controllable and when it can be set near the bed it is one of the best forms of outlets to adopt. The discharge for APM and AOSM is given by the formula :

$$Q = 4.04\ Bd\sqrt{H}$$

Q in cumecs
B, *d*, H in metres

B is width of the flume (throat); d is elevation of the roof block above the crest (vertical height of opening); H is the head measured from the water surface to the soffit of the block opening.

Pipe-Cum-Semi-Module

It comprises an iron pipe taking off a channel (on the upstream side) and opening into a tank about 1 m square on the downstream side of the bank. On the down-stream wall of the tank a module is fitted which may be a pipe working free fall, an open flume, an APM or an AOSM, set anywhere according to the design. Its efficiency depends upon the type of the module used. Silt draw into the outlet depends on the position of the pipe with respect to the bed of the channel and the pipe is fixed either horizontal or sloping.

Pipe or Barrel Outlets

The simplest and the oldest known type of oldest outlet is the earthenware pipe (colaba). Now cast iron or concrete pipes are commonly used. When a pipe discharges freely in air, it is semi-modular and when the pipe is submerged (in the water-course) it is non-modular. A submerged pipe outlet can pass the required discharge with a very small working head, even 30 mm, with which no semi-module can function. Pipes are usually embedded in concrete with upstream and downstream face walls (or head walls) of masonry of sufficient lengths to retain the earth slopes They are usually placed by the bed level of the distributary so as to draw their full share of the silt. The minimum diameter of a pipe is 75 mm, and 150 mm is considered standard size. The level of the pipe should be kept at least 150 mm above the level of the highest land command under the pipe. Discharge through pipes is calculated as given in Section 14 under "Flow Through Pipes"; friction and entry losses are taken into account.

In UP the outlet pipe is fixed 22.5 cm below the water surface level for distributaries and minors. Co-efficient "*c*" is taken as 0.63. For determining the outlet size for a given command the discharging capacities used by Irrigation Department of UP are give in the following table :—

Ventage (dia of circular pipe)	Average discharge in cumecs for			
	Free overfall	Outlet with submarged outfall		
		0 to 20% lift areas	21 to 50% lift areas	over 50% lift areas
15 cm	0.025	0.0185	0.0154	0.011
12.5 ,,	0.019	0.0140	—	—
10 ,,	0.011	0.0080	0.0070	0.006
7 ,,	0.006	0.0040	—	—

The friction loss can also be found from the tables and formulae given under "Hydraulics" and "Water Supply".

For practical purposes discharge is calculated from the simple formula :

$$Q=cA\sqrt{2gH}$$

If the outlet has a free fall, H is measured from the centre of the pipe or barrel to the supply level in the distributary. If the outlet is drowned, i e., discharges into a water-course in which the water level is above the top of the barrel, then H is the difference in the water level in the water-course and the ditributary.

The co-efficient "c" varies with the length and size of the pipe, its material, the shape at entry and other varying factors that influence the the discharge through a pipe outlet. The value of the co-efficient increases if the water level in the water-course rises and changes the outlet from a free fall into a drowned outlet. The co-efficient "c" varies from 0.63 for a free fall to 0.80 for a submerged outlet, but at the same time there is reduction in the head. The co-efficient is generally taken 0 63 for a cast iron pipe outlet with free fall and 0.74 for a submerged outlet. Allowing for the obstructions etc. in a drowned outlet, the value of "c" may be fixed at 0.70 instead of 0.74. So long as the head is more than double the diameter of the pipe, the error in using a co-efficient of 0.73 is small but the value falls off rapidly for small heads. These figures are taken for all sizes of pipes but are more correct for 150 mm pipes of encrusted iron or concrete and 3.6 to 4.0 m lengths. (Also see further).

Scratchley Outlet

This differs from the pipe outlet only at its down-stream end where the barrel opens into a 60 to 90 cm square cistern, at the downstream end of which a cast iron or stone orifice of the dimensions required for the design discharge is fixed in a masonry wall. The length of orifice is kept 1.5 to 3 times its shortest transverse dimension. As the pipe is fixed at the channel bed level, the orifice can be fixed at a higher level so as to ensure semi-modularity. Many engineers prefer the use of Scratchley outlet to the simple pipe outlet.

Sluices. The sluice type of outlet is used where variable supplies are required to be given at different times, as from tanks or reservoirs or inundation canals. Simple design consists of circular or rectangular pipe or vent under the embankment between two masonry walls at the ends, provided with sliding shutters which are generally with locking arrangsments.

Since supply levels in channels vary very often (thus changing the working heads), ideal conditions for outlet design are seldom available, which make the selection of outlet type difficult. A minimum working head is essential for the efficient working of outlets and the site for fixing the outlet has to be selected with respect to the commands of the outlets. Semi-modular types are recommended for general use. Non-modular types should be avoided as far as possible, but where the available working head is limited, the Scratchley type may be used.

15. MICELLANEOUS IRRIGATION STRUCTURES

Silt Excluders and Silt Ejectors

Are devices by which the entry of coarse bed-silt into a canal is regulated; silt is either extracted from the water entering the canal or is precluded from entering the canal. These devices are built at the head of the main regulator and at branch or distributary regulators. The silt excluding device built in conjunction with the Head Regulator of a canal on the ubstream side in the river is called a *silt excluder*, and when the silt excluding structure is constructed across a canal at a point after the head regulator, it is usually called a *silt extractor or silt ejector*. The word extractor is often loosely used for both the excluder and the ejector. Silt ejectors in canals are more efficient than the silt excluders at the head of a canal because conditions necessary for silt exclusion can be built more easily in the canal bed than in the river bed.

At the time of high floods the head regulator of a canal is closed to avoid the canal being silted up by the heavily silt laden water of the river. Silt observations are made at all headworks for the quantity of silt passing into the canal and when the percentage of silt charge entering the canal exceeds a certain figure fixed for each case, the canal is closed. The silt charge that a canal in regime can carry is known as *regime charge*, and which should not be exceeded to avoid silting up of the canal. There have been instances where silt carried into the canal during high floods so depleted its capacity that it could not carry the water needed for irrigation and it became necessary to close canal and clean it during the height of irrigation season at great expense. The basic principle on which silt excluders or silt extractors are designed lies in the fact that in a flowing stream carrying silt in suspension the concentration of the silt charge and silt grade in the lower layers is higher than in the upper ones. Conditions are created to provide concentration of silt charge near the bed and also for the silt to settle into the lower layers, by reducing the velocity, providing a smooth surface at bed and sides to reduce friction, and by taking the water at a gentle angle so as not to cause any turbulence which may stir up the silt. The top silt-free water is admitted into the canal and the lower high silt-laden water is passed through tunnels and under-sluices with a high velocity without any disturbance.

The following methods are adopted for silt control at Head Works:

(a) A dividing wall or raised crust is provided in the river on the side the canal takes off and a pocket or pond is created in front of the scouring sluice between the head regulator and the dividing wall. Silt deposits in the pocket and only clear water enters the canal. When sufficient silt has accumulated in the pocket it is scoured out through the sluice gates. This is called *Still Pond System*.

(b) Scouring sluices or under sluices are provided in the main weir wall. Silt of the head regulator is made at a higher level than the bed of the approach channel, or the silt of the scouring sluices is lowered, so as to admit only the clear top water into the canal.

(c) Silt excluders are provided in the pocket of the scouring sluice just above the head regulator. These consist of tunnels which are channels 1.8 to 2.4 m wide with top covered with RC slabs. The top of the slab is at the sill level of the head regulator. The silt excluders extract silt from the water and lead it to the river or other natural drainage. The channel between the extractor and the river through which the escapage water is discharged is known as *Tail Race.* The tail race should have a steep gradient.

(d) Pavement in the approach channel is provided so as to reduce disturbance. (e) Velocity of water at the intake is reduced by providing wider head regulator (f) Upstream noses of head regulators are bent. (g) A small discharge is admitted into the canal when the water is very turbid, and a large discharge when the water is clear.

Silt Extraction from Canals

Silt Ejector consists of a number of piers 60 to 90 cm high, for the full width of the main canal which are covered with an R/C slab to form an under tunnel Two or three sets are provided one below the other in the first reaches of the canal and at the end a cross regulater is constructed with a weir at the downstream side of the canal, and the silt-laden water is lead off to the river through the under tunnel.

Sometimes ejectors are made in the form of a *Saddle Syphon* which is a syphon capable of being primed with a small additional discharge for flushing out the silt. A guide vane and an inlet tunnel is provided for the proper functioning of the syphon.

The canal length between the head regulator and the ejector is called an *Approach Channel.* Generally a large quantity of silt is deposited in the first 8 to 16 km of a channel. The first reaches in a canal should be given steeper slope to produce a higher velocity to carry down the silt.

Silt Escape is only a weir wall with its top at FSL provided with vents and gates, having the silt of the opening of the vents about 30 to 60 cm below the bed level of the channel.

Silt Traps: Pits or basins are formed in the bed at the canal inlet, which on account of increased section reduce the velocity of water and induce deposition of silt, which is flushed out periodically through an escape.

Silt Vanes are walls or obstructions made to obstruct or divert current to exclude silt entering the smaller channel. The vanes are made parallel to each other and are either straight or curved and are of short height only.

Reservoirs

To prevent silt from entering a reservoir, the system of serial reservoirs (serial tanks) is recommended. Here the uppermost reservoir gets silted up and the other reservoirs are left free. Vegetation in the catchment area will also arrest silt flowing into a reservoir.

Strengthening of Canal Banks by Silting

(Also see under "Spurs & Groynes").

The In-and-Out system: Additional parallel banks are constructed outside the original canal banks. Cross banks are put at Intervals varying from 150 to 300 metres to connect the two banks and form series of compartments. These compartments are provided with inlets from the canal at the upstream end and outlets at the downstream ends. (Inlets and outlets are at convergent and divergent angles with the bank.) A part of the canal water is passed through these compartments and silt is thus deposited.

The Long Reach system; External parallel banks are constructed as for the "In-and-Out" system with cross banks at intervals of 1200 to 1500 metres. Inlets are pìovided to the compartment at its head and outlets at the tail of the compartment. Whole of the canal water is diverted into one compartment at a time and the canal contiguous to the silting compartment is blocked at both ends to increase the silt deposit. This system is not so efficient as the first one.

The Internal silting system: Is for new canals. The banks are set back a small distance from the normal section and enducements (groynes) are constructed to encourage the deposit of silt internally on the berms.

The above methods are adopted for strengthening banks of soft and pervious materials. To obtain satisfactory results with a long series of reaches, each reach should be closed and made reasonably secure by obtaining sufficiently heavy deposit and then only other reaches taken in hand.

Cross Drainage Works—Aqueducts and Siphons

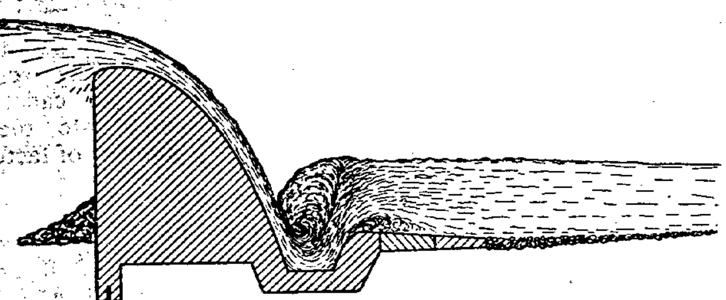
An *aqueduct* literally means a channel for conveying water and it may be either above or below the ground. In irrigation engineering the term is confined to mean a structure carrying an irrigation canal over a drainage channel without having to lower down the bed of the drainage channel for the crossing. The aqueduct is a culvert or a slab drain when the canal is carried within earthen embankments without any masonry retaining walls over the cross drainage work. These are suitable for crossings over small streams and where the difference of level between the bottom of the canal and the high flood level of the drainage stream is small. When the canal is led under the drainage channel, and the level of the drainage is so much above that of the canal which does not require dropping down the bed of the canal, the crossing is termed *super-passage.* When the bed level of the drainage channel has to be depressed below its natural level to pass it under the canal (aqueduct) it is called a *siphon aqueduct* but when a canal is similarly passed below a drainage channel or another irrigation channel, the work is termed *canal siphon* or *siphon* which is really an inverted siphon as the canal is dropped down below its general level and raised again. When the bed of a channel is dropped at the entry to the crossing and is not raised back at the exit but continues at the depressed level it is not termed a siphon aqueduct but simply an aqueduct.

The best site for cross-drainage works is where the drainage and canal cross at right angles with fairly straight lengths at both upstream and downstream sides. A good rule is that the straight length on the upstream side should be not less than 10 times the bed width of the stream in case of a small and quick flowing, and 5 times for a large and slow moving stream; and the straight length on the downstream side should be double that on the upstream side. The waterway required for the drainage channel should be calculated as explained in Sections 16 and 19 and some extra should be taken in the form of freeboard or clear headway above the anticipated highest flood level to prevent blocking up of the waterway by silt or debris.

The velocity allowed in an aqueduct is 1.5 m per sec. or twice that in the channel, whichever is less, which is obtained either by increasing the bed fall in the aqueduct section or giving a fall at the inlet. The waterway can also thus be made narrower and cost reduced. No advantage may however accrue by reducing the width of a short aqueduct. The structure may be built of masonry arches, RC slabs or box culverts. Size of abutments and thickness of arch may be worked out as explained in Sections 7 and 19. Abutments should be designed to resist saturated earth pressure; the width may be 1/5th of the depth of water greater than given in the rule in Section 19 to allow for the additional weight and water in the canal. It is usual to make the thickness of the roof 2/5 to 1/2 of the canal head upstream of the work. The roof should be safe against compression caused when the canal is flowing full, and also safe against bursting pressure or upward thrust caused by the water when the aqueduct is running full and canal is dry. This will be= (height of aqueduct opening+drop of water level in the aqueduct) × weight of water.

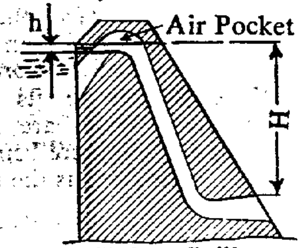
The foundations should be carried below the scour depth. Where the foundations have been so carried and the section of the drainage channel has not been reduced there should be no necessity of providing a pacca pavement in the bed of the culvert or aqueduct. Where, however, the section has been somewhat restricted which does not increase the velocity more than 2 to 2.5 m per sec., the bed should be paved with loose stones or concrete blocks. Where the section has been considerably reduced resulting in high velocity, solid pavement should be provided designed to withstand uplift pressure resulting from the head of water in the canal. Sometimes inverted arches (or curved bottom) are used for the floors to resist the upward pressure, which are supposed to transfer the pressure to the piers or abutments and thus the entire weight of abutments and superstructure helps in resisting the pressure. (It is doubtful whether the inverted arches do really act in this way. In bridges there is no appreciable head of water against the work, therefore inverted arches need not be provided.) RC box culverts can resist much more upward thrust. For small discharges hume pipes embedded in concrete can be used.

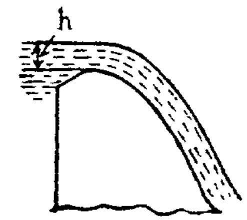
Siphons used are generally of two types : (i) vertical drop or well type and (ii) sloping approach and exit (iii) vertical drop at the approach with inclined exit. The vertical drop type is constructed by providing rectangular vertical drops or wells at the entry and exit ends with horizontal barrel in the centre. This type is suitable only where the water is clear, it entails a lot of loss of head and deposited debris are difficult to be removed. Sloping type is more suitable especially with muddy waters.



Pocket at toe of Spillway to prevent erosion

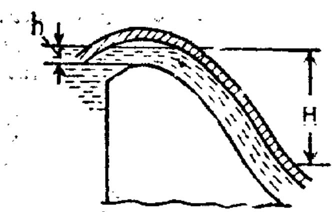
See Page 14/18





Siphon Spillway
Operating Head H

It is essential to have self cleansing or scouring velocity in siphons, *i.e.*, a velocity of 2.5 to 3.5 m per sec. to keep them clean of debris or silt. Entry and exit should be inclined at a slope of 1 in 4 and should be wide rather than narrow. The roof must continuously slope downward at the entrance and upward at the exit: as otherwise air will be entrapped in the barrel reducing the discharge. Discharge through siphon aqueduct can be worked out by the formula given in Section 14.



Syphon pipe flows (litres per second)

Diameter of syphon mm	*Head cm* 5	10	15	20
10	0.05	0.07	0.08	0.09
20	0.19	0.26	0.32	0.73
30	0.42	0.59	0.73	0.84
40	0.75	1.06	1.29	1.49
50	1.17	1.65	2.02	2.33

Cross drainage works of masonry may be splayed on the upstream side 1 to 1 or 1 to 2 contraction and downstream sides 2 to 1 to 3 to 1 expansion. In most of the works double sets of wing walls will be required for canal banks and drainage side slopes. Provide paved flooring for half the length of contraction upstream and three-fourth the length of expansion downstream.

Canal Head-Works

Head works comprise the construction of a permanent weir or a dam across the river along with other subsidiary works. The function of a head works is to control the flow of water in the river and to divert the same to the canals according to the requirements. The weir raises the level of the supply to increase command of the canal water. Arrangements are also made to control the silt entry into the canal. Selection of site for head works is governed by a number of factors.

Weirs (Defined under "Glossary of Terms.")

There are mainly two types of weirs according to the functions they perform. Weirs are usually of small heights, 3 to 4.5 m as distinguished from dams. (i) A storage weir, which is a high weir and its function is to store water at its back and then deliver it into the main canal when required. It has undersluices. It is an over flow dam. (ii) Intake weir is of low height and its function is to raise the level of the water on the upstream side and divert it into the canal immediately. Such a weir and its ancillary works are called *diversion head works.* Such a weir may be with shutters or counter-balanced gates on its crest to regulate the water level for diversion to the canal. Weirs are made of different shapes, e.g., vertical drop weirs; slope weirs, there being slopes both on the upstream and downstream sides. Weir floor is the most important structure of a canal head works. See Page 17/102

Bligh's Creep Theory for the Design of Weir Floors or Aprons

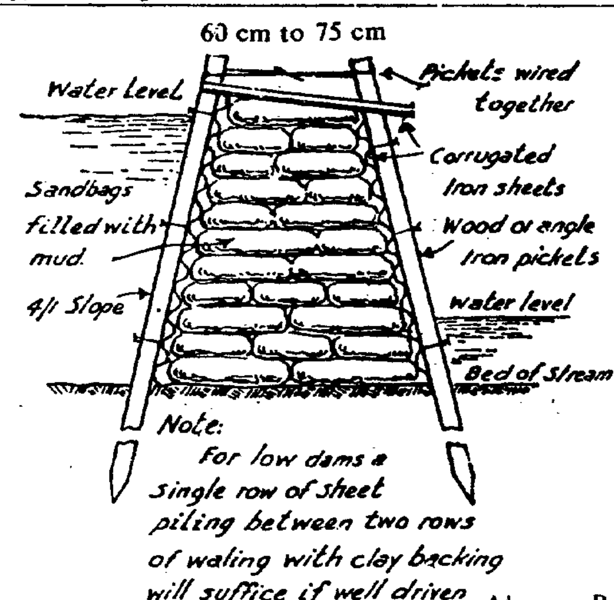
Pressure is exerted on the underside of floors, and which tries to lift up the floor founded on permeable soils, resulting from the movement of sub-soil water due to head of water produced from the difference of water levels on the upstream and downstream sides of a weir. Bligh assumed the percolation water to "creep" along the contact of the base profile of the weir floor with the sub-soil (inclusive of cut-offs). The water starts creeping from the upstream end of the upstream pavement and its energy or pressure goes on decreasing along the flow line in proportion to the path traversed which is different for various grades of soils. This sub-soil water ultimately comes out at the downstream end of the downstream pucca apron with a velocity known as *exit velocity* This theory was subsequently modified by Khosla and others according to which, when deep vertical cut-offs are provided the creeping of the water along the bottom of the cut-off wall becomes impossible.

Provision against failure by uplift by the sub-soil water, mentioned above, and the exit velocity (or exit gradient according to Khosla) is made by making the floor of sufficient thickness to resist the sub-soil pressure. Although the depth of water over the upstream portion of

the floor is more than the uplift pressure, but to enable it to resist action of flowing water and also from other practical considerations, its thickness is kept over 60 cm. A long length of the upstream apron lengthens the path of the sub-soil water creep and lessens the uplift pressure on the downstream floor. An impervious upstream apron also keeps the likely scour of the upstream river bed away from the weir wall. In continuation of the upstream apron, a length of loose stone talus is provided on the upstream side to keep the erosion and scour further away from the pacca pavement. The stone protections settle down in the scour and keep the scour holes at a safe distance. The top level of the upstream apron is usually kept at river bed level upstream to suit the silt extractor.

A downstream pucca apron resists the uplift pressure due to subsoil water and also takes the dynamic action of the falling water. A thickness of at least 120 cm is provided at its downstream end with an increase at the upstream end. The downstream apon is provided at or below the channel bed level. A length of loose stone talus is provided at the end of the pacca apron which acts as a filter for the sub-soil water. The length of the apron should be sufficient to safeguard against the undermining of the foundation soil. Length of apron is worked out according to the creep theories.

Cut-off walls increase the creep of the percolating sub-surface flow and decrease the likelihood of undermining the foundation strata. Vertical cut-offs are provided at the ends of pucca aprons on both upstream and downstream sides and are taken down below the maximum depths of likely scour. Depth is also governed by safe exit gradient or exit velocity, according to the theories mentioned above.



Also see Page 37.56

Improvised Dam

Channel capacities in litres/second for various channel slopes and sections

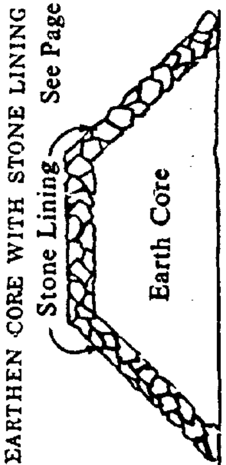
Channels are trapezoidal with side slopes 1½:1 (horizontal:vertical) and roughness coefficient 0.035

Bed width B *and water depth* H *in metres*

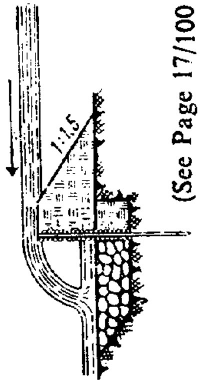
Channel slope		B: 0.10 H: 0.15	B: 0.15 H: 0.25	B: 0.20 H: 0.30	B: 0.25 H: 0.40	B: 0.30 H: 0.50	B: 0.40 H: 0.65	B: 0.50 H: 0.85	B: 0.60 H: 1.00
1/10,000	0.01%	2.5	9.3	16	33	59	121	243	378
1/5,000	0.02%	3.6	13	23	47	84	172	345	518
1/2,000	0.05%	5.6	21	36	74	133	271	545	845
1/1,000	0.1%	7.8	30	51	107	190	390	770	1200
1/500	0.2%	11	42	71	148	266	540	1090	1690
1/333	0.3%	13.5	51	87	183	330	670	1230	2065
1/250	0.4%	15.5	59	100	212	380	765	1540	2385*
1/200	0.5%	17	66	113	237	422	855	1720*	2670*
1/100	1.0%	24	94	160	330	595*	1210*	2430*	3775*

*Velocities greater than 1 m/sec. (See Page 17/36 Channels with weeds and bushes $n = 0.025$-0.035

EARTHEN CORE WITH STONE LINING See Page 17/56

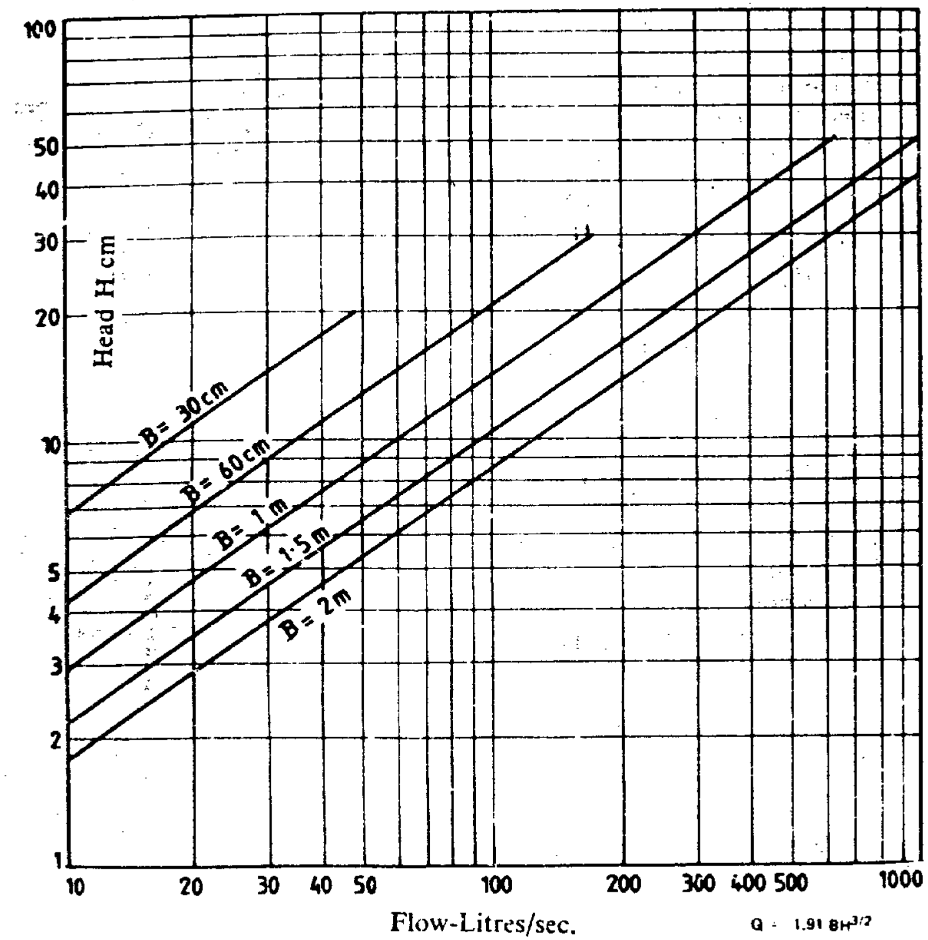


Weir Floor or Apron

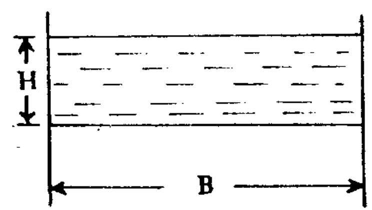


(See Page 17/100)

MEASUREMENT OF FLOW OVER HORIZONTAL WEIRS



$Q = 1.91\,BH^{3/2}$



Also see page 14/13, 17/34

Energy which Nature has provided, such as the wind, should be taken advantage of whenever possible. In many Northern European and Western Hemisphere countries, wind energy is used for pumping water for farms, homes, and small communities. This method is excellent for obtaining a steady flow of water from a well at a very low cost.

For proper operation, the following conditions must be met :

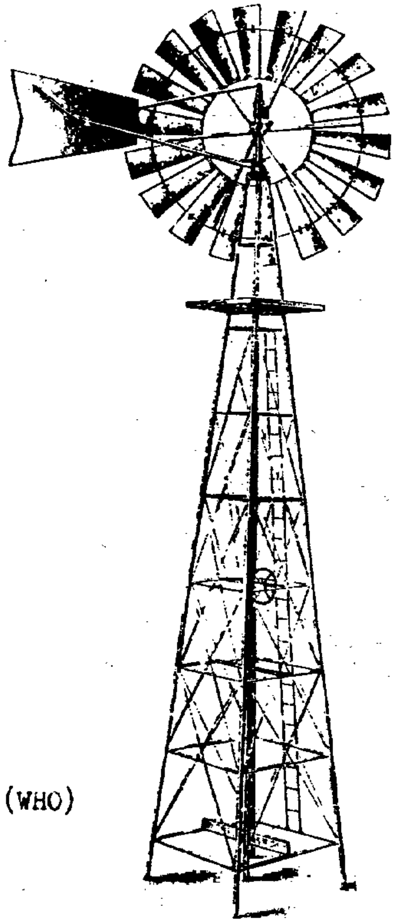
(1) winds of more than 8 km per hour during at least 60% of the time;

(2) available windmill equipment;

(3) wells that can be pumped for many hours' duration each day;

(4) storage capacity of three days' supply (or more) to take advantage of long pumping periods and to provide for calm periods when there is no wind;

(5) clear sweep of wind to the windmill. This can be obtained by the use of a tower to raise the windmill 4.5-6 m (15-20 ft) or more above the surrounding obstacles.



(WHO)

The pump used in connexion with windmills is usually of the reciprocating type and has an extension of the piston rod above the upper guide with a hole for connexion with the pump rod from the windmill

Provision may also be made for pumping by hand or for using animal power when extremely long periods of calm are likely to occur. The gearing on windmills varies with manufacturers and wind conditions : some pump one stroke with each revolution of the mill, while others pump one stroke with three or four revolutions. The former requires more and higher-velocity winds; the latter is advantageous in areas where there are winds of low velocity much of the time.

SECTION 18

ROADS AND HIGHWAYS

1. GLOSSARY OF HIGHWAY ENGINEERING TERMS

(Based generally on the Indian Roads Congress and British Standards Institution.)

Abrasion—The removal of material from the surface of a solid by grinding or rubbing action.

Aggregate—For concrete works the work aggregate suggests collection into a mass, and is used for any hard material (stone or brick) for mixing into small fragments with cement or mortar and form concrete. For (bitumenous) pavement it is meant to include angular pieces of hard crushed stone. The word coarse aggregate is usually employed for material coarser than 6 mm and fine aggregate down up to sand.

Arterial Road—See under *Trunk Road.*

Asphalt—A mixture of bitumen and mineral matter which may occur in natural deposits, or be produced by artificial means. In the first class we have the so-called Natural or Rock Asphalts and in the second the Residual or Petroleum Asphalts. (Bitumen is the binding material in asphalt.)

Asphalt Emulsion—A combination of asphalt with a small amount of soap-forming compound and water.

Asphaltic Cement—Asphaltic bitumen or the product resulting from a mixture of asphalt and flux oils or asphaltic bitumen and flux oils producing a binder having cementing qualities suitable for the manufacture of asphalt pavements. It is refined asphalt.

Asphaltic Concrete—A pre-mix of bitumen (with or without filler), sand, and not less than 30 per cent by weight of mineral aggregate of a size larger than sand.

"Shelmacadam" is a cold premixed macadam and "Shelcrete" is cold asphaltic concrete. These terms are used by Burmah-Shell Oil Co., for their produts.

Asphaltic Macadam—A mixture of bitumen (with or without filler) and a mineral aggregate of a size larger than sand. It can be made by the grouting or pre-mixed methods.

Attrition—Mutual rubbing or grinding within the mass of mineral fragments under the action of traffic thereby producing an alteration in their shapes and sizes.

Ballast—(*i*) Small stones or gravel with grit, sand and clayey materials, of which the major proportion of the particles are retained on a standard sieve having 4 square meshes to the linear 25 mm and which when consolidated yields a coherent layer.

(*ii*) Stone or gravel of irregular unscreened sizes which may contain smaller material and also sand.

Bajri—Is a term largely used to denote stone screenings ranging from fine stuff to about 25 mm gauge. This generally refers to a stone of soft variety for dressing of paths and side walks and as binding for the consolidation of water-bound macadam roads.

Bank—(*i*) An earth slope formed or trimmed to shape. (*ii*) A ridge of earth, stones, etc. naturally existing or specially constructed to guide the flow or prevent overflow in floods.

Banking—See under *Super-Elevation*.

Base Coat—An intermediate course between the base course and the wearing coat.

Base Course—That part of the construction resting upon the sub-grade, and through which the load is transmitted to the sub-grade or the supporting soil. A base course is the layer immediately under the wearing surface.

Benching, Stepping—The formation of a series of small level platforms or steps upon an incline or slope.

Berm—See under *Shoulder*.

Binder—Is a term applied to tar or bitumen used for binding road metal.

Bitumen Macadam—Consists of bitumen only.

Bituminous Macadam—Bitumen or tar macadam.

Bituminous Cement—A general term for bitumenous materials which bind together adjacent solid substances of a suitable nature.

Bitumen—Is by-product of the distillation or evaporation of crude petroleum either by natural process or in a refinery and is the basic constituent of asphalt. It is characteristically solid or semi-solid, black or brown in colour, is sticky and melts or softens on the application of heat. Bitumen used for roads is usually a highly refined product, containing from 90 to 99 per cent of bitumen soluble in Carbon Disulphide. (This is Asphaltic Bitumen and is referred to as Bitumen). Bitumen is marketed in various grades suitable for various purposes and the different grades vary in "penetration". Natural bitumen is found in a lake in the Island of Trinidad.

Black-top Surface—A general term applied to wearing coats or surfaces of roads in which tar or bitumen is used as a binder.

Bleeding—The exudation of bituminous material on a road surface after construction.

Blinding, Gritting or Dressing—Spreading of stone chips, sand or other fine material on a road surface after application of bituminous material, or to fill the voids or interstices in a water-bound macadam surface. *Blindage*—the fine material so used.

Blind Alley—A way or road open at one end only.

Blotter—A covering of a suitable material to absorb excess binder or to overcome bleeding.

Blown Bitumen—Also known as *oxidized* bitumen. Is produced by blowing air through molten, steam refined asphaltic bitumen. This process produces a bitumen with comparatively high melting point and lower ductility. Blown bitumen has better weathering properties than steam refined type.

By-pass Road—A road to enable through traffic to avoid congested areas or other obstructions to movement.

Camber ; Transverse Slope—The convexity given to the curved cross-section of a carriageway, between the crown and the edge of the carriageway ; it is the difference in level between the crown and the edge of the carriageway. Sometimes called *crossfall*.

Carpet—A wearing surfacing obtained by laying bitumen or tar concrete in two or more coats in a thickmess of more than 25 mm.

Carriageway—That portion of the roadway designed and constructed for vehicular traffic.

Cant—See under *Super-Elevation*.

Cattle Creep—See under *Stock Subway*.

Catch Drain—Is a drain provided in the slope of a cutting to intercept the water flowing down the cut slope.

Chipping—The term is generally intended to include uncrushed gravel as well as crushed rock, of a gauge finer than 20 mm (25 mm to 3 mm according to B.S.S.)

Chips—Small angular fragments of stone containing no dust.

Coal Tar—Is a bye-product in the manufacture of gas from coal. It is viscous or liquid, resulting initially from the destructive distillation of coal which has been so refined as to be suitable for road work.

Coal tar has some volatile oils which evaporate by exposure, leaving the tar brittle and friable. That is why overheating of tar is prohibited.

Corrugations—Ripples, waves or undulations which are liable to appear in all types of road surfaces.

Crazing—The breaking up of a surface layer through cracking into some irregular shaped areas.

Creep—The slow plastic movement of the material in a surface layer in the line and direction of traffic flow or gradient.

Creteways—A carriageway in which a cement concrete wearing surface is provided for the wheel tracks only.

Crossfall—The fall given to the surface of any part of a roadway, at right angles to road length.

Crown—The highest point (in cross-section) of a curved road surface, commonly at or near the centre. The level of crown is called road surface level.

Crude oil—Unrefined petroleum.

Cut—The material excavated to make a cutting.

Cut-back—A solution of bitumen in a volatile or partly volatile solvent such as kerosene or creosote. The addition of the solvent lowers the viscosity of the bitumen, (makes it more freely flowing) thus it can coat cold chippings more easily. When a cut-back is applied on water-bound surface, (the kerosene evaporates in a few hours) it soaks in and hardens to bitumen. It is also called "fluxed" bitumen. Cut-backs contain about 80 per cent bitumen and 20 per cent solvent.

Unlike emulsions, cut-backs have to be used on dry surface and with dry aggregate. Cut-backs can be either applied cold or brought to working consistency by moderately heating to about 137° C. before use to ensure sufficient fluidity and adhesion to stone, as opposed to heating to about 175° C. for bitumens.

Cutting—That portion of the site of a road where the formation has been excavated below the ground level.

Detour—An alternate circuitous route for traffic going around a closed portion of a road : a temporary route.

Dragging—The operation of smoothing out and reshaping irregularities in surface earthwork by means of a drag. (See further, under "Plant and Machinery Terms").

Dressing—See under *Blindage.*

Drive-way—A way to secure access from a road to private property.

Edging—Bricks (or blocks of concrete or stone) embedded along the edges of a pavement to protect the pavement from damage caused by traffic.

Embankment—An earthwork raised above the natural ground by the deposition of material to support construction at a higher level.

Emulsion—A freely flowing liquid at ordinary temperature in which a substantial amount of bitumen or tar is suspended in a solution of water in a finally divided and stable state. Emulsions contain about 50 to 65 per cent of bitumen. Can be used in all climates and are very useful for patch repairs on bitumenous surfaces. They are used cold and can work with wet chippings. When emulsion is spread on the road it "breaks" and changes from brown to black colour and the water soaks in or evaporates allowing the bitumen particles to reunite and lie on the surface. Emulsions are more easily applied than hot binders. This performance, however, is affected to a much greater degree by adverse weather. Because of relatively thin film of binder that remains on the road, smaller chippings (not more than 6 mm) must be used with emulsions than with hot binders.

Before the application of emulsion the road surface should be thoroughly cleaned and slightly damped with water, and chippings spread and rolled before the emulsion has "broken".

Fair-weather Road—A road that can be used by traffic during dry weather only and not during monsoons.

Fascines—Bundles of grass tied and laid across a sandy track for passing temporary traffic.

Fat—Containing an excess of bituminous material.

Fender Kerb or Wheel Guard—Kerb so placed as to prevent the encroachment of, or to secure the constraint of wheeled traffic.

Filler—Any fine mineral powder added to bituminous mixture in the course of manufacture, and which has been ground to such a degree of fineness that not less than 85 per cent by weight passes a 75 micron sieve. The common fillers are—limestone dust, cement, granite dust, slate dust, slag dust, coal dust, china clay and fuller's earth.

Lime seems to possess excellent qualities as a filler and is used most. Fine sand passed through the 75 micron sieve should also be taken into account along with the quantity of filler as it also helps to some extent.

The functions of a filler are :—(*a*) to increase the viscosity of the binder and hence increase density and stability of the mixture, (*b*) to enable a thicker film of binder to be held by the mixture, (*c*) to improve the resistance of the binder to weathering, (*d*) to increase the effective volume of the binder, and (*e*) to reduce the apparent temperature susceptibility of the mixture (for dense surfacings-filler/binder mixtures have lower temperature susceptibilities than straight binders of the same viscosity). It tends to reduce the brittleness of a mix. in cold weather and the quantity of the filler can be considerably increased. After compaction the surface should show a close texture.

Flagstone—A flat and relatively thin slab of natural or artificial stone for pavements subjected only to foot traffic.

Flash point—The lowest temperature at which the vapour of a substance momentarily takes fire but does not continue to burn, under specified conditions of test.

Fluxing—Is softening hard bitumens or asphalts which are too hard for use, to the desired consistency by incorporation of certain oils. (The product is called Flux oil).

Fluxing Agent—A substantially non-volatile material (Flux oil) used for reducing the consistency of a bitumen (softening bitumen).

Fly-over—A junction so designed that traffic streams are divided to enable them to pass over or under each other.

Formation Width—Is the finished top width of earthwork in fill or cut for receiving the road structure. It is the "roadway" as already defined.

Formation—The surface of the ground in its final shape and level after completion of earthwork.

Foundation—Denotes that portion of a road structure lying on the formation level.

Fretting—The loosening of a wearing surface under the action of traffic or weather, associated with the failure of the binding agent to keep the surface consolidated.

Grading, Trimming—The operation of excavating and shaping the surface of earthworks. The final shaping of earthworks.

Gravel—Rounded or water worn stones of irregular shape and size occurring in natural deposits with or without finer material. Gravel is usually harder and more rounded than bajri and may contain certain amount of earth or clay mixed with it.

Grit—Fine small sized sharp-edged stone aggregate or coarse sand used for blinding road surfaces which have received a bituminous dressing. It gives a suggestion of roughness in the stone and of roughness to the work.

Gritting—The operation of spreading small broken stones, chippings, or gravel.

Grouted macadam—A consolidated wearing surface formed by the application of a binder (Bitumen or Tar) in liquid state into the interstices of the mineral aggregate after the latter has been spread on the foundation. Consolidation may take place before or after the application of the binder.

A macadam crust in which the stone aggregate is bound together by a binder applied to penetrate to the desired depth.

Grouting—The action by which a binder in liquid form (cement, tar, bitumen, etc.), is made to penetrate into joints, fissures or cracks in concrete work or between blocks, (or road aggregate) under the action of gravity or by applied pressure.

Grubbing—Uprooting and removing the stumps and roots of small trees, plants, hedges, etc. from the sito of the works.

Gutter—An open drain constructed along the sides of a carriageway (in town areas) to carry away the water drained from the surface.

Hardcore—A consolidated layer of broken stone, brick, slag or concrete in sizes of about half a brick, with some proportion of smaller material.

Haul—See under *Lead.*

Haunch—See under *Shoulder.*

Highway—An important road in a road system.

Hoggin—Fine sand, earthy gravel, moorum, or other fine material, that forms the slurry grout in water-bound macadam surfaces.

Ignition point—(Burning point)—The temperature at which the vapour of a substance takes fire and continues to burn, under specified conditions of test.

Island—A central or subsidiary area in a carriageway at road junctions, shaped and placed so as to constrain and control traffic movement.

Lake Asphalt—An asphalt which as found in nature is in a condition of flow or fluidity.

Lay-by—The local widening of a carriageway to enable vehicles to draw off the road for temporary parking or stoppage without obstruction to traffic flow.

Lead, Haul, Run—The distance over which excavated material is transported (or carried) for use as filling or to the a bank.

Lean—The opposite to "Fat". Containing a deficiency of bituminous material. or conversely, containing an excess of aggregate.

Levelling Course—A course placed for the purpose of shaping old surface to proper cross section to receive a subsequent surface course.

Liquid Seal—Is a term used to indicate that the material used for dressing is in a liquid form and does not require to be heated.

Lockspit (Dagbel)—A narrow continuous V shaped cut made in the ground surface along a defined line of demarcation.

Loop Road—A route formed by a road or a series of roads to avoid an obstruction or provide an alternative way for traffic.

Macadam—Broken stone, road stone or road metal : Crushed or broken stone of regular size below 75 mm for road construction.

Mastic Asphalt—Asphalt or bitumen heated and mixed with fine mineral fillers (lime-stone powder, sand or chipping, etc.) to form a coherent voidless impermeable mass, solid or semi-solid under normal temperature and of such consistency that it can be spread when hot by hand 25 mm to 50 mm thick with wooden floats and sets on cooling to give a firm impervious surface. The bitumen has 8 to 10 per cent of sand.

The mastic is laid at a temperature of 160° to 175° C. on a prepared surface. Chippings are spread over the laid asphalt where the thickness is over 12 mm or under heavy traffic to reinforce the mastic, and compacted.

Mastic asphalt is said to be much more durable than the ordinary asphalt and easily excavated unlike concrete. It is good for road surfacing during monsoons. It is more durable at road junctions and traffic rotaries where there is higher acceleration, retardation and load bearing ; around collars of manholes which wear away very fast ; and an extra coat on pedestrian footbridges.

Matrix—The binding material in which the larger particles of mineral aggregate are embedded.

Median Sirip—A dividing strip in the middle of a roadway.

National Highway ; Provincial Highway ; District Road—Roads classified as such by an Authority. National highways are the most important roads connecting capital cities of different states (or provinces). Provincial highways are the main roads within a state connecting important towns of the state. District roads are of lesser importance than provincial highways.

Pavement—Is the hard crust placed on the soil formation after the completion of the earthework.

Paving—Separate blocks or units (usually stone, cement concrete or wood blocks) fitted closely together over a road to serve as a surface.

Picking—The loosening of the top surface of a road by pick axes or similar tools.

Pitch or Coal Tar Pitch is the black or dark brown solid or semi-solid residue from partial evaporation or distillation of tars.

Pot-holes—Marked local depressions in a surface layer, roughly circular in plan, arising from the wearing away of material by traffic or by some other agent.

Prime Coat—The initial application of a binder to an absorbent highway surface prior to the construction of a wearing coat.

Primer—A binder of low viscosity which on application to a surface, other than a black-top surface, is complelely absorbed. Its purpose is to water-proof the existing surface and prepare it to serve as a base for the construction of a black-top surface.

A primer may be a road oil, a cut-back asphalt or a low viscosity road tar. Some volatile oil is mixed with bitumen to make it less viscous and more highly penetrative binder. A coat of primer is given over dusty, porous or soft roads (such as moorum, kankar, soft sandstone, laterite, limestone, brick aggregate) before applying bitumen, as it will not bind to a dusty surface. The function of a primer is to penetrate into the road and to coat the blindage thoroughly up to a depth of 25 to 40 mm.

Radial Road—A road which provides direct communication between the centre of an urban area and the outer districts.

Ramp—A short steeply inclined way connecting surfaces at different levels, Generally made for repair platforms. Maximum rise 1 in 6, prefer 1 in 7.

Refuge—A raised pavement or platform, or a guarded area, so sited in a carriageway as to divide the streams of traffic and to provide a safety area for pedestrians. (Usually provided at the entrance of radial roads to rotary carriageway).

Right-of-way—(*i*) The land secured and reserved for development of a road and all structures pertaining to the road. (*ii*) The privilege of use of a way, acquired by the traffic by law, custom or usage.

Ring Road—A circumferential road built around an urban area to enable free flow of traffic.

Road—A way for vehicles and for other types of traffic over which they may lawfully pass. It includes the entire area comprising the roadway and all structures pertaining to the road within the limits of the defined boundary or "right-of-way."

Road oil—A term applied to various type of liquids or cut-back asphalts, heavy oils, etc., which are applied to road surface to lay dust, or for surface treatments.

Roadway—That portion of a road (included within the construction limits) ordinarily used for traffic. It includes carriageway and shoulders.

Rock Asphalt—A natural rock formation, usually of lime-stone or sand-stone, impregnated with bitumen throughout its mass.

Rolled Asphalt—A dense mixture of stone, sand, filler and bitumen mixed and laid hot, and consolidated by rolling while still warm.

Run—See under *Lead*.

Rubble—Pieces of stone or broken brick or concrete of irregular size and shape.

Rut—A groove or depression formed in a surface layer longitudinal to the road by the wheels of travelling vehicles.

Sag—The hollow or depression formed by the junction of two falling gradients.

Sand Paper Surface—A rough surface texture for road surfacing produced by the pressure of protuberant sharp particles of mineral aggregate which are not larger than about 6 mm size.

Scarifying—The loosening of the top surface of a road by mechanical or other means.

Screenings—The small size stone particles sieved through the lowest mesh of 6 mm prescribed for chipping sizes.

Seal Coal or Sealing Coat—A dressing of tar or bitumen blinded with grit, etc., applied to open textured bituminous surfaces to render the surface watertight and strengthen the macadam. This may be with pre-coated chippings and applied as surface dressing. Thickness is about 12 mm and the size of grit used varies from 10 mm down to sand. A seal coat is more or less like a renewal coat of surface dressing.

It should be the aim of the engineer to avoid the necessity of a seal coat in order to reduce the cost. Grading of aggregate and addition of fine material to the mixture achieves this object.

Service Road—(*i*) A subsidiary road constructed between a road and buildings or properties facing thereon and connected only at selected points with the principal road. (*ii*) A way at the back of buildings for "servicing" and providing other means of access.

Sheet Asphalt—A pre-mix of bitumen (with or without filler) and sand, and containing coarse aggregate not exceeding 30 per cent. This is really a dense carpet where stone metal is discarded and chippings limited to 30 per cent, the rest being sand. Sheet asphalt is laid in thicknesses varying from about 20 to 40 mm.

Shoot—See under *Spoil Bank*.

Shingle—Consists of coarse rounded or water-worn stone, detritus or pebbles larger than gravel and smaller than boulders and is available in hill streams.

Shoulder, Haunch or Berm—(*i*) The portion immediately beyond the edges of a carriageway (usually of earth unmetalled) on which vehicular traffic may pass occasionally (while crossing). (*ii*) The strip of land between side drain and the lower edge of bank.

Side Ditch—Is a roadside drain or channel provided at the toe of a road bank.

Sod—A rectangular piece of turf.

Spoil—Surplus excavated material.

Spoil Bank, Tip, Shoot—An earthwork bank formed by depositing spoil (outside the limits of the works).

Stock Subway or Cattle Creep—A shallow subway constructed to permit passage of cattle underneath a road or a railway.

Straight-run Bitumen—Is steam refined bitumen. Bitumen made by the straight distillation of suitable crude oils ; steam is injected into the oil so that the distillation is carried out at a lower temperature.

Stepping—See under *Benching*.

Street—A road in built-up area.

Stripping—The preliminary operation of clearing the site of the works of turf, grass, weeds, brushwood and other extraneous material.

Sub-crust—An intermediate layer acting as a cushion between the pavement.

Subway—An underground passage or tunnel to permit the movement of traffic, or to accommodate service pipes, cables, sewers etc.

Summit—The peak formed by the junction of two rising gradients.

Super-elevation, Banking or Cant—The inward tilt or transverse inclination given to the cross-section of a carriageway on a horizontal curve to reduce the effects of centrifugal force on a moving vehicle.

Tack Coat—The initial application of a binder to an existing surface given to ensure thorough bond between the new construction and the existing surface.

Tip—See under *Spoil Bank*.

Trackways—A carriageway in which wearing surface is provided on the wheel tracks only (usually of bricks, stone or concrete slabs).

Traffic Density—Is the number of vehicles using the road per hour during peak periods and is the average of several peak days. The daily traffic is approximately ten times the maximum hourly traffic.

Traffic Lane—Taken as unit of width of a carriageway and which is supposed to accommodate only a single line of vehicular traffic; while crossing, vehicles have to use berms.

Transition Length—The length of the transition curve connecting a straight length of road with another main curve which may be circular or transitional.

Transverse Slope—See under *Camber*

Trimming—See under *Grading*

Trunk Road, Arterial Road—A main channel or traffic route which forms an essential part of the highway system of the country.

Turf—The surface of grass land consisting of earth or mould filled with matted roots of grass and other herbs.

Water-bound macadam—The surface layer of a road in which the road metal has been consolidated with water and earthy material or rock particles.

A type of surfacing in which stone fragments are first interlocked by rolling and then bound with smaller stone, gravel, etc., which is forced into the interstices by brooming, watering and rolling.

Weaving Length—The length of carriageway between adjacent radial routes around a traffic roundabout.

Wheel Guard—See under *Fender Curve*.

Plant and Machinery Terms

Batching Plant—A mechanical equipment designed to measure the proportions of the various materials required to form a charge, *e.g.*, as in the mixing of concrete.

Bulldozer—Is a tractor on the front of which is mounted a curved strong adjustable steel blade which is employed for spreading and levelling by pushing loose excavated material. A *tree dozer* is used for felling trees and a *stumper dozer* is used for uprooting stumps.

Caterpillar track—An endless tread, generally of metal links, running over two or more wheels for the purpose of distributing the wheel loads over a greater area so as to permit of a vehicle so fitted passing over soft or uneven ground.

Drag—A machine fitted with two or more oblique blades, generally of steel, for scraping off and reshaping irregularities in the surface of earth or similar low type roads.

Dumper—A vehicle for transporting excavating material, so designed as to be capable of discharging its load by forward tipping.

Grader—A machine provided with an adjustable blade or scraper within the wheel base for shaping the road, subgrade or subsoil by loosening or moving the superficial materials laterally. It is either self-propelled or is toed by a tractor.

Hopper—A funnel-shaped storage receptacle, through which material can be measured or periodically discharged.

Jumper—A heavy bar chisel or drill worked either by hand or by means of a hammer, used in making blasting holes in rock.

Scarifier—An independent machine or attached apparatus for scoring and loosening the surface of a road to a regulated depth. The teeth of scarifier, which are known as tynes, are set with a forward slant.

Scarifier tyne—The pointed steel bar or rod acting as the cutting unit of a scarifier.

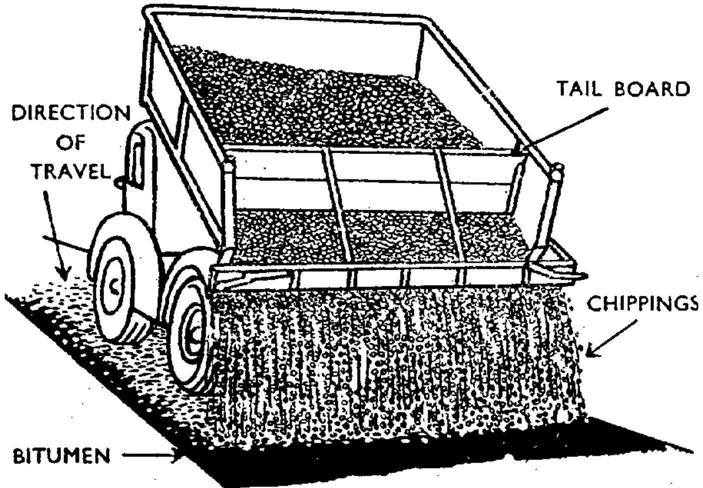
Scoop—A machine consisting essentially of a bucket or shallow container with a cutting edge, designed to excavate, load and transport over relatively short distances, and dump soft or previously loosened material.

Scraper—It consists of a large scoop with cutting edge. It excavates, transports and dumps the material where required. The cutting blade maintaines a constant digging depth.

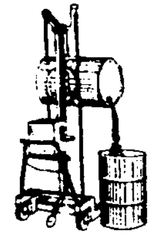
Screed—A strip of wood or metal, used as a guide for a template straight edge, for finishing a floated surface to a required profile.

Template—A full-sized mould, pattern or frame shaped to serve as a guide in forming or testing contour or shape.

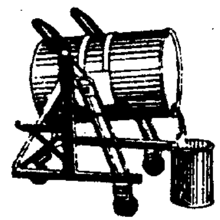
Tractor—Is a self-propelled powerful tractive machine which is used either for towing other machines or equipments are fixed to it to form a self-sufficient unit. This machine is carried either on wheels or crawler track. A crawler track is a device consisting of an endless chain of plates which bear upon the ground and this device is used in place of wheels. (See "Caterpillar track").



Aggregate Spreader

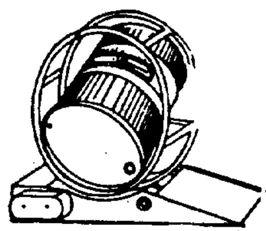


Drum Jockey

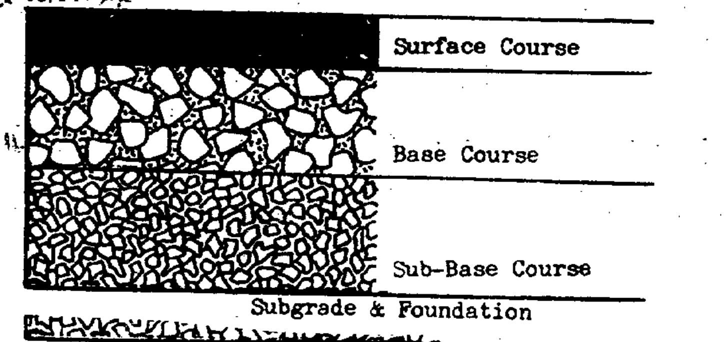


Drum Lifter

Bitumen Drum Mixer



Drum Mixer

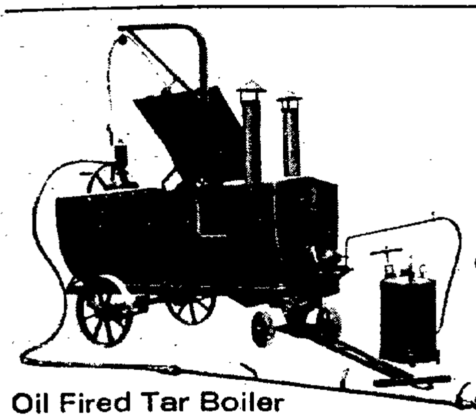


Subgrade & Foundation

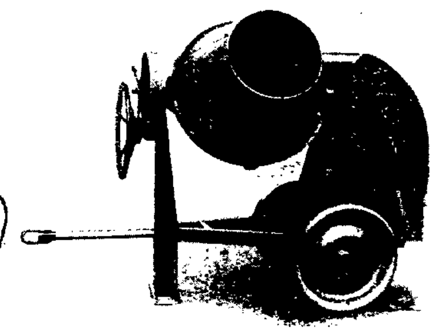
New Design of Creteways for Rural Roads

A new design of creteways for flood-prone rural roads has been developed at Central Road Research Institute, New Delhi. It consists of two tracks of cement concrete blocks, 90 cm wide, 60 cm long and 10 cm thick. The slabs in each track are interconnected with mild steel dowel bars. Spacing between the two tracks of creteway is 75 cm. These creteways, where needed, are to be laid over low-cost semi-rigid sub-bases such as lime-flyash stabilized soil and lean cement-flyash concrete depending on subgrade and moisture conditions. They can be used at locations subject to flooding but not prone to scour. They lend themselves to easy repair and replacement, and any dislocation occurring under floods can be easily rectified after the floods recede. In this process, blocks can be removed and reused where the alignment is required to be changed as a flood control measure or after finalization of cross-drainage works.

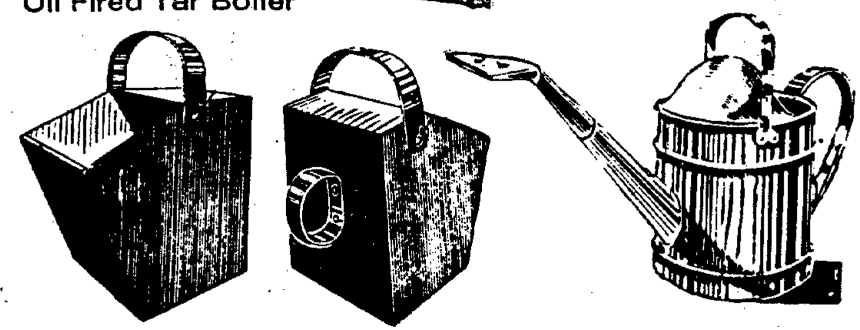
The creteways are intended for light rural traffic of bullock carts, tractors, jeeps, etc. as well as for occasional passage of buses and trucks. **(CSIR News, August 15, 1981).**



Oil Fired Tar Boiler



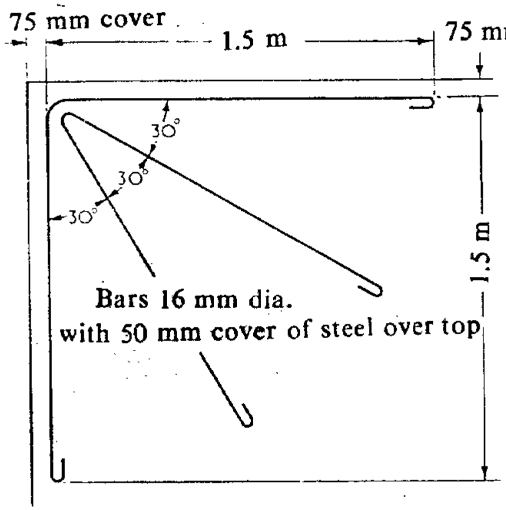
TILTING MIXER



Annexes

Footpaths or Sidewalks

Minimum width should be 1 5 m. The width should be increased by 1 m in business and shopping areas. Footpaths adjoining shopping frontages should be atleast 3.5 m and a minimum of 4.5 m adjoining longer shopping frontages



CORNER REINFORCEMENT

Where concrete slabs are less than 30 cm thick or where the slabs are on a poor foundation, it is advisable to strengthen their corners by means of "hairpin" reinforcement as shown in the illustration. Acute-angled corners should be avoided if possible as the loading stress in such corners is very high. If however, they are unavoidable, as in some slabs at road junctions, corner reinforcement must be used. (See page 18/102)

Strengthening of Edges & Corners of Cocrete Road Slabs

Cement Concrete Roads versus Asphalt Roads :

A concrete road with proper foundations costs about five times that of an Asphalt road. Concrete road lasts at least 20 years against the 5 year life-span of an asphalt road. Durability of asphalt roads depends upon the number of vehicles using the road. This factor does not affect concrete roads. Concrete per sq. metre costs nearly Rs. 500/- against Rs. 135/- required for asphalting. (Bombay rates of 1988)

See page 18/114

See page 18/16

2. ROAD STRUCTURE

The structure of a road consists of: the *formation* or *sub-grade* and the *pavement*. The structural element of the pavement is the foundation (soling or bottoming) also called *sub-base*, and the *base*. The base may be surfaced either with a concrete or a bituminous *surfacing*.

The Sub-grade or Formation. It is the soil foundation—the surface of the natural ground (in its final shape after completion of earthwork) on which the entire road structure rests. The importance of the sub-grade lies in the fact that if it fails the performance of the whole road will be affected. A sub-grade must be able to resist the effects of both traffic and weather.

Foundation, Soling or Bottoming. The function of this course is to spread the traffic loads and the weight of the roadway above (base). The soling may consist of either: *(i)* Hand packed big size stones called rubble, or *(ii)* Bricks laid flat, or on edge, or *(iii)* Over-burnt brick bats well rammed, called hardcore.

The Base. The base course is the major structural component of a road and is composed of stone aggregate or road metal well consolidated. The main considerations of a base are: Its thickness, stability under traffic loads and resistance to weathering. The stability of the base depends on its thickness and the thickness of the base construction is mainly a function of the strength of the sub-grade and the maximum wheel loads that are anticipated with the intensity of traffic.

Sometimes foundation (soling) and base are combined and the whole is called the base.

The soling (or the base) is most commonly placed directly on the sub-grade but where the sub-grade is of poor quality or has poor drainage properties, a layer of granular material consisting of rubble, brickbats clinker, ashes, etc., is interposed between the soling and the sub-grade. This layer is known as *sub-base*. A sub-base is usually placed under a concrete road.

The Surfacing. Is the uppermost part of the road structure. Its purpose is to minimize the abrasion of the road by traffic, act as a cushion between the wheel and the base, and reduce the adverse effects of climate. By acting as an impervious layer it enables the road to shed storm water that would otherwise damage the road. It is the *wearing surface.*

3. CAUSES OF DISINTEGRATION OF ROADS IN INDIA

Useful Life of Roads

The useful life of a road depends upon — The type of construction (or road structure), intensity and nature of traffic, bearing capacity of the sub-grade (soil), climatic conditions, and maintenance. A properly constructed water-bound macadam road will carry some 250 to 350 medium-weight vehicles per traffic lane per day without excessive maintenance. A bitumen-grouted macadam, provided that is laid on a sound sub-grade, will carry up to 1000 or more medium weight vehicles per traffic lane per day. A water-bound macadam road is not considered suitable for traffic above 1000 tonnes (combined).

The following figures are usually taken for estimating purposes: (Density of traffic considered is for iron tyred and pneumatic tyred traffic combined.)

Water-bound macadam roads:

4 years for traffic up to 250 tonnes per day (150 vehicles).
3 years for traffic up to 500 tonnes per day.
1 to 2 years for traffic up to 1000 tonnes per day.

Bituminous treated roads:

12 to 15 years for traffic up to 1200 tonnes per day. These roads will require renewals (re-metalling) at such intervals if routine maintenance of surface dressing, etc., is carried out regularly at every 2 to 3 years.

The life of a cement concrete road is taken at about 30 years.

Water-bound Macadam Roads

(a) Heavy mixed traffic and adverse weather conditions damage the water-bound macadam roads in no time. Steel tyred bullock carts crush the road metal and the fast moving pneumatic tyred vehicles create a partial vacuum and loosen the metal. The pulverized metal is blown off by the winds.

(b) The rain water wash away the soil binder and the stone aggregates protude out or get loose on the surface layer thus forming pot-holes and ruts.

(c) The severe heat of the sun in day time and cool temperature at night.

Corrugations on Water-bound Macadam—Corrugations are wave-like deformations on the road.

(d) Wheel spun throwing up loose surface blindage.

(e) Defective rolling of thick coats of metal which produce the effect of the metal creaping in front of the roller, and then forming a hard ridge over which the roller rides to commence another depression. To prevent the formation of initial waves during consolidation—the roller-gearing should be in good condition so as to prevent jerky rolling, and long runs should be taken to minimize stopping and reversing the roller. Longitudinal evenness should be checked with a 15-metre string.

(f) Imperfect grading of coarse aggregate may cause uneven displacement of stones resulting into corrugations under the load of traffic.

It has been observed that corrugations form more on banked portions of the road than in cuttings, suggesting that the sub-grade may be a contributing cause. It has also been observed that if traffic is not allowed on a newly rolled water-bound macadam road for some days, the formation of corrugations is delayed. This suggests that it is inadvisable to allow fast moving mechanically propelled traffic on road crusts which are still plastic due to a large amount of moisture. Where there is a good soling below the road crust, corrugations are at a minimum.

Poor sub-grade and lack of proper drainage may be the causes of corrugations. This has been discussed in detail later.

The application of a surface treatment of tar or bitumen prevents the sucking action of rubber tyres and gives the road a water-proof surface which is supposed to be impervious to weather conditions.

Surface Treated Roads

Due to Defective Sub-grades and Foundations

Even with light traffic there are many failures with surface painting. They are due not so much to the wrong use of surface treatment but to the fact that very little attention is paid to the proper preparation of the sub-grade and the foundations. In most of the cases the base of the road is the weak point and not the surface. The chief enemy of surface treatment, however, is the dampness in the foundations. This is caused either by condensation or by water working in from the sides of the road by capillary action. Further, if the sub-grade is weak, the surface treatment does not prevent movement in the sub-grade due to the pounding action of sub-grades.

It must be realized that it is the sub-grade which really carries the weight of the vehicles and that the function of the road surface (road structure) is merely to distribute that weight over a larger area. If the sub-grade is composed of unsuitable material, or if water has access to it, the earth underneath will move and the road surface will fail. The character of the sub-grade has a great effect upon the stability of a road surface, no matter what material may be used for the latter. Therefore, a study of the soil structure is a very important factor. (See under "Sub-grade and its Preparation" in the following pages.)

General Defects on Asphalt Roads

An asphalt road crust has no strength of its own to withstand pressure and impact of traffic, but acts as a resilient soft and smooth cushion between the foundation of the road and the traffic. Due to heat asphalt gets semiplastic and loses its property of resilience. Constant softness tends to produce waves and corrugations in the surface; ruts are formed under iron-tyred heavy traffic and the asphalt gets stuck to the wheels of the vehicles and the road is damaged to a considerable extent. Care should be taken to use the correct type of bitumen (as regards its penetration) for the particular temperature, and when such defects are noted on a road, precautions should be taken by diverting the "track lines," and the softened surface dusted with sand or fine grit and rammed, if necessary. (See also under "Bleeding".)

The following are some faults and failures which occur in treated surfaces

Bleeding or Flushing is the exudation as a liquid of some of the binding material (bitumen or tar) from the surface layer. Generally accurs on surfaces that have been given a seal coat. Bleeding is generally caused by (a) use of an excess of binder which may be due to the bitumen not being hot enough at the time of application and not properly spread out,

(b) insufficient quantity of blinding material, (c) loss of cover aggregate due to lack of adhesion with the binder.

To stop bleeding more blindage (fine aggregate or coarse sand) should be applied and continued till the bleeding stops.

Fatting up is same as bleeding due to excess binder.

Streaking or Striations— is usually caused by non-uniform application of the binder due to (a) mechanical faults in the equipment, (b) improper adjustment or careless operation of the binder distributor, (c) inexperience of the sprayerman.

Scabbing or dislodgment of cover aggregate due to (a) insufficient binder to hold the aggregate firmly in place, (b) the use of too hard a grade of the binder, (c) lack of adhesion of the aggregate with the binder due to dust or moisture on the aggregate, (d) whip-off by fast traffic, allowed before development of adhesion, (e) rain immediately after or during construction, if traffic has been allowed on the road before the surface has dried, (f) undue delay between the spraying of the binder, spreading of the aggregate, and rolling.

Poor bond or poor adhesion to the road surface will subsequently lead to the formation of pot-holes. Poor bond is usually due to (a) pockets of dust or other fine material, (b) presence of moisture, (c) low pavement temperature prevailing during spraying, (d) use of a grade of binder too hard for the prevailing conditions.

Corrugations—Due to instability of base or poor original riding surface.

Cracking—Due to foundations or sub-soil movement.

General Fretting or Crazing—Due to dressing being worn out.

Loss of Stone—Due to rain within a few hours of laying, or due to lack of binder.

Waving—May be due to an excess of binder which acts as a lubricant; or due to the sub-grade being smooth and allowing the road crust to slide on it. It may also be due to unsuitable metal which does not interlock. Excess of bitumen or tar and too little grit result in a soft pliable coating which will be pushed by the action or traffic.

The cause can be found by digging up the metal at intervals and examining it. To cure waving the whole surface has to be scarified, say to a depth of 40 mm. Then roll, spread metal and apply a seal coat. Or alternatively, a premixed carpet of about 40 mm thickness can be laid and rolled smooth. Grooves may be made in the road surface, as described later, so as to form a key for the premixed carpet.

Ridges—Usually appear on surface dressed portions done by spraying where the operator has not been sufficiently careful to see that each sweep of the sprayer does not overlap. The valleys should be resurfaced and brought up to the level of the ridges.

Failure of materials which may result from lack of control

Wrong type of tar or bitumen : The binder may be too hard or too soft for the surfacing required. These are checked by a tar distillation test and viscosity or penetration test.

Incorrect quantity of binder : Too little binder may result in a surfacing being brittle and having a tendency to fretting or crumbling. Too much may cause smoothing and softness.

Incorrect proportioning of aggregate and filler, resulting in too open or too dense a mix. This is shown by the sieving test.

Cracks in treated pavements may close under the kneading action of heavy traffic if it is a graded mix, and a mix that does not close up under compaction shows a deficiency in filler. A suitably graded sand with correct percentage of filler and binder will produce a close and dense surface after compaction. Cracks should however, be filled in with asphalt grout and coarse grit if they do not close under traffic.

Overheating : This can cause loss of the more volatile oils in the binder and may result in brittleness and lack of cohesion of mixed materials.

4. SUB-SOIL DRAINAGE AND MOISTURE CONTROL

The removal and diversion of water from road surface and adjoining land is called *surface drainage* and removal from the sub-grade is called *sub-soil drainage* or *sub-surface drainage*. *Road drainage* is the removal of water from the road surface as well as from the sub-grade.

It has been appreciated since roads were first constructed that their stability can only be maintained if the soil foundation remains in a relatively dry condition. As the moisture content increases the strength of sub-soil decreases. This effect of moisture content is more marked in the case of clay soils than with granular soils such as sands and gravels.

The control of the moisture content of the sub-grade is an essential feature of a good road. Water moving freely under the action of gravity can affect the moisture content of a sub-grade by reaching it through a pervious road surface, by seepage and by a rise in level of the water-table. Moisture can also enter or leave the sub-grade, either as a liquid or as vapour, under the action of forces entirely inherent in the soil itself. Moisture content also increases when the above evaporation (movement of moisture) is stopped due to putting on top of an impervious layer of road surfacing. About one metre below the road surface moisture content stays about the same.

The main object, therefore, is to maintain the sub-soil under and adjacent to the carriage-way in as uniform a condition as possible by preventing moisture entering the road bed rather than by drawing all water from it. It is not necessary to drain granular soils such as sand and gravel because they are not appreciably affected by changes of moisture content. Water cannot easily be drained from heavy clays because it is held in suspension by capillary forces as in blotting paper.

In the majority of cases saturation or softening of the road bed can be prevented by providing longitudinal ditches of sufficient depth paralled to the road on both sides. The cross-section of a side ditch is usually V-shaped or trapezoidal. These ditches may be filled with stones, brick ballast or gravel, etc., of size 20 mm to 40mm (See also in the following pages under "Roadside Drainage.")

A clay sub-soil is subject to volume changes arising from changes in the moisture content and such a soil is apt to become water-logged during monsoons or in the winter, and dry out and crack in dry months. This can be reduced by the incorporation of granular material such as gravel, sand or clinker, either by harrowing or by rolling the material into the clay or, by consolidation to a thickness of about 75 mm. Care should be taken to prevent direct contact between the clay sub-soil and the road foundation proper, and for this purpose a layer of granular material, water-proof paper, clinker concrete or ballast concrete of lean mix have been used successfully to meet varying conditions. It is also important that the ingress of water to clay subsoil should be prevented; the road surface should be well sealed and, particularly in cuttings, intercepting drains should be laid along the edges of the road.

The sub-grade also requires protection against loss of moisture. A decrease in the moisture content of sub-grade soils and base materials may occur as a result of the transpiration of moisture by vegetation growing close to the road. This loss in moisture is accentuated during prolonged drought and particularly so with clay soils. The effect can be minimized by not permitting the growth of large trees or shrubs (if such a condition is otherwise suitable) within about 5 metres of the edge of the carriage-way, especially in the case of concrete roads.

Before designing the sub-soil drainage system for a road, a survey should be made of the soil and water conditions in the sub-soil. Free water can be intercepted or controlled and a high water-table can be lowered by the installation of sub-soil drains. The position of the drains should be such that the water-table is maintained at least 1.2 metres below the formation level. In the case of concrete roads the base of the slab shall not be less than 30 cm above the anticipated water level in the said drains or the surrounding country. If the water-table stays below 1.8 metres, it may be ignored.

The back-fill used in the drainage trench should consist of a properly designed filter material, especially where the sub-soil is liable to cause silting of the backfill. The limit for the coarse particles of the filter material is based on the size of holes in the pipes (for perforated pipes) or the gap at the joints in the case of open jointed pipes. The size o most of the filter material must be greater than twice the size of this gap. In the case of porous pipes this requirement is unnecessary. These drains are sometimes called *French Drains*. The sub-soil drains are also called *Mitre* drains and are made cross-wise from the centre of the road sloping diagonally downwards towards the flow (staggered in the herring-bone fashion).

For concrete roads, the drains should not be laid in a herring-bone fashion as they are considered to disturb the formation creating non-uniform conditions of support to the concrete slabs.

The filter material can be clinker, gravel or brick rubble which does not contain fine particles and which can be properly packed. A suitable grading is 20 mm to 75 mm.

Size, spacing and depth of the laterals (cross drains) below formation level depend upon the nature of the soil, its moisture retentive power,

rainfall and the longitudinal slope of the ground. The depth varies from 0.45 to 1.2 metres. The drains should be laid not less than 45 cm and not more than 1.2 metres below the formation in sandy soils, not less than 60 cm in silty soils in wet locations and 0.9 to 1.2 metres in dense soils. Soils with high degree of capilarity require drains to be placed at a lower depth than in porous soils. In water-logged situations, the sub-soil drains to be effective should be at a sufficient height above high water level, the formation being raised to suitable heights. The mains and sub-mains may be 1.8 to 2.4 metres below the natural surface. The depth is taken to be at the invert of the drain. Pipes (laterals) should not be less than 75 mm in diameter and need not be more than 100 mm except where the ground is water-logged, or where the fall is less than 1 in 200, or the lateral is more than 150 metres long when it may sometimes be 130 mm. Sizes of mains will be bigger according to the laterals joining them, and may be 150 to 200 mm or more.

Drains should be installed well in advance of the road construction to allow as much time as possible for the establishment of equilibrium moisture conditions.

The following table may be taken as a guide for the spacing of sub-soil drains, in metres:

Nature of Soil For Properties of Soils, see "Soil Mechanics"	Depth of invert of drain	
	0.6 m to 0.9 m	0.9 m to 1.2 m
Clay	7.6 to 9.1	9.1 to 10.7
Sandy clay	10.7 to 13.7	12.2 to 13.7
Clay loam	13.7 to 16.8	16.8 to 19.8
Loam	18 3 to 27.4	25.9 to 30.5
Sandy loam	25.9 to 30.5	30.5 to 45.7
Sand	45.7 to 61.0	61.0 to 91.4

A thumb rule is known that a tile or drain line will draw 12 metres on each side for each metre of depth.

C.G. Elliot gives the following guidance for the spacing of sub-soil drains:

Close dense soils, largely clay	...	9 m to 12 m
Coastal plain lands composed of mixed clays with fine sand and of uniform structure	...	18 m
Alluvial grounds or heavy soils but with granular structure	...	21 m to 24 m
Alluvial glacial drift and sandy loam soils with clay sub-soils	...	30 m
Sandy lands and soils containing considerable quantity of vegetable matter	...	45 m to 60 m

5. ROAD ROLLERS

The various types of rollers which are used for compaction are: Sheeps-foot; Pneumatic-tyred; Smooth-wheel and Vibratory rollers.

Rollers vary in weight from 2 tonnes for bullock rollers to 15 tonnes for steam rollers. (Cylindrical hand driven rollers are lighter than 2 tonnes). Type of the roller to be used and its weight depends upon the nature of the work to be done, properties of the soil or the aggregate to be consolidated.

The following types of road roller are generally used :—

Cylindrical Roller : This is a light roller of iron, concrete, or stone; drawn by hand or bullocks. The size varies, but it is generally about one metre in dia. and about 1½ metres long. Ground pressure is about 7 kg. per sq. cm.

Sheeps-foot Roller: This roller consists of a hollow cylindrical steel drum on which projecting feet like truncated pyramids or staggered teeth are mounted. Various makes are available having different diametrs and widths of drum and different lengths and shapes of feet. The most common type is the one having two drums 1.22 metres wide and 1.06 metres in diameter with feet 20 cm long. The rollers are described either as taper-foot or club-foot rollers according to the shape of the feet.

Drawn by a tractor or bullocks. Not effective with sandy soils but is a useful compacting unit on clayey soils at low moisture content. Compacts more densely to a certain depth than any other plant. They should be used on layers of loose material less than 23 cm thick. (The thickness of compacting layer is kept about 5 cm more than the length of each foot.) The soil is supposed to be consolidated when the impression by the projecting teeth is not more than 12 mm deep or when the surface has been rolled 16 to 20 times. 10 to 20 passes are generally required to give complete coverage. The density of the consolidated soil should be about 1.48. The top layer has to be finished with a smooth wheel roller. These rollers are also called **Tamping Rollers.** Pressure on the feet may be increased by filling the drum with wet sand or some other material, which may be 4 to 7 kg/sq. cm for light rollers and up to 25 to 70 kg/sq. cm for giant rollers.

Pneumatic-tyred or Wheeled Rubber-tyred Rollers : These rollers are generally 6 to 10 tonnes in weight and fitted with a number of wheels (up to about 15), mounted on two or more axles, under a loading platform. Used for compacting cold laid bituminous pavements, soft base course material or layers of loose soil. These rollers are also suitable for compacting closely graded sands, and fine-grained cohesive soils at moisture content approaching their plastic limits, though the compaction is not as high as that with the smooth-wheel roller. They are particularly efficient when used to finish off the embankment compacted by sheeps-toot roller or on loose sandy soils. Motive power is provided by a tractor.

Smooth-Wheeled Rollers

There are two types of smooth-wheeled rollers : (i) Tandem rollers, and (ii) Three-wheeled rollers.

Tandem Rollers : These are two-wheeled light weight smooth-tyred rollers. The wheels are of equal width. Best machines for the initial rolling of cold-laid bituminous pavements and surface dressings. Not suitable for compacting base courses of hard material. Ground pressure is 10 to 17 kg/sq. cm. Usual weight is 2 to 8 tonnes.

Three Wheeled Rollers : Power driven (steam or diesel). Commonly used in India. Smooth-wheel rollers are most suitable for consolidating stone soling, gravel, sand, hard core, ballast and surface dressings. Not suitable for consolidating embankments and soft sub-grades, but are better suited than any other plant for compacting silty and sandy soils and with fewer passes. When the moisture content is a little more than optimum it will compact more easily. Usual weight is 8 to 10 tonnes.

The two types (steam and diesel) are very much alike, the difference being mainly in power unit. Adjustable weight devices are available which can be fitted to the wheels so that the rolling pressure can be varied to suit different consolidation requirements. When engaged on heavy work, the sliding weights must always be at the rear of the roller. The sliding weight must never be moved when the roller is on a gradient.

The steam road roller can stand heavier wear and tear and is much simpler to work than the diesel roller but it takes over an hour to start up and cannot be temporarily shut off, while the diesel type can be started up and shut down in a few minutes and does not consume fuel when standing temporarily idle on a job. Steam road rollers are now getting out-dated. Diesel rollers are cheaper in running cost.

The weight of a 8 or 10-tonne roller in working is about 2 to 3 tonnes (varying from 20 to 30 per cent) above its normal weight. Width of contact between roller wheels and road surface is about 75 mm.

Some rollers are made with its prime movers or engine as a separate unit which is a tractor, and is mounted on the roller, and which has its own advantages.

The rolls can be ballasted with sand, water or water and sand to increase the weight of the roller. (The weight of a roller is prescribed as either ballasted or unballasted.)

Scrappers are provided on all the wheels in adjustable positions covering the full width of the roll, with water sprinkling arrangement, for scraping of the mud and keeping the wheels clean during rolling.

The maximum grade a road roller can climb is 1 in 5.

Vibratory Rollers : This type of roller is fitted with one or two smooth surfaced steel wheels 0.9 m to 1.5 m in diameter and 1.2 m to 1.8 m wide. Self-propelled vibratory rollers are now available weighing from 4 to 6 tonnes. Vibrations are generated by the rotation of an eccentric shaft inside. A vibratory roller is used for compacting granular base courses. It is sometimes used for asphaltic concrete work.

Working and Care of Diesel Road Rollers

(*i*) When starting a new roller or the one that has been standing for sometime, the cylinder head cover of the engine should be removed and some lubricating oil poured over the valve stems and tappets. Also pour about three tea-spoonfuls (and not more) of lubricating oil over the piston through the counter-sunk hole leading to the air-intake part of the top of each cylinder head.

(*ii*) Smear all the moving parts with oil.

(*iii*) The fuel system must be primed. Open the main cock on the fuel tank and move the fuel pump cut-out lever to "running" position. Unscrew and ease the air vent screws on the fuel oil filter and fuel pumps and retighten them as soon as oil begins to trickle out without any air bubbles. De-compress the engine and give a few turns to the flywheel with the starting handle until a creaking sound of the nozzle inside the cylinder is heard; if no creaking sound is heard, disconnect the fuel delivery pipe union on the injector and give a few more turns to the engine until fuel flows freely. Retighten the union and wait for the creaking noise as before. Give the engine a few sharp turns, turning of the engine slowly (when the oil starts trickling) will be of no use. When the engine has attained full speed, the oil pressure should be 4.5 to 7 kg. Never run the engine on no-load or on light load for a long period.

(*iv*) In cold weather, after putting in the lubricating oil as described in (*i*) above, give the engine a few turns, keeping the fuel cut-out lever on the "off" position, before starting.

(*v*) To stop the engine, take off the engine load and move down the fuel pump cut-out lever to the stop position. Decompress the engine just before it is on the point of stopping. Never stop the engine by shutting the fuel supply main cock, otherwise it would be necessary to prime the fuel system again for the next start.

(*vi*) The hand brake is normally used as a parking brake and it can also be used to assist the foot brake on steep gradients.

Periodical attention after every 8 running hours:—

Check the lubricating oil level, the water level and the fuel supply. Clean the lubricating oil filter by giving about one turn to the top handle. Drain the valve tappet chamber.

During working of the engine see that the cooling water temperature remains between 50 to 80 deg. C., and the lubricating oil pressure between 4.5 to 7 kg.

6. ROLLING

The weight of the roller to be used should be according to the size and hardness of the stone and the thickness of the layers of the material to be consolidated. For ordinary consolidation of soft stone, 6 to 8 tonnes roller is good; for brick aggregate, 6-tonne; and for highways and roads near towns, 10 to 12-tonne rollers are satisfactory for hard stone

metal up to about 50 mm size. The depth of spread of loose material of any type which can be compacted effectively by the roller should be determined by actual test at site. Generally the depth of loose soil should not exceed 15 to 23 cm and that of stone metal 90 mm. If more material is to be consolidated it should be done in layers.

Rolling should commence at the edges and progress towards the centre except in super-elvated portions where it should proceed from the inner edge to the outer. Each pass of the roller should uniformly overlap not less than one-third of the track made in the preceding pass.

The number of passes required of a roller to give good compaction of any material should also be determined by actual test at site. With good tough metal as many as 50 passes over the dry material may be required to give good results.

The roller should be operated at the minimum speed which the roller gearing will permit while consolidating base and soling courses and should be about 3 km per hour for smooth-wheel roller and between 5 to 6 km per hour with pneumatic-tyred and sheeps-foot rollers. For lighter work the speed of a smooth-wheel roller may be increased to 5 to 6 km per hour. (A roller travels about 13 km in a day while working on consolidation.)

Working with one roller only for metalling, the length of road closed to traffic should be three times the length which can be consolidated by the roller. The first third will be scored, the middle third spread with loose metal, and the last third consolidated. Working with two rollers grouped, the first will carry out dry rolling and the second wet rolling and surfacing so that progress is doubled, and the length of road isolated will be twice as great. A 3-metre wooden straight edge should be kept for checking the surface during rolling.

Methods of Base Compaction

Material	Weight of Roller tonnes	Max. thickness of loose material to be compacted in cm	Moisture content (per cent)
Hard brickbats or crushed rock ...	10 or more	20	Not important
Ashes, clinker ...	8	15	12—20
Stable clayey gravels ...	8	23	6—10
Clayey sands ...	6—8	23	10—18
Lean-mix concrete	6—8	23	...

Recommended plant for particular soils and the number of "passes" for sufficient compaction: (See also similar table in the Section on "Irrigation" under "Embankments").

Plant	Clayey soil	Silty soil	Sandy soil	Sand
Smooth-wheel roller 7 tonne ...	...	8—16	8—16	8—18
Pneumatic-tyred roller ...	...	...	8—18	8—16
Sheeps-foot roller	8—16	...	...	...
Smooth-wheeled roller ...	4—8	...	...	...

(Central Road Research Institute, Delhi).

Increasing number of "passess" (*i.e.*, compacting more) may not produce a higher density of the soil, but on the other hand there may be reduction in the relative compaction.

The compaction obtained is determined by measuring the dry density of the compacted soil, usually by the sand replacement method (explained under "Soil Mechanics").

Out-put of a 10-tonne Road Roller (Approx):

A roller consolidates about 20 to 25 cu. m of hard road metal, well rolled to full compaction, in a day (working eight hours), For soft metal hard rolling is inadvisable and in such cases 55 to 80 cu. m may be consolidated. (If more work is pushed through it may mean consolidation not fully done.)

For rolling soling, about one kilometre of a 3.8 metre wide road should be done in a day,

For rolling on re-metalling work, 300 to 400 sq. m are consolidated in a day.

In the case of rolling on pre-mixed metal (base course), the rate of consolidation should not exceed 140 to 180 sq. m per hour.

Rolling Surface Dressings : (Approx.) with 8 to 10-tonne smooth-wheeled roller.

1600 to 1800 sq. m per day	On 12 mm chips, spread at about 1.5 cu. m per 100 sq. m
2000 to 2200 sq. m per day	On 6 mm chips, spread at about 1 cu. m per 100 sq. m
1000 sq. m per day	On two coats of surface dressing done together
500 to 600 sq. m per day	Consolidating premixed carpet

In bullock rolling (with light-weight rollers) long lengths of rolling are necessary to prevent waste of time in turning round. With stream or diesel rollers short lengths are no abjection and are less impediment to traffic; about 1/5th kilometre should be the limit, 6 to 8 trips of the roller are generally sufficient for surface dressing work.

Compacting Efficiency of Different Types of Rollers

(a) For compacting water-bound macadam, the compacting efficiency of a vibratory roller is higher than that of a 6-tonne smooth-wheel roller, and is equal to that of a 10-tonne smooth-wheel roller.

(b) For bituminous pre-mix surfaces, and for compacting silty and sandy soils, there is no significant difference in the compactive performance of a vibratory roller and a 6-tonne smooth-wheel roller.

(The vibratory roller considered in the experiment was of 1.6 tonnes weight, self-propelled, with two speeds of 15.24 m and 30.48 m per minute, with vibration frequency of 2,400 to 3,400 cycles per minute.)

(*From IRC Road Research Bulletin No. 3*)

High Speed Diesel oil is used in diesel road rollers, and consumption is about 9 litres per hour under normal working conditions for a 10-tonne roller. Consumption of oil depends upon the type of engine and its horse power.

7. PREPARATION OF SUB-GRADE

The proper preparation of the sub-grade for any road is of utmost importance before the road structure (pavement) is laid over it. Unless the foundation is hard and firm and properly shaped the resulting road will be bad and will remain bad. Special attention must be given to the compaction of the sub-grade and its drainage. Improperly made sub-grades are the cause of great waste of money and frequently when the road breaks up blame is placed on the particular type of the binder used whereas the real cause was probably a badly prepared sub-grade.

A sub-grade is not properly drained until it has been impossible for any rain water to remain upon it for a longer period than 12 hours and impossible for any surface or irrigation water to increase the moisture content. It should also be made impossible for any water to remain stagnant in the road side drains and gain admission to the road surface by capillary attraction. Many failures in road construction work are due to the lack of proper drainage of the sub-grade.

While the above is true for normal sub-grades the notes about moisture content in the soil should not be forgotten. Cases may occur where in order to maintain the moisture content in the soil by capillary attraction it is advisable actually to arrange for water to remain in the side drains. Soil with a great excess of sand would be an example. Precautions are necessary in the case of sub-grades composed of cohesive soils to prevent volume changes arising from variations in moisture content.

A certain degree of moisture present in the materials keeps them in a stabilized condition; but the moisture content is liable to vary with every change of the weather and a variation of it will cause a change in the support value of the material. The sub-grade must be prevented from becoming so dry that it breaks up from want of cohesion, or so wet that it forms mud.

The only material that can be used for the sub-grade is the natural soil. All soils are composed of sand, silt and clay in varying proportions; the proportions of the materials and their properties affect their stability under load. An ideal road material is one in which all these are present in proper proportions so as to obtain the maximum stability, otherwise an unstable road will result.

An intimate and compact mixture of the following will make a stabilized soil:

Sand	—	70 to 85 p.c.	It will be usually sufficient to have 70 p.c. sand and 30 p.c. clay and silt together.
Silt	—	10 to 20 p.c.	
Clay	—	5 to 10 p.c.	

(See also under "Soil Stabilised Roads.")

Campaction of the soil is caused by traffic which results in the differential settlement of the sub-grade. Sandy sub-grades are particularly susceptible to compaction especially where vibrations are produced. Clay sub-grades however may also be liable to plastic deformation under repeated loads. Soil below a sand layer, if clay, will settle, and should be tested for load.

Where the sub-grade is of low bearing capacity, some method should be adopted for increasing the bearing capacity. Soils with bearing capacity of less than 5 1/2 tonnes per sq. metre are not usually suitable for sub-grades under heavy traffic loads. (See under "Design of Pavments".) Patches of soft soil, if any, should be excavated and removed and substituted by good granular soil. Sometimes about 15 cm or more of the unreliable sub-grade material is removed from the site of the road and good soil put in instead.

Suitability of Natural Soil for Road Work

In order to determine the suitability of a particular soil it is essential to know the proportion of each material in the soil and also whether the particular material possesses the properties required of it. For instance, a road soil that is 95 per cent sand will not cohere, and a clay that does not possess the property of cohesion is useless as a road material and will do nothing but make dust. The following field tests can be made:

(*i*) To find the proportion of sand in the soil:

Take a sample of the soil (dry) and weigh it. Put it in a glass and fill with water. Agitate it and pour off the clay. Do this several times until nothing but sand remains in the glass. Dry the sand and weigh it. The result will give the percentage of sand in the soil. The remainder is clay and silt.

(*ii*) To determine the proportion of clay and silt :—

Silt is generally darker in colour than clay and a sample that contains too high a percentage of silt will not have the characteristic brown colour of clay. Silt settles more rapidly than clay. If the sample is put into a glass and mixed with water and allowed to settle the clay will remain muddy while the silt will settle within a few seconds. A sample that clears very quickly has too much silt, some clay should be added to it.

(*iii*) *Qualities of clays*

Pure clay is very retentive of moisture and becomes plastic and unstable when wet, and as it abrades easily, produces all dust when dry. The extent to which these objections occur depends on whether the clay is of the "slaking or non-slaking" variety. The slaking variety is undesirable as it is more muddy in wet weather and more dusty in dry weather. To determine the qualities of various clays by the slaking test :—

Make several balls of the same size of the different clays and dry them out. Place them in water so that they are covered entirely. The balls which hold their shape longest after being placed in water have the highest resistance to slaking, and that clay is to be preferred for use in the road. It is important in this test that if various clays are being compared, the proportion of sand in each sample should be the same and should not exceed about 25 per cent. If sand is in excess it should be removed before doing the slaking test.

If the clay is of the slaking variety the balls will disintegrate almost as soon as they are put in water, such a clay is not suitable for road work. Samples that contain too much silt will not show good non-slaking qualities and will break up at once in water. Clay requires to be added in such samples.

To test the suitability of sand—Place a sample of the sand in a vessel containing water and agitate the water until the sand is thoroughly in suspension. Then when the sand has been allowed to settle pour off the water slowly. A good quality sand will not be carried off with the water but will remain in the vessel until practically all the water has been drained off. A bad quality sand will not meet this test and is not suitable for use on roads.

Compaction of the sub-grade. Compaction has been dealt with in detail under "Embankments" in the Sections on "Irrigation" and "Soil Mechanics". Clay soils in cuttings do not generally need further compaction. In fact, heavy compaction of such a soil may be harmful as it may destroy the internal structure of the soil, resulting in loss of strength.

Granular soils may, however, need compaction. To obtain a high state of compaction of granular soil in cuttings, the best method is to excavate the soil to a depth of 60 cm below the final formation level and then replace it and compact it in 15 cm thick layers. Another method is to compact the soil formation with either frog rammers or vibrating plate machines. These machines compact granular soils satisfactorily to a depth of about 30 cm.

It is considered inadvisable to compact cohesive soil subgrades below their optimum moisture content in cases where they are likely to be subjected to the ingress of moisture. (See under "Rolling".)

See also under "Soling or Bottoming" in the pages following.

8. CONSTRUCTION OF BASES & SUB-BASES

Granular Sub-Base

The work consists of laying and compacting granular material such as natural sands, moorum, gravel, laterite, kankar on other naturally occuring or artificial soft aggregates, on prepared sub-grade.

Grading Requirements of Granular Sub-Base Materials

Sieve designation	Per cent by weight passing the sieve	
80 mm	100	The fraction of material passing 20 mm sieve shall give a CBR value of at least 20 per cent or more after preparing the samples at maximum dry density and optimum moisture content, and soaking the same in water for 4 days. (See under "Thickness of Pavement Design" in the page following.)
40 mm	85-100	
10 mm	45-100	
4.75 mm	25-85	
600 micron	8-45	
75 micron	0-10	

The thickness of loose layers shall be so regulated that the maximum thickness of the layer after consolidation does not exceed 150 mm. Consolidation shall be done with 8 to 10-tonne smooth wheeled roller after sprinkling the loose material with sufficient water.

Stabilised Soil Sub-Base

The work consists of laying and compacting a sub-base course of mechanically stabilised soil or soil stabilised with lime or cement on prepared subgrade.

Blending materials for mechanical stabilisation may be gravel, crushed stone, crushed slag, brick metal, soft aggegates like laterite and kankar, natural sand or clay depending upon the grading requirements. The thickness of any layer to be stabilised shall be not less than 100 mm with maximum up to 200 mm, when compacted. Care shall be taken to see that the campaction of cement stabilised material is completed within two hours of its mixing, or such shorter period as may be found necessary in drying weather.

In the case of lime or cement stabilised construction, it shall be ensured during rolling that the campaction plant does not bear directly on hardened or partially hardened treated material previously laid other than what may be necessary for achieving the specified compaction at the joint. The final surface shall be well closed, free from movement under compaction plant, and any compaction planes, ridges, cracks or loose material. The sub-base course stabilised with lime or cement shall be suitably cured for 7 days, or less as necessary, soon after which subsequent pavement courses shall be laid to prevent the surface from drying out and becoming friable. No traffic of any kind shall play over the completed sub-base.

Grading of Mixed Materials for Mechanical Stabilisation—See Table "Typical Grading Limits for Soil-Aggregate Mixtures" under Water-Bound Macadam Roads.

(Based on IRC Specifications)

Crushed Cement Concrete Base/Sub-Base Course

The work consists of breaking and crushing the existing damaged cement concrete pavements and recompacting the same as base/sub-base course in one or more layers. It may also include treating the surface of the top layer with a penetration coat of bitumen. The cement concrete slabs are crushed to a size not exceeding 75 mm, grading to be as for water-bound macadam, as far as possible. The course should be constructed as water-bound macadam except that no screenings or binding material need be applied.

Bitumen for penetration macadam is sprayed at the rate of 25 kg per 10 sq. metres area. Immeditately after the application of binder, the key aggregates should be spread uniformly on the surface at the rate of 0.13 cu. metre per 10 sq. metre area. If necessary, the surface should be broomed to ensure uniform spread of chippings.

Except in the case of bituminous constructions, *shoulders* should be constructed in advance of the laying of pavement courses. The compacted thickness of each layer of shoulder should correspond to the compacted layer of the pavement course to be laid adjacent to it. After compaction, the inside edges of shoulders should be trimmed vertical and included area cleaned of all spilled material before proceeding with the construction of the pavement layer.

Base Course. In a road structure the base course is the layer immediately under the surface course (or wearing surface). Because the base course lies under the pavement surface, it is subject to severe loading. It follows, therefore, that the materials in a base course must be of high load carrying quality, and the construction well done. Under some conditions two layers of base course, one of a higher type than the other, may be used to good advantage that will result in a lower overall cost. A material not suited for use directly under the pavement but cheaper than the usual base course material may be interposed between base course and subgrade. This course is designated as "sub-base". The thickness of base-course layers is controlled by the character of the underlying subgrade over which it distributes the wheel loads delivered from above.

9. PREPARATION OF BASE FOR LAYING BITUMINOUS COURSES (Based on IRC Specifications)

The base to be prepared may be an existing water-bound macadam course or black topped surface. The work consists of repairing potholes and bringing the surface to the specified lines, grade and cross sections, through a levelling course, if necessary.

Patching of Potholes. Potholes should be cut as nearly square or rectangular as possible, the sides being cut vertical (preferably undercut to help in keying-in of the new material), and scarified to a depth of 40 to 63 mm, and all loose material cleaned out, In the case of water-bound macadam surfaces the potholes should be filled with coarse aggregate and screenings (of the same specifications as for water-bound macadam roads) and compacted with heavy hand rammers. In the case of existing black top surfaces, the bottom and sides of all the potholes and depressions should then the painted thinly with hot tar, bitumen or emulsion (as the case may be) and filled with premixed chippings, the size of the metal depending upon the depth of the potholes. The metal should be rammed in layers of 25 mm at a time (the hand rammer being dipped in water from time to time so that the coated metal may not stick to it). Where the depressions are slight say, up to 25 mm depth, the best and quickest method is to brush the spot clean, fan it with a gunny bag to remove the dust and apply a small quantity of cold emulsion or hot tar brushing it evenly over the area, and then covering it with small chips and ramming with a hand rammer.

The finished filling should be "proud" on the surface by about 6 mm to allow for subsequent settlement under traffic. After repairs to a patch the surface should be lightly dusted with sand before opening it to traffic so that the coated material may not be picked up by the wheels. Gauge of the patching metal should not normally exceed 40 mm.

Laying a Levelling Course. Where a levelling course is necessary, it should be laid after filling the potholes. The base on which the levelling course is to be laid should be thoroughly swept and scraped clean of dust and any other extraneous material. The levelling course can be either of the following two types depending upon the type and thickness of irregularities required to be made up:.

(i) Type A: Where the irregularities to be made up are of non-uniform character and need low thickness of leveliing course. Use premixed bituminous material given under "Open Graded Premix Carpets 2 cm thick".

(ii) Type B: Use the premix specifications given for 75 mm and 50 mm bituminous macadams, using 2.5 per cent binder instead of 3.5 per cent prescribed therein.

After laying the levelling course it is compacted to the desired levels, grade and camber after applying a tack coat.

After applying a tack coat the levelling course is laid and compacted to the desired levels, grade and camber.

10 SOLING OR BOTTOMING

The primary function of soling is to distribute the load over a soft subgrade (ground surface) in such a way that there will be no sinking of the road crust into the subgrade under the load of the traffic on the road.

Soling Stones. The size of stone should not be more than 22.5 cm, nor less than 10 cm in any direction, and height equal to the soling course with tolerance of 25 mm.

Soling Bricks. Soling bricks are second class bricks which are slightly overburnt, or jhamma, and of reasonably regular shape. The soling bricks should not absorb more than one-fourth their weight of water after one hour's immersion and should show no signs of efflorescence on drying. The soling bricks need not have 'frogs'.

Stacking of Materials for Road Works at Site

The *soling stones* should be stacked in units of 1 to 1.25 metre height, and of uniform width and length, and all uniformly distributed along the road. Soling stones are measured in cu. metres. Actual dimensions of stacks should be recorded and the total quantity reduced by 15 per cent to arrive at the net quantity for payment. Lengths, breadths and height should be measured correct to a cm. (CPWD specifications)

The soling bricks should be stacked in units of two-brick width, 10-brick high and of such length as to contain sufficient bricks for soling of 30 metre length of the road. Stacks should be uniformly distributed along the road.

Stacking of moorum, bajri, stone grit, etc. Ground where stacks are proposed to be made should be dresed to a uniform level and all humps, depressions, etc. removed or filled up. Stacks are made in units of one cu. m, with wooden boxes open at both ends and of 2 x 2 x 0.25 m dimensions.

Stacking of stone aggregate. Ground where stacks are proposed to be made, should be dressed to a uniform level as for above. Stacks are made in units of one metre top width, 2.2 m bottom width, 60 cm height and length in multiples of 3 m. Where berm width is limited or for repair works, stacks may be in units of 40 cm top width, 1.4 m bottom width, 50 cm height and length in multiples of 3 m. Templates of wood or steel should be made for making the stacks. The stacks should be uniformly distributed along the road. Stone metal should not be dumped in heaps directly on the area where these are to be laid.

Stone metal is measured in cu. metres and correct to a cm to arrive at the contents. The total quantity so arrived is reduced by 7.5 per cent to arrive at the net quantity for payment. (CPWD specifications)

Preparation of the Subgrade for Laying Soling. Soling is normally laid in a trench made in the subgrade (or ground surface). The width of the trench should be 30 cm (15 cm on either side) more than that of the proposed finished road surface, and depth below the proposed finished level equal to the combined depth of soling and metalling (due allowance being made for consolidation). Depth of soling is 15 cm consolidated in cutting and in good soil, and 22.5 cm consolidated in filling, made-up soil, or poor soil, which loose layers 17.5 cm and 26.0 cm respectively have to be laid with suitable templates. The trench dug for the soling should be made to have a profile of the same camber as the proposed finished road surface.

All soft places in the subgrade should be excavated and filled in with firmer spoil and well rammed before laying the bottoming or other foundation. The roller shall run over the subgrade till the soil is evenly and densely consolidated. All undulations in the surface that develop due to rolling should be made good with earth or quarry spoils and the surface re-rolled. 8 to 12 tonnes power roller may be used and which shall pass minimum of five runs on the subgrade.

No soling or metal should be laid on a new road in filling (where additional earth has been put) unless it has been exposed to one rainy season, and the formation brought to proper level again and consolidated.

A road can also be opened to traffic after soling has been laid and pending metalling, any unevennes developed made good.

Also see under "Preparation of Sub-grade".

Soling Stones – laying and packing. Stones for soling should be of height equal to the thickness of the soling with base area of not less than 250 sq.

cm, nor more than 500 sq. cm, and not to exceed in length and breadth twice its thickness. For sections in good soil, 13 to 15 cm boulder (when measured across the proudest part in any direction) are generally specified to be increased to 23 cm if the subgrade is of poor soil.

Soling stones should be hand packed as close as possible with their broadest side downwards and greatest length across the road, to the required camber of the top surface of the road by laying correct to the templates placed 15 metres apart. Joints should be staggered. Large size stones should be placed at the edges and centre of the road. Gauge pegs are driven at small distances to indicate the thickness of the stones to be laid. All interstices between stones should be wedged in with smaller stones, well driven in to make tight packing and complete filling of the interstices.

Consolidation of soling The soling should be thoroughly consolidated with power rollers of 8 to 10 tonnes weight, starting at the edges and working towards the centre. In case of super-elevated curves, rolling should commence from the inside (lower) edge and progress gradually towards the outside (upper) edge. Roller should be run over the same surface for at least eight times till the soling course it well consolidated. The surface should be checked by templates, any hollows formed filled in so as to conform with the gradient and camber of the final road surface level desired.

A layer of moorum or sandy soil, of thinkness not less than 2.5 cm, should be spread over the consolidated stone soling, watered and rolled with a light roller before the wearing course is laid, so as to form a cushion.

Boulders should not be laid directly on a clayey soil subgrade. The fine soil will enter the open graded water-bound macadam through the voids in the soling boulders due to change in moisture content in the clay soil or due to repetitive wheel loads and weaken the pavement. A layer of 8 to 10 cm of fine graded granular material laid over the foundation will prevent working up of the clay into the base course.

For one unit of stone (boulders) soling (taken as consolidated) 1.38 units of stone are collected.

Soling should not be laid in two layers as the top of the bottom layer has generally an uneven surface, and the upper layer is likely to rock under traffic. In cases where the full depth of soling specified cannot be obtained by a single rubble layer, a layer of large size metal of the necessary thinkness should be laid over the rubble to fill the interstices in it and to give the necessary depth of soling.

Some engineers say that if a layer of stand or small size gravel, about 10 cm thick, is spread and consolidated over the ground under the soling it will serve as a cushion underneath the road and prolong its life.

Brick Soling

A cushion of 12 to 25 mm of sand is recommended under brick soling. Where brick-soling is specified, the bricks are laid flat or on edge with close fine joints using wooden mallets. Bricks are laid with their length along the width of the road with joints evently spaced parallel and at right angles to the centre line of the road, adjacent layers breaking joint. One-

brick profiles are made at right angles to the length of the road at 2.5 to 3 m intervals and one in the centre lengthwise. Edging is made with single bricks laid on edge or on one end, parallel to the road. The profiles are first made and then cross bricks filled in. Soling with flat bricks may be laid in two or more courses breaking joints in successive course also with a layer of sand between the courses to act as a cushion. In places where the soil is loamy, the soling may be of brick-on-edge as this subgrade affords good support to the bricks. After laying the soling bricks, sandy soil, moorun, grit or dry sand should be spread over the soling to a thickness of 25 mm and worked into the joints by brooming so as to fill the joints completely. This earth must be allowed to remain as a protective covering to the soling until such time till the road is metalled.

Brick soling should be rolled with a light roller of say 6 to 8 tonnes weight without crushing and brought to camber.

Hardcore Soling. May be of overburnt brickbats, kankar, laterite, moorum, etc., well consolidated into the soling trench. This type of soling serves well over hard and firm subgrades. Size may be 100 mm for soft material, but should not exceed 80 mm gauge where crushed or broken stones are used as otherwise good consolidation will not be done.

11 SELECTION OF STONE METAL

General Properties of Road Metal—Shape and Texture

In order to resist successfully weathering, abrasion and crushing due to stresses brought on by traffic, the ideal material for road metal is a gloss-textured, hard and tough stone, broken into angular fragments of sizes conforming to the required proportions to give correct grading. "Toughness" is opposite to brittleness and is the property which enables the stone to resist breaking when struck with a hammer, and this quality is essential in a road metal to withstand the impact blows caused by traffic The best quality is produced by hand-breaking with hammers, as the fracture occurs along the natural line of cleavage, leaving sharp edges.

For water-bound macadam roads the metal should possess good cementation properties in addition to the above qualities. It is this quality which gives kanker, limestone and laterite roads an excellent riding surface particularly in dry weather, provided that the traffic is not too great. But, such roads wear away very soon into continuous small ruts and are dusty when dry and muddy when wet.

Particle shape. A great deal of strength of a road is derived from the mechanical interlock of the aggregates which vary with the aggregate shape. Aggregates of angular shape, square and sharp with rough surfaces achieve the best mechanical interlock and offer greatest resistance to displacement under the repeated shocks of traffic and make the strongest surface. Stones with rounded edges have less interlocking properties than broken stone, Moreover, more rounded the aggregate the smoother is the surface texture likely to be and the more difficult will it be to ensure satisfactory and continued adhesion of binder to stone. A flaky aggregate or oblong pebbles should always be avoided. (Flaky means that thickness is small as compared to the sides.)

Gravel, generally being rounded in shape, is less satisfactory as aggregate than the angular fragments derived from crushing. The worst kind of metal is that derived from round boulders of small dimensions and varying kinds of long exposed rocks. Where boulder metal has to be used, the diameter of boulders must be not less than 125 mm, and the gauge of metal not more than 40 mm, otherwise proper consolidation will be impossible. Soft sand-stones should never be used. Hard uniform limestone is easily consolidated and gives a smooth surface.

Most of the road aggregates are crushed or broken natural rocks. Based on the strength property, road metal is divided into hard aggregates and soft aggregates. Moorum, laterite, kankar, slag and brick aggregates are considered as soft aggregates. The specific gravity of an aggregate is considered to be a measure of the quality or strength of the material. Stones having low specific gravity values are generally weaker than those having higher values. Also, stones with higher water absorption ratio are porous and weak. A piece of stone should not absorb more than one per cent of its weight of water.

For bituminous surface treatment two qualities for stones are essential:

(a) The capacity of retaining the film of bituminous material applied to the stone in all weather conditions especially in wet conditions, and
(b) The capacity to carry the traffic load without crushing.

Bitumen and tar adhere well to stones normally used as road aggregates, provided that the latter are dry and clean, but water has a great affinity for stone surfaces than has either bitumen or tar, and when a coated stone comes into contact with water there is a tendency for the binder to be displaced. The degree to which binder will adhere to the presence of water depends partly upon the properties of the stone, of which the texture is the most important factor, and partly upon those of the binder.

Field Tests for Suitability of Road Metal

Hardness of a stone can be ascertained by scratching it with the point of a pen-knife, if it produces a deep scratch, the stone is too soft as a road metal. But all limestones are readily scratched and in that case a shallow scratch may be taken as satisfactory.

Brittleness of a stone can be tested by hitting it with a sharp blow with a small hammer of about 1/4 kg weight. If the stone breaks readily, especially into several pieces, it is too brittle for road work.

Examination should be carried out on a freshly broken surface and not on the one which has been exposed to the weather for sometime. A good stone should show a bright, clean crystalline appearance, a dirty brown or pale geenish tinge is generally indicative of poor quality of stone. The stone should be free from patches of glassy material and from large cavaties.

Laboratory tests are for: Crushing; Abrasion or Hardness; Attrition or Wear; Toughness (Page Impact test); Cementation; Water Absorption; Specific Gravity; Bitumen Adhesion.

The first quality varies with different varities of stone, but basalt, dolerite and hard limestones are considered good, while granite and quartzite

(unless the surface texture is suitable and the material is not dusty) are comparatively bad, although quite hard for load carriage capacity. Glassy or very smooth rooks are not generally considered suitable as such surfaces do not promote good adhesion and stripping may readily take place. A surface which is granular such as that exhibited by some sandstones, and oolitic and other limestones, tends to crumble or be mechanically weak and in many cases the material is highly absorbent towards bitumen. Dusty surfaces which often occur with granular rocks and also with some quarzites are to be avoided as the dust prohibits proper contact between aggregate and bitumen with unsatisfactory results. The group of rocks which exhibit a crystalline surface texture are found the most suitable for adhesion with bitumen.

The second quality is determined by crushing and abrasion tests. Basalt or dolerite are generally good in this test also. The resistance to abrasion of limestones vary considerably in different types and even the hardest cannot compare with hard sandstones or igneous rocks, although limestones are highly valued as road-stone because of their good bitumen adhesion.

The rougher the texture, the greater the porosity, and the lower the specific gravity, the greater is the amount of binder required.

IRC Standard Sizes for Broken Stones and Chippings for Road Work

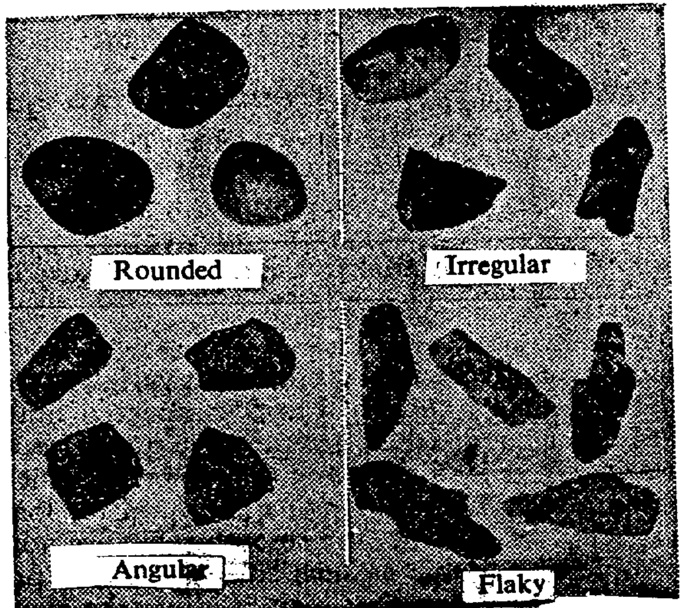
Size standard or nominal	*Wholly passing square mesh sieve of size*	*Wholly retained on square mesh sieve of size*
90 mm	100 mm	63 mm
63 mm	75 mm	50 mm
50 mm	63 mm	40 mm
40 mm	50 mm	25 mm
32 mm	40 mm	25 mm
25 mm	40 mm	20 mm
20 mm	25 mm	12.5 mm
12 mm	20 mm	10 mm
10 mm	12.5 mm	6.3 mm
6 mm	10 mm	2.36 mm
Sand	2.36 mm	75 micron
Medium coarse sand or fine grit	1.70 mm	180 micron

The fraction passing 2.36 mm sieve and retained on 75 micron sieve are considered as *fine* aggregate.

It should be noted that the classification of sand in bituminous construction differs from that specified in Soil Mechanics.

The test for determining the aggregate grading is conducted by shaking a weighed quantity of dry sample over a set of standard sieves of square openings, and weighing the aggregate retained on each sieve. The results are reported as total per cent by weight passing each sieve.

The shape and texture of a stone are defined according to the following scheme:



TYPICAL SHAPES OF ROAD STONES (12 mm size)

COMPARISON OF DIFFERENT STANDARD SIEVES FOR ROAD WORKS

Indian Standard	Equivalent BS & ASTM	Indian Standard	Equivalent BS No.	Equivalent ASTM No.
100 mm	4 inches	4.75 mm	4-3/16 in.	4
80 ,,	3 ,,	2.36 ,,	7-1/10 in.	8
63 ,,	2½ ,,	2 00 ,,	8	10
50 ,,	2 ,,	1.70 ,,	10	12
40 ,,	1½ ,,	1.18 ,,	14	16
25 ,,	1 ,,	850 micron	18	20
20 ,,	¾ ,,	600 ,,	25-1/42 in.	30
12.5 ,,	½ ,,	500 ,,	30	35
10 ,,	3/8 ,,	425 ,,	36	40
6.3 ,,	¼ ,,	300 ,,	52-1/84 in.	50
		180 ,,	85	80
Also given in Section 8		150 ,,	100	100
"Reinforced Concrete"		75 ,,	200	200

12. WATER-BOUND MACADAM ROADS

The term water-bound macadam means a road made of coarse aggregates mechanically interlocked by rolling, and bonded together with screenings, binding material (which fill in the voids), and water. laid on a prepared subgrade, sub-base, base or existing pavement, as the case may be. (Binding is achieved by stone dust as filler, with water.) Water-bound macadam may be used as a sub-base, base course or surfacing course. The aggregates used are either crushed or broken stone, crushed slag, over-burnt brick aggregate, or one of the naturally occurring aggregates such as kankar, laterite.

Thickness of stone metal to be consolidated depends upon the intensity of traffic, weight of the commercial vehicles that would use the road, bearing capacity of the soil, and the hardness of the metal to be used. Thickness of loose metal (and which is collected as site) generally specified is: 2-10 cm, 1-13 cm, 1-10 cm, which will consolidate to about 16.0 cm, 10.0 cm, and 7.5 cm respectively. Thickness of consolidated metal should not normally be less than 11.5 cm except where the road is to be subsequently treated with bitumen or tar, where it may be 7.5 cm with hard metal. (For thinkness of pavement crust that should be built for a particular road, see under "Thickness Design of Pavement"). It is recommended by IRC that the thickness of a single compacted layer should no-exceed 10 cm for (hard stone). Where iron-tyred cart traffic pret dominates, a thicker coat with hard metal should be specified. The "intensity" of load of a laden iron-tyred cart is more than that due to a 12-tonne road roller, under its iron tyre. (Iron-tyred wheels of carts are now being rapidly replaced by rubber tyres.)

For important roads on medium sub-grades, two courses of stone metal each 7.5 cm thick over a sub-base of 7.5 cm of gravel or clinker should be provided. For light traffic roads, one layer of 11.5 cm (15 cm of loose metal) thickness instead of 2-7.5 cm may be provided.

Anything above 7.5 cm of compacted thickness of hard metal and 11.5 cm of soft metal should be laid and compacted (rolled) in two or more separate courses. The lower course being nearly consolidated. not completely so, before the next layer is put on. No blindage is used on the first layer.

If the water-bound macadam is laid in two distinct sizes of aggregate to be applied in two layers, each should be laid separately and one to fill the void in the other, (the lower layer rolled before the upper layer is put on), it will make a much better and smoother surface.

Where water-bound madadam is to be laid over an existing black topped surface, 50 mm furrows should be cut at an angle of 45 deg. to the centre line of the road at one metre intervals in the black top surface before laying the coarse aggregate.

Size and Grading Requirements of Course Aggregate for Water-Bound Macadam

IRC: 19-1972

Grading No.	Size range	Sieve designation	Per cent by weight passing the sieve
1	90 mm to 40 mm	100 mm	100
		80 mm	65—85
	Jhama Brick aggregate	63 mm	25—60
		40 mm	0—15
		20 mm	0—5
2	63 mm to 40 mm		100
	Soft stones—limestone, flint, kankar, quartzite, laterite and also vitrified brick ballast, etc.	80 mm	90—100
		63 mm	35—70
		50 mm	0—15
		40 mm	0—5
		20 mm	
3	50 mm to 20 mm		100
	Hard stone-granite, trap, basalt, diorite etc.	63 mm	95—100
		50 mm	35—70
		40 mm	0—10
		20 mm	0—5
		10 mm	

Grading No. 1 is more suitable for sub-base courses, but it is not tenable for a compacted layer thickness of less than 90 mm.

For crushable type of aggregates like brick metal, kankar and laterite or other aggregates which get crushed excessively under the roller, the above suggested gradings need not be strictly enforced but the material should generally be within the size range indicated.

Other Suggested Gradings of Road Metal for Water-Bound Macadam

Sizes are 'nominal size'

50 mm	50%	50 mm	60%	50 mm	60%
40 mm	25%	20 mm	30%	40 mm	30%
20 mm	15%	6 mm	10%	20 mm	10%
12 mm to dust	10%				
63 mm to 32 mm	50%	63 mm to 32 mm	50%	40 mm	60%
32 mm to dust	50%	32 mm to 12 mm	30%	25 mm	30%
		12 mm to dust	20%	12 mm	10%

The metal should be screened to remove fines from 12 mm size to dust where coarse aggregate of size 63 mm to 40 mm is to be used, and 10 mm size to dust where coarse aggregate of size 50 mm to 25 mm is to be used.

There is no relative direct proportion between loose stack and consolidated thickness as it depends upon the amount of compaction size and hardness of the stones. Loose crushed stone is compacted about 20 per cent when rolled and will then have about 30 per cent voids. These must be filled in to ensure stability of the layer.

Size of road metal. The maximum size of aggregate to be used in any particular case would depend upon the type of aggregate available, thickness of the layer to be consolidated and the weight of the rollers. A general rule is that the maximum size may not exceed three-quarters of the total consolidated thickness of any one layer of construction.

Some engineers recommend that the maximum size of coarse aggregate should be equal to the consolidated thickness of each layer. This provides stability because the larger stones engage the base course and the surface has no tendency to 'roll' or 'wave'.

The following sizes (nominal) of stone metal are usually recommended:

(i) 63 mm to 40 mm where soft stones like limestone, flint, kankar, laterite and also vitrified brick ballast (see "Use of Brick Metal" under "Selection of Stone Metal") are to be used.

(ii) 50 mm to 20 mm where harder varieties of stones like granite, trap, basalt, diorite, etc., are to be used.

The harder the material, the smaller must it be broken. Coarse grained rocks are not suitable for use in small sizes. The larger the individual sizes of the aggregate, the greater will be the stability and strength, because large aggregate have fewer voids than small aggregate. The larger the size of stone, the more difficult it is to spread it uniformly. Larger size of metal tends to produce a rougher surface than smaller size, but it bears iron-tyred wheel traffic better.

Spreading of Stone Metal. The road metal should be spread uniformly and evenly over the prepared base to the required thickness. The stone aggregate is raked off the stacks or stockpiles along the side of the road-way or directly from vehicles with rakes so as to leave behind fines and dust, bigger size being placed at the bottom. In no case should the meta be dumped in heaps directly on the surface prepared to receive the meta nor should hauling over uncompacted or partially compacted base b permitted.

The metal should be spread uniformly to proper file and camber b using templates across the road about 6 to 8 metres apart. The botton member of the template should be of a depth equal to the unrolled thick ness desired of the metal. All high or low spots should be remedied b removing or adding metal as required. There should be no segregatio of large or fine particles. The road metal should not normally be sprea more than 3 days in advance of the subsequent construction operations.

Consolidation of Road Metal — Rolling. Rolling is done wit three wheeled power roller of 6 to 8-tonne capacity or tandem or vibrator roller. The weight of the roller depending upon the size and hardness of t stone metal and the thickness of the layer to be consolidated. (See und "Rolling") Rolling should begin longitudinally at the edges with roll running forward and backward to lock the stones firmly at the edges an until the edges have been firmly compacted. The roller should th gradually progress inwards parallel to the centre line of the road ov lapping preceding tracks by at least one-half width and should contin until the entire area of the course has been rolled by the rear whe Where no kerbs exist, one-half the width of the rear roller wheel shou overlap the shoulder sufficient times to compact the shoulder firmly against the pavement.

Excessive dry rolling should be avoided as it is apt to cause corrugations in the suface. Rolling should be discontinued when the aggregates are partially compacted with sufficient void space in them to permit application of screenings. However, where screenings are not to be applied, as in the case of crushed aggregates like brick metal, laterite and kankar, compaction should be continued until the aggregates are thoroughly keyed. During rolling, slight sprinkling of water may be done, if necessary. Rolling should not be done when the subgrade is soft or yielding or when it causes a wave-like motion in the subgrade or sub-base course. When water-bound macadam is to be laid on a subgrade composed of clay, fine sand or other soils that may be forced up into the coarse aggregate during the rolling operations, an insulation layer of granular material should be provided for blanketing the sub-grade. (See under "Construction of Bases and Sub-Bases")

The metal is rolled dry until well compacted and there is no appreciable movement (in the metal) when walked upon, or no appreciable wave or creeping of the stone in front of an advancing roller, and no lines of roller are left on the surface. Till this is done, no blindage material is to be put on.

During the process of rolling camber and grade of the surface should be checked from time to time transversely and longitudinally with templates and any irregularities corrected by loosening the surface by hand picking or raking tools, surplus material removed from high spots and depressions filled in and surface rolled again. Screenings should not be added to make up the depressions.

No fresh metal should be added once dry consolidation has been started as it is merely crushed and serves no useful purpose. If new metal must be added once consolidation starts, the part of the pavement must be fully raked up so that the new metal is thoroughly incorporated into the body of the pavement.

When rolling a surface in two halves, a strip of about 25 to 30 cm along the centre of the road should be left unrolled while consolidating the first half. This should be properly jointed while the metal is being spread on the second half and consolidated with it. This obviates the occurrence of a continuous longitudinal furrow along the ridge of the road.

No rolling should be done where signs of metal crushing are noticed or rolling causes wavelike motion in the base course of the subgrade. Over-rolling should not be done. About 20 to 30 strips of the roller would be sufficient.

A rough test for finding if the consolidation has been fully done is: A piece of about 25 mm stone is put on the consolidated surface and roller passed over it, it will be driven in if the consolidation is incomplete. Or, a fully loaded bullock cart (not of the heaviest type with iron-tyred wheels) going over it makes no impression.

In an ordinary water-bound macadam road where the metal is all more or less of one size, consolidation is obtained by chips broken off

by intensive rolling which cement the stones together. But such an aggregate does not interlock properly and large and numerous voids are left. Road tends to disintegrate rapidly under traffic though the surface may look all right at first. If the metal is properly graded the particles interlock by wedging with small size stones and form a well bonded surface. A thicker coat of metal is also necessary since consolidation of a thin coat cannot be effected. But, for the road which is to subsequently grouted, (with bitumen or tar) one size of metal is preferable to enable the binder to penetrate.

Where water is unobtainable for consolidation, a temporary road can be made with lime-stone (and possibly with other kinds of metal that have good binding properties) by dry rolling only, then speading a thick layer of shale or bajri, and leaving the traffic to do the rest.

For joints across the road width, end of each layer should be given a flat slope and well consolidated together so as to avoid formation of bumps.

Some engineers recommend that the first course should be consolidated and thrown open to traffic for a period of 2 to 6 weeks depending upon the intensity of traffic. The surface of the first course should then be scarified, undulations filled up and the second course then laid according to profile and consolidated.

Templates. It is absolutely essential to use properly made full width gauges or templates fitted with a central plummet and both edges fixed with it. The depth of the plank forming the gauge should be the thickness of the stone metal layer so that when the metal has been properly spread the gauges are buried just flush with the surface. The intermediate work is then easily tested with a cord stretched between the gauges. Three templates should be provided and used with a distance of about 6 to 8 metres between each. A spirit level should invariably be used with the template to ensure that the edges of the metalling are truly level.

The camber of template should not be flatter than 1 in 72 for roads to be carpeted or concreted. 1 in 48 if the road is to be painted and 1 in 36 if it is to remain water-bound.

Shoulder supports are made to give lateral support to the road metal and to prevent it from spreading while rolling. Two parallel small mud walls or bunds of clay puddle, or bajri or metal mixed with clay, 20 cm wide and of height equal to the unconsolidated thickness of the metal, with a clear distance between them equal to the width to be metalled. Where bricks are used for soling, these can be laid either to project 15 cm beyond the edge of the metalling and clay walls made for support of the metal or, bricks can be placed instead upright on end to project from the face of the soling so as to act as a kerb. Berms of earth 30 cm wide should be made to act as backing for the bunds. (Alsd see under "Crushed Cement Concrete Base/Sub-Base Course")

Sometime extra thickness is provided at the edges of the road (called haunches) for a width of about 45 cm and a depth of 15 cm below the bottom of the soling with extra stones to strengthen the edges which are the weakest part of a road.

Screenings

Screenings are used to fill interstices in the coarse aggregate and forming a smooth riding surface. As far as possible the screenings should be of the same material as the coarse aggregate. Brick kiln rubbish, limestone dust, red bajri or moorum, kankar nodules and gravel (other than river borne rounded aggregate) can also be used instead. Red gravely moorum and kankar have the advantage of holding the metal if they penetrate in the voids between metal pieces.

When the first dry rolling has been completed and the metal has been thoroughly set, screening is spread over the surface and brushed backwards and forwards to fill in the interstices completely. The screenings should not be dumped in piles over the metal but should be spread uniformly and evenly with a twisting motion from baskets or hand shovels to avoid segregation, and should be applied at a slow rate in three or more applications as necessary. In no case should the screenings be applied so fast and thick as to form cakes or ridges on the surface making the filling of voids difficult or preventing the direct bearing of roller on the coarse aggregate. Spreading and brushing of screenings should proceed simultaneously so that local excesses or deficiencies of material can be corrected at once. Dry rolling and brooming should be contiuued while the screenings are being spread so that the jarring effect of the roller will cause the screenings to settle into the voids of the metal (coarse aggregate). The spreading, rolling and brooming of screenings should be taken up on sections which can be completed within one day's operation. Damp and wet screenings should not be used.

After speading the screenings, the surface should be copiously sprinkled with water, swept and rolled. Hand brooms should be used to sweep the wet screenings into the voids, and to distribute them evenly. The sprinkling, sweeping and rolling should be continued and additional screenings applied where necessary until the coarse aggregate is well bonded and firmly set for its entire depth, and until a grout has been formed of screenings and water that will fill all voids and form a wave of grout ahead of the wheels of the roller. Care should be taken to see that the base of subgrade does not get damaged due to the addition of excessive quantities of water during the process.

After the final compaction of the water-bound macadam course, the road should be allowed to dry overnight. Next morning hungry spots should be filled with screenings of binding materials and when the surface is still damp, a layer of sand or earth about 6 mm thick, should be spread on the surface, lighly sprinkled with water, if necessary, and rolled. No traffic should be allowed till the macadam has set.

Where tracks are formed by the traffic on the new road, barriers should be put on such tracks, which may consist of tree branches, to divert the traffic.

Binding Material

Binding material is required to prevent ravelling of water-bound macadam and consists of fine grained matter.

It is occasionally necessary to use metal which without the addition of some kind of binding meterial will not consolidate at all. If the consolidation of such metal is carefully watched, it will be noticed that after a comparatively small amount of rolling, either wet or dry, the metal begins to move in waves in front of the roller, and the longer rolling is continued the more unstable it becomes. It would appear that the edges of stone grind off into a fine sand which acts as a lubricant. It is consequently necessary with such metal to use a layer of the best binding material available, which must be spread at the comparatively early stage, when further rolling produces no improvement. It should be noted that the binding material is not mixed with the metal before rolling is commenced.

Application of Binding Material

After the application of screenings, the binding material, where it is required to be used, should be applied at a uniform and slow rate in two or more successive thin layers. After each application of the binding material, the surface should be copiously sprinkled with water and the resulting slurry swept in with hand brooms so as to fill the voids properly. This should be followed by rolling with a 6 to 10—tonne roller, during which water should be applied to the wheels of the roller to wash down the binding material that may get stuck to them. The spreading of binding material, sprinkling of water, sweeping with brooms and rolling should continue until the slurry of the binding material and water forms a wave ahead of the wheels of the moving roller.

A final topping of sand while the surface is damp after the moorum blindage will give good results.

Hungry spots should be filled with screenings or binding material, lightly sprinkled with water if necessary, and rolled.

Binding material should not be spread in thick layers, especially if the clay content is high. A thick layer of sticky material lifts up metal under rolling. The blinding material should be free from any admixtures of clay or dust. If the coarse aggregates (stone) is completely devoid of binding properties and no small stuff is obtained from scarifying, earth may be used (when no other binding material is available) up to 5 per cent of the stone consolidated for binding, but it is not a sound practice. Black soil should not be used for blindage.

The quantity of blindage on WBM roads is a layer about 6 to 12 mm. IRC, however, recommend as follows:

The quantity of binding material will depend on the type of screenings and the function of the road. Generally the quantity required for 75 mm compacted thickness will be 0.06 to 0.09 cu. metres/10 sq. metres in the case of WBM sub-base or base course, and 0.10 to 0.15 cu. metres/10 sq. metrés when the WBM is to function as a surfacing course. For 100 mm thickness, the quantity needed respectively will be 0.08 to 0.10 cu. metre and 0.12 to 0.16 cu. metre per 10 sq. metres of road surface.

Application of binding material may not be necessary where the screenings consist of crushable type material like moorum, laterite or soft gravel.

When a surface is to be painted, no material should be used for blindage which is of such a nature that it cakes hard and is, therefore, difficult to remove. Many moorums must be avoided for this reason.

GRADING REQUIREMENTS OF SCREENINGS FOR WATER-BOUND MACADAM

IRC: 19-1972

Grading/ Classification	Size of screenings	Sieve designation	Per cent by weight passing the sieve
A	12.5 mm	12.5 mm	100
		10.0 mm	90-100
		*4.75 mm	10-30
		*150 micron	0-8
B	10.0 mm	10.0 mm	100
		4.75 mm	85-100
		150 micron	10-30

*4.75 mm is No. 480 (ISI) and 150 micron is No. 15 (ISI). In case of soft aggregates such as kanker, laterite, the use of screenings may be eliminated.

Screenings of Type A in the table should be used in conjunction with coarse aggregates of grading No. 1, and of Type B with coarse aggregates of grading No. 3. With coarse aggregates of grading No. 2, either Type A or Type B screenings may be used.

The use of screenings may be dispensed with in the case of crushable type soft coarse aggregates such as brick metal, kankar and laterite.

The gradings and the quantities of screenings required are given in the table at the next page.

Size of Materials for Test Sieve-Square Holes

Size of Opening	Wire cloth Wire dia. SWG No.	Perforated plates, Plate thickness BG No.
75 mm	0	12
63 mm	2	14
50 mm	4	16
40 mm	6	16
32 mm	6	16
20 mm	8	16
12.5 mm	10	16
10 mm	12	18
6.3 mm	14	18
2.3 mm	15	20

If perforated sheet having circular holes is used for screens, the diameter of the holes shall be $1^1/_8$ times the side of the squares specified.

Screens shall not be set at a slope steeper than 45 deg. to the horizontal.

Approximate Quantities of Coarse Aggregates and Screenings Required for 10 mm Compacted Thickness of WBM Sub-Base Course for 10 sq. metres

Coarse Aggregates			Screenings		
Classification	Size range	Loose quantity	Stone screenings		Crushable type such as moorum or gravel
			Grading classification	Loose quantity	Loose quantity
Grading I	90 to 40 mm	1.21 to 1.43 cu.m	Type A 12.5 mm	0.40 to 0.44 cu. m	0.44 to 0.47 cu. m

Approximate Quantities of Coarse Aggregates and Screenings Required for 75 mm Compacted Thickness of WBM Base Course/ Surfacing Course for 10 sq. metres

Course Aggregates			Screenings			
Classification	Size range	Loose quantity	Stone screenings			Crushable type such as moorum or gravel
			Grading classification or size	For WBM base course	For WBM surfacing course	All cases
Grading II	63 to 40	0.91 to 1.07	Type A 12.5 mm	0.18 to 0.21	0.14 to 0.17	0.33 to 0.35
-do-	-do-	-do-	Type B 9 mm	0.30 to 0.33	0.24 to 0.26	
Grading III	50 to 20 mm	-do- cu. m	-do-	0.27 to 0.30 cu. m	0.22 to 0.24 cu. m	-do- -do- cu. m

Use of Low Grade Aggregate in Road Pavement Construction

Where hard aggregates are not available within economical loads, locally available low grade (somewhat soft but not very soft) aggregates such as — Laterite, Kankar, Shale, Moorum, Soft Gravel, Dhandla, Brick aggregate, cr other soft stones may be used. These may be available atsite in the form of soil-aggregate mixtures like soil-gravel, or may be blended with soil with suitable aggregate gradings.

The aggregates should be reasonably well graded so as to achieve a dense and well interlocked mass. A few typical approximate gradings are given in the table following for general application.

Suitability of any aggregates for use in pavement courses should be judged on the basis of CBR values. (See under "Design of Pavements" in the pages following). CBR should desirably be not less than 20 for use as sub-base. In the case of base courses, the acceptable value of CBR for heavily trafficked routes is normally 80, but a somewhat lower value could be permitted for arid areas or light volume roads. Whether moorum is used as sub-base for high class roads, or as surfacing for lightly trafficked roads, the soacked CBR value should not be less than 20. (IRC recommendations). Where this requirement is not satisfied, or a still higher strength is desired, this could be achieved through cement or lime stabilisation. (See under "Gravel, Kankar, Brick and Soil Stabilised Roads" in the pages following).

One of the significant characteristics of most types of low grade aggregates is loss of mechanical strength upon wetting. It should also be noted that aggregates can sometimes be of a very variable quality. Therefore, for evaluating the suitablity of a particular type of aggregate mere nomenclature should not be the guide for selection but their actual physical characteristics should be studied.

Low grade aggregates can be used for sub-base or base courses of road pavements or even sometimes as surfacing. The suitability of aggregates, except for material like moorum, should be based on wet aggregate strength.

Minimum thickness for any courses should be 10-15 cm, except in the case of moorum, when it should be 15 cm.

Moorum or Red Bajri for road work should be obtained from pits of weather disintegrated rocks. It should preferably contain silicious material and natural mixture of clay of calcarious origin. The size of moorum should not be more than 20 mm.

Laterite for use as soling or metalling, should be hard, compact, heavy and of dark colour. Light coloured sandy laterite and that containing much ochreous clay should not be used.

Kankar should be tough, having a blue almost opalescent fracture. It should not contain any clay in the cavities between nodules. (Also see under "Kankar Roads" and "Foundations Under Concrete Pavements — Cement Concrete Reads".)

Crushed Slag should be manufactured from air-cooled blast furnace slag. It should be of angular shape, reasonably uniform in quality and density, and generally free from any thin, elongated and soft pieces, dirt

or other objectionable matter. Crushed slag should not weigh less than 1120 kg per cu. metre, and the percentage of glassy material in it should not be in excess of 20. Water absorption of slag should not exceed 10 per cent.

Typical Grading Limits for Soil-Aggregate Mixtures

Sieve designation	Nominal Maximum Size of Material — Per cent by weight passing the sieve: 80 mm	40 mm	20 mm	10 mm	5 mm
80 mm	100	—	—		
40 mm	80—100	100	—		
20 mm	60-80	80—100	100	—	
10 mm	45—65	55—80	80—100	100	—
4.75 mm	30—50	40—60	50—75	80—100	100
2.36 mm	—	30—50	35—60	50—80	80—100
1.18 mm	—	—	—	40—65	50—80
600 micron	10—30	15—30	15—35	—	30—60
300 micron	—	—	—	20—40	20—45
75 micron	5—15	5—15	5—15	10—25	10—25
IRC: 63-1976	Suitable for Base Courses			Suitable both for Base and Surfacing	

Not less than 10% should be retained between each pair of successive sieves specified for use except for the larger pair.

Use of Brick Aggregate

Where stone metal is not economically available, brick ballast of the following specifications may be used:—

A brick metal water-bound macadam road surfaced with hard stone chips can stand a traffic of about 800 to 1000 tonnes per day. Painting considerably increases its strength of crushing under iron-tyred traffic. Brick metal when soaked in High Viscosity tar and placed over a bed of tar mixed sand with a seal coat of tar and sand, can withstand fairly heavy traffic, and can be used for carpets up to about 5 cm thick. The brick metal is soaked in hot tar for about 3 hours before laying. It has been indicated that there is an overall increase in crushing strength on all types of brick metal after soaking in road tar.

It has been observed that overburnt brick-metal absorb about 4 per cent, Ist class about 17 per cent, and 2nd class about 27 per cent, of their weight of tar. The increase in crushing strength over untreated metal has been noted to be about 18 per cent, 28 per cent, and 48 per cent, respectively. Tests carried out show that brick ballast has better affinity to asphalt than tar. Adhesion of binder to moist brick ballast is not satisfactory.

The bricks should preferably be slightly over-burnt or thoroughly well-burnt, dense and deep red in colour. Spongy or vitrified material, as a result of excessive overburning, is useless and should be rejected. The size of brick ballast should be from 63 to 40 mm, well graded for water-bound macadams and down to 6 mm for carpets.

For a water-bound macadam road a minimum thickness of 11.5 cm of brick metal should be consolidated over a kankar or brick soling. The roller used should not exceed 5 tonnes weight. Brick ballast should not be dry rolled. (Some engineers, however, recommend dry rolling). Sufficient hoggin should be left over the sub-grade to enable it to work up and fill the interstices of brick ballast for some depth. After consolidation, the brick ballast surface should be kept constantly blinded with earth or sand till it dries up and the first coat of painting is done, and no traffic should be allowed on the road in the meantime. The first coat of painting should be given soon after consolidation and when the road has dried. The road surface should be thoroughly cleaned and brushed and surfacing done as usual. A two-coat work is recommended. Brick ballast has a tendency to absorb some binder, therefore, an excess of about 10 per cent of binder and grit are required than needed for a stone metal surface. The grit for each coat should be spread in two layers and each layer should be rolled separately. Hard stone chips should only be used.

This type of road is not suitable for heavy bullock cart traffic, but is very economical for built-up areas with light traffic.

The following quantities of material are required for a 5-cm thick brick metal carpet:—

Brick metal (25 to 40 mm size)	4.8 cu.m for 100 sq.m
Tar for brick metal	96 kg/cu.m of metal
Sand (dry, medium coarse)	2.4 cu.m for 100 sq.m
Tar for sand (High Viscosity tar is used)	80 kg/cu.m of sand

(Most of the above information is based on the results of experiments carried out as reported in IRC Technical Papers Nos. 154 and 172.)

13. MAINTENANCE AND REPAIR OF ROADS

(Causes of Disintegration of Roads in India are given at page 18/17)

It is very important that preventive maintenance should be carried out at the first indication of failure, and immediately it becomes necessary. Delay will cause minor defects to develop into major ones as a result of traffic action or weather.

The failure of a road structure may be due to:—

(a) Inadequate drainage or poor maintenance of the drainage system; (b) Bad design, i.e., insufficient thickness of construction to carry the traffic load imposed; (c) Faulty construction, of which the commonest example is inadequate compaction of the subgrade and base; (d) Poor surface maintenance; (e) Open textured road surfaces are more liable to fail than close textured surfaces.

The source of trouble should always be determined before making repairs. Surface repairs on a defective base or subgrade is a waste. The condition of the road base should always be investigated. Undue strengthening of weak spots may create differences in traffic wear and impact which cause additional damage to the adjoining surface areas.

Every effort must be made to reduce interference with traffic with good organization of repair work. Warning signs and barriers should be carefully sited. Maintenance materials should be kept available in small dumps along the roadside, to be drawn on as required by maintenance road gangs.

Potholes and Ruts. Unless potholes and ruts are repaired atonce they will quickly cause further damage to the surface. The disintegration of large areas is usually due to failure to repair potholes promptly. Deep potholes are usually the result of poor drainage, or soft pockets in the sub-grade. The road, in this case, should be dug out down to the solid sub-grade and the fault made good with well-graded aggregate. The top should be bound with screenings, watered and rolled.

In the case of bituminous surfaces potholes usually form only when the binder has lost its adhesive properties for some reason with the result that the aggregate is displaced by traffic action. (See under "Patch Repairs".)

Corrugations on a bituminous road surface is usually due to inadequate compaction when it was laid, and to an excess of binder. It is best put right by scarifying the surface as explained under potholes.

Re-metalling

Scoring or Picking is roughening the old road surface with pick axes prior to spreading the new metal.

Scarifying is loosening the old metalled surface by digging up to a depth of from 4 cm to 7.5 cm preparatory to re-metalling. Where the new metal added is less than 7.5 cm the work is sometimes termed *re-sectioning.*

All dirt, dust, cacked up mud, slush, animal droppings, vegetation and all other rubbish should be removed from the water-bound surface before scarifying.

Scarifying is a cheap method of resurfacing old roads, but should not be attempted unless the average thickness of metal over the soling is at least 10 cm. The life of a remetalled road is only about 3/4th of the life of a road given full new coat.

The old surface is scarified to a depth of about 4 cm for hard stones and up to 6.3 cm to 7.5 cm for soft stones. Scarifying is done either by hand picking or by power (independent scarifier or tynes fixed to the roller). The loosened metal is raked over to bring the metal from 20 mm gauge upwards to the surface. (The rakes should have prongs giving 25 mm clear between each prong.) New metal of 40 mm gauge is added at the top of the old to make a total thickness of at least 11.5 cm, and formed to required shape and then consolidated and surface dressed. Fresh metal of the same kind only should be mixed with the old metal.

Where there is a thin crust of hard metal on the existing road and it is proposed to lay the new metalling over the old without scarifying, the old surface should be scored for the full width of the road with diagonal (criss-cross) lines 40 mm deep and 30 cm apart for thin coats and 40 cm apart for thick coats of metal to provide the necessary key. The lines should be at 45 deg. to the centre line of the road. Where, however, the existing road is of soft metal or brick ballast, V-shaped trenches should be made, about 80 mm wide and 5.. mm deep, at intervals of 60 cm, instead of the "scored lines" for the key.

The main object of scarifying is to obtain proper binding between the new layer and the original surface, therefore, the original surface should not be disturbed for more than what is necessary.

Where the scarifying is done for the full depth of the metal, all the scarified material is removed away from the road surface and screened. Chippings from 3 mm to 20 mm gauge and mixture of dust and fine grit are sifted separately. The chipping are spread over the subgrade, adding more earth or such matter if necessary, to form a thick cushion under the metal. The foundation thus prepared is thoroughly watered (but not rolled) and the old metal (big size) is spread over it and combed through to bring all the big stuff on top. New metal 50 mm to 63 mm gauge is added on the top and consolidated. The mixture of dust and fine grit is used for final grouting of the surface after consolidation. Some engineers prefer to pick and loose the old water-bound surface before spreading the new metal to provide a bond between the old surface and the new surface.

The foundation is loosened by means of pick axes and dressed to definite and uniform cross section, It will be necessary to strengthen the edges. which may be done by digging shallow trenches 23 cm wide along side of the formation and filling them with new metal,

Before deciding how much new metal is required, the thickness of the existing metal should be ascertained. As a general guide for ordering metal the following figures may be taken:

Thickness of existing coat	Amount of new metal required for a 3.8 metre wide road, per kilometre
80 mm and under	210 cu.m
100 mm	165 "
125 mm	130 "
150 mm	90 "

Owing to the metal being graded and due to a large percentage of old stuff being re-used, the surface is stabilized much sooner. The wet earth underneath also tends to help in this respect. For wet rolling the water should be gradually added so that complete mechanical locking of the metal can occur before part of the earth cushion underneath begins to be forced to the surface through the interstices. Interstices should however, be filled by earth squeezed up from below. When consolidation has been completed the fine material from screening is spread over the surface and thoroughly washed in with copious water and allowed to stand for 24 hours. When it has partially dried a final light rolling may be given. Traffic should be kept off the road for 2 days and road watered for 7 days after rolling

An essential part of the specification is that first coat of surface dressing should be put not sooner than 14 days and when the road has thoroughly dried, all the fine stuff must be cleaned out for a depth of 10 mm before the application of surface dressing.

14. USE OF ASPHALTIC BITUMENS AND ROAD TARS

Commercial names and classifications of these materials are subject to periodic change as new types are developed and older ones discarded. Latest specifications should be obtained from the manufacturers.

Both bitumen and tar are dark brown to black in colour. Bitumen is a petroleum product whereas tar is produced by the destructive distillation of coal. (See under "Glossary of Highway Engineering Terms.")

Choice of Material—These binders are manufactured in many different grades of consistency, from very fluid that pour at atmospheric temperatures to hard materials. The road engineer is presented with a somewhat bewildering number of alternatives. The choice of a particular type of surfacing material is determined by economic as well as by technical considerations. There are always movements in the road surface due to temperature variations and traffic forces; if the binder hardens through exposure to weather or for other reasons, it becomes brittle at low temperatures, high stresses are developed when movements occur, and the bond between adjacent stones can then be easily destroyed. To reduce the effect of hardening, as soft a grade of bitumen as practicable should be used, without danger of the surface "pushing" or "waving".

Road surfacings vary widely in their texture; in the more open-textured type the tar or bitumen acts as an adhesive between stone and road or between stone and stone, and in the dense close-textured type the tar or bitumen acts in combination with the fine aggregate as a plastic mortar. These binders are required to remain in a plastic candition so that the surfacing can accommodate itself without cracking to small movements induced in the road by temperature variations, moisture changes and traffic. Binders must also be sufficiently viscous and resilient possessing shock absorbing qualities to resist the forces of traffic (impact). Adhesion and viscosity are thus the important physical properties of road tars and bitumens.

Tar and bitumen are not suited to damp or water logged areas as regards road construction works. Both road tar and bitumen are adversely affected by the action of weather.

Tests and Suitability of Binders: (General Properties)

Penetration—Is a measure of hardness or consistency and therefore of primary importance as an indication of suitability for any particular construction or condition. This test is probably the most important of all tests on bitumens. Penetration is measured by a standard Penetrometer. Penetration is the distance measured in units of 1/10 mm that a standard blunt-point needle will penetrate a sample at 25 deg. C. when the needle is loaded with 100 grams applied for 5 seconds.

The higher the penetration number the softer the bitumen. The lower the penetration, the higher will be the melting point and lower the ductility; in other words, the harder the bitumen the higher will be the melting point, but the ductility will be less, for the bitumen becomes more brittle the harder it is. The harder the bitumen the quicker it sets; a soft bitumen takes comparatively a longer time to set.

Normally bitumens with greater penetration are used for surface painting and semi-grouting work, and bitumens with lesser penetration are used for full grouting work.

Properties of Road Tars

There are 5 grades of road tars designated RT-1 to RT-5 according to their viscosities. The viscosity increases with grade number. Their use is as follows:

RT-1. Light grade (penetrating oil) for surface painting and under exceptionally cold weather conditions and for use on hill roads at high elevations. Is applied cold.

RT-2. For surface painting under normal Indian climatic conditions. To be heated to temperatures 93 to 104 deg. C.

RT-3 (i) For surface painting and renewal coats and (ii) For pre-mixing chips for top course and light carpets. To be heated to temperatures 104 to 116 deg. C.

RT-4. For pre-mixing tar macadam (base course). Heavy grade. To be heated to temperatures 132 deg. C.

RT-5. For grouting. To be heated to temperatures 138 deg. C.

(*Also see* under "Viscosity").

Bitumen Cutbacks

(*Also see* under "Glossary of Highway Engineering Terms")

Cutbacks are softened bitumens (solid bitumen thinned with a volatile distillate such as petrol, kerosene, diesel oil or tar oil), which can be used without heating or require only light heating. Bitumen cutbacks are classified into three types:

Rapid-curing (RC)—Light type for use as primer.
Medium-curing (MC)—Medium type for surface-dressing.
Slow-curing (SC)—Heavy type for pre-mixing.
(The term 'setting' is also used instead of 'curing').

Each of these three types of cutbacks are produced in six grades, numbered from 0 to 5 (designated as RC-0, RC-3, MC-3 or SC-5, etc.) according to their viscosities or fluidity, the percentage of bitumen increasing as the numbers rise. For example, SC-0 contains about 40 per cent bitumen, whilst SC-5 contains about 80 per cent bitumen. (RC-1, MC-1, SC-1 are all of the same viscosity.)

Numbers 0 and 1 are of very low consistency and are fluid at ordinary temperatures and flow like oil. Numbers 2 and 3 are thicker and flow slowly at ordinary room temperatures. Numbers 4 and 5 are thick and sticky, almost semi-solid, and poured with difficulty until heated.

A cutback bitumen is used for cold mix specifications as it will coat cold chippings in drum mixers. Unlike a straight run bitumen, a cutback does not harden too rapidly on contact with cold aggregate.

Field Test for Type of Cutback

Rapid-curing: A thin layer spread on an even surface will become a

sticky substance in about 10 to 15 minutes which adheres to the fingers. Medium and slow curing cutbacks will take hours or even days to become tacky.

Medium-curing cutback, when heated, gives off an odour of kerosene. Slow-curing cutback, when heated, produces an odour of hot lubricating oil.

Road Oils

These are cutback bitumens with a high percentage of diesel or heavy-type oil. They are used for waterproofing earth roads, and for dust control.

Bitumen Emulsions

(*Also see* under "Glossary of Highway Engineering Terms")

Bitumen emulsified in water is called "bitumen emulsion". Bitumen blended with petroleum distillate is known as "cutback bitumen."

Bitumen emulsions are chocolate-brown in colour. If mixed with petrol, kerosene or diesel oil, globules of water will appear. Emulsions have the advantage that they can be applied cold and can be used on damp surfaces and wet aggregate although they are not always successful if laid in wet weather. (Cutbacks can be used when the road surface and also the aggregate are dry.) Emulsions are not so competent, especially under heavy traffic, as hot bitmen or tar, and should be used with discretion. They are not also cheap in real cost as about half the volume is water. (Emulsion contain water cutbacks contain solvent.)

There are three grades of emulsions—(i) Quick-breaking or Rapid-setting (RS), (ii) Medium-breaking or Medium-setting (MS) and, (iii) Slow-breaking or Slow-setting (SS). (The terms 'curing' or 'cutting' are also used instead of 'setting' and 'breaking'). The time taken for "breaking" of an emulsion is the measure of its grade and the type of work for which to be used.

In order to decide whether the sample is suitable for surface-dressing or grouting, for coated macadam, or for soil stabilization, it is necessary to know whether it is fast-breaking or slow-breaking. The breaking rate can be assessed as follows:

Put some clean gravel or crushed stone into a pan and try to mix the emulsion with it, using a metal spoon. A fast-breaking emulsion cannot be mixed in this way, as it breaks immediately, gumming up the spoon with bitumen. While, a slow-breaking emulsion will mix perfectly, coating the spoon, the pan and the aggregate.

Normally two types of bitumen emulsions are used for road work in India—(i) quick breaking and (ii) slow-breaking Quick-breaking emulsions are used for surface-painting, tack coats, seal coats, and grouting. The emulsions for such works should have minimum 50 per cent of bitumen content. Slow-breaking emulsions are used for pre-mixing and patch repairs during wet weather. This emulsion should have min: 60 per cent of bitumen content.

Bitumen emulsion is applied at atmospheric temperature but no work should be carried out in frosty weather as emulsion is liable to be adversely affected by frost. No work should be undertaken when it is actually raining.

Road Tars versus Bitumens

Different Characteristics of Bitumens and Tars

(i) The weathering properties of bituments are superior to those of tars. Generally greater deterioration is produced in tar than in bitumen when exposed to equal weather conditions. Bitumens have a better durability and resistance to weathering than tar.

(ii) Tars are more susceptible to temperature changes than are bitumens. Hot weather will soften a tar surface more than a surfacing made with bitumen of the same viscosity, and it will become more brittle at low temperatures than bitumen. In other words—tar becomes brittle in cold weather and the surface treated with tar is apt to bleed in hot weather if a little extra quantity has been used. (An addition of 10 to 20 per cen of bitumen mixed to heated tar will increase the consistency of the tar and it will thus reduce its bleeding property.) Tar is, therefore, considered unsuitable for locations with wide temperature changes. Gritting or surface dressing can be delayed a little where bitumen has been used but not with tar. (On the whole tar has been found less satisfactory than bitumem for surface coats.)

(iii) Surface dressings with bitumen are more prone to failure by water displacement than those made with tar. Tars generally adhere better than bitumens on wet aggregate.

(iv) Tars can be brought to a spraying condition at lower temperatures than those needed for bitumens; stones need not be heated to high temperatures.

(v) Setting time for tars is more than that for bitumens and this property is useful in the production of pre-coated aggregate which can be transported to large distances or kept for sometime before spreading.

(vi) Tars harden much quicker than bitumens.

(vii) Tars have higher specific gravity than bitumens and lower viscosity (or greater fluidity) and these properties give them greater penetrating power and which are more marked during summers. Higher viscosities can generally be used with tars than with bitumens—higher viscosities being used in warmer weather.

(viii) Tars produce a less slippery surface than bitumens.

(ix) Bitumens have a tendency to stay at or just near the surface resulting in a rich and fat surface.

(x) Roads built of bitumen need constant traffic to be maintained in good order, otherwise the surface will crack and reduce the life of the road.

(xi) Tars make harder surfaces (but such surfaces are brittle) than bitumens and should be preferred for roads in areas where bullock carts or other hard-tyred traffic predominates. Bitumens make more elastic surfaces and are better suited for pneumatic-traffic. Hardening of bitumens is very gradual.

(xii) Tar is more suitable for dense fine-grained surfaces and bitumen for more open surfaces.

(xiii) Volume of tar required is about 10 per cent less than that of bitumen for the same type of road work.

(xiv) Tar is cheaper than bitumen.

(xv) A primer is not generally needed with tar.

(xvi) Road tars do not dissolve in a petroleum distilate such as petrol (gasoline), kerosene, diesel oil. As such, tar carpets have proved to be good meterial for parking sites as it remains unaffected by spillage of oil and petrol from automobiles.

Bitumen and tar can also be mixed together and used.

Field Tests to Distinguish Between Bitumen and Tar

(i) The characteristic odour of bitumen is similar to that of lubricating oil, whereas tars have a peculiar pungent smell, and usually smell like creosote. (ii) Mix a sample of the material with either petrol, kerosene or diesel oil. Bitumen will dissolve to produce a uniform dark liquid. Tar, on the other hand, will not dissolve and mix will show two separate substances, the light-coloured oil, and a dark undissolved mass.

There is great difference in the temperature of the bitumen, which is about 177 deg. C. and that of the aggregate, which at atmospheric temperature is about 32 to 38 deg. C. when they are mixed. The contact of the latter chills the former and reduces its temperature, hardening it, and it is impossible to mix a hardened bitumen with any aggregate. Since it is not usually practicable to heat the aggregate, therefore, a bitumen of such a consistency is produced that would overcome this difficulty. Soft grade (slow setting) asphalts (120/150 or 180/200 penetration) can be added to hard grade asphalts (20/32 to 80/100 penetration) to retard their setting which enables them to coat cold aggregate during pre-mixing, or when painting is done under extreme cold weather conditions. But, it is not possible to produce a correct blend at site by mixing them without the skill and the technical knowledge, therefore, the specific brands of bitumen (hard or soft grade) should be ordered that would suit the type of the work and the climate.

Naphtha (about 5 to 20 per cent of asphalt) will permit coating of the cool aggregate with hot asphalt. Hydrated lime (0.5 to 1.0 per cent of total mix weight added with the fine aggregate in the mixer) will improve the coating properties of the binder and will also accelerate setting of the mix.

Heating of Binders. Bitumens and tars should not be heated to temperature higher than that specified for the particular job. The material should not be poured on to the heated surface of the boiler as it is liable to be burned. Heating must be gradual. As the boiler becomes empty the fires should be gradually withdrawn or damped so as to avoid any noticeable increase in temperature in the smaller quantity that remains. Boilers must be carefully cleaned every month.

Melting or Softening Point. This determines the temperature at which a grade of bitumen reaches a certain degree of softness or fluidity. Asphalts have no definite melting point *i.e.*, there is no temperature at which they change abruptly from solids to liquids. Test is done by the "ring and ball" method. A sample of bitumen is melted and poured into a standard brass ring placed on a plate. When the bitumen has cooled, a standardized steel ball is placed upon the bitumen and the ring is suspended in a water bath. The temperature at which the bitumen softens sufficiently to allow the ball to pass through the ring and touch the bottom of the bath is called is the melting point.

Flash point is defined as the temperature at which bitumen gives off volatile matter which can be ignited by the small head of a gas flame.

Ductility indicates the binding power and adhesiveness. Measures brittleness; if too low, the asphalt will crack. It the property which permits the permanent distortion of a material without rupture.

Viscosity of a liquid is the property that retards flow so that the higher the viscosity the slower is the flow of the liquid. This test is used with tars as a test of consistency in lieu of the "penetration" test for asphalts. It is carried out in a Viscometer. Materials which are too viscous to be tested in a viscometer, are tested by the so called "Float Test."

Coal tar and bitumen compounds though not considered normally as dangerous can become so when heated in drums, either due to local boiling of the material at the point of heating or due to the liberation of inflammable gases which catch fire and cause an explosion. Care must, therefore, be exercised to ensure that heating is gradual and spread over a wide area. If due to solidification material in closed drums has to be heated, the fire should be kept well away from the open screw cap holes.

The effective use of bitumens and road tars in any operation depends on their application at the appropriate viscosity, which differs for different operations such as spraying, mixing, pumping, compaction. The appropriate viscosity is brought about by heating the material to an appropriate temperature. As bitumens and tars supplied by manufacturers at site of work are more or less in semifluid state they have to be heated to lower their viscosities, except for certain cutbacks and emulsions which are meant for cold application.

Application temperatures for different grades of bitumens and tars are given below:

	Spraying deg. C	*Mixing deg.C.*
(a) Straight-run or Paving Bitumens		
30/40 penetration	170-185	170-185
60/70 ,,	165-180	155-165
80/100 ,,	160-175	150-165
180/200 ,,	150-165	135-150
A-90, S-90 grades	175-190	150-175
Bitumens should not be heated beyond 200 deg. C.		

(b) Cutback Bitumens	160-161	150-165
(Some cutbacks are for cold application and are used without heating)		
(c) Road Tars		
Grade RT-2	93-104	93-104
RT-3	104-116	116
RT-4	132	132
RT-5	138	138

Field test for straight-run bitumens

Grade	*Penetration No.*	*Test with pointed pencil*
Hard	25-50	Little or no penetration
Medium	50-100	Penetrates slowly but steadily
Soft	100-200	Penetrates freely

The higher the viscosity the greater is the strength of the surfacing up to a limit, when viscosity reaches some high value it imparts brittleness to the material. The viscosity of the binder should be selected according to the intensity of the traffic on the road, and also the temperature in which the work is to be carried out. Low viscosity binder can be laid cold and high viscosity binder has to be laid hot and is used in warmer weather. A more viscous binder is specified for the more heavily trafficked roads to resist distortion under the intense traffic and should be laid warm or hot within a few hours of manufacture. Where binders of relatively low viscosity are used the resulting surfacing is initially soft and may be deformed if it receives traffic immediately after being laid.

High viscosity binders are able to resist the action of water on aggregates better than low viscosity binders. Viscosity should be sufficiently low to enable it to "wet" both the road and the chippings, but on the other hand it should be sufficiently high to enable the tars to hold the stones securely against the action of traffic in the early life of the dressing. The viscosity of tar and cut-back bitumens increases rapidly with loss of oil content (by evaporation). For this reason, care must be taken to avoid over-heating or unduly prolonged heating of the tar in open or partially-open vessels, otherwise serious increase in high viscosity tars may occur. Road failures have occasionally been traced to this cause. Therefore, totally-enclosed boilers should be used.

Adhesion. The construction or maintenance of a road is primarily a problem of adhesion. Both tars and bitumens adhere well to all stones normally used as road aggregates, provided they are dry and not unduly dusty. Moisture is the chief cause of stripping and swelling. Some stones have greater surface affinity for water than for bitumen or tar, and when a coated stone comes into contact with water there is a tendency for the binder to be displaced from the surface of the stone. In a road mixture containing fines susceptible to expansion when wet, stripping is accompanied by swelling of the road mass.

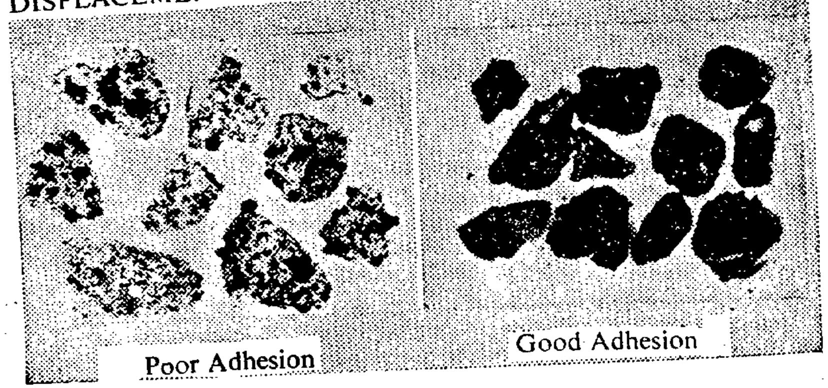
The adhesion of both bitumen and tar is weakened by the presence of water and there is a tendency for the binder to be displaced or strippeu from the stone. The higher the viscosity of the binder, the greater the resistance to this effect.

The binder in a freshly laid surface dressing or surfacing is particularly sensitive to the displacing action of water; if prolonged rain falls before the binder has reached a sufficiently high viscosity, the adhesive joint may be broken and disintegration set in. Although failures of this type are not frequent, but when they do occur there is no remedial measure; a new surfacing has to be laid. In this connection tar binders show better adhesion properties than bitumens; it is, in fact, very rare for tar macadam to fail through water displacement of the binder.

The properties of the aggregate, especially its surface texture, have an important bearing on adhesion. Smooth glassy surfaces, and both fine textured and coarsely-crystalline, generally give poor adhesion; the rough-textured stones generally give the best adhesion. There is an indication that low silica-content rocks have better resistance to stripping than high silica-content rocks of similar texture. Limestones have good adhesive properties except when crystalline.

Simple Test for Adhesion. Take a few samples of the different stones and coat them with the binder heated to the specified temperature. Let the sample of the coated stones cool for about half-an-hour. Put the different samples in glass bottles and fill the bottles with clean water. The displacement of the binder is assessed at intervals by visual inspection. the area of the stone surface exposed in each case is the displacement. Usually a little further displacement occurs after two days' immersion in water.

DISPLACEMENT OF BINDERS IN WATER IMMERSION TEST



Weight of Binders. Petroleum asphalts have specific gravities of 0.98 to 1.02, tars 1.08 to 1.24, cut-backs have generally 0.93 to 0.98, and emulsions about 1.0. In practice (in the field) it is near enough to take 1 litre to weigh 1 kilogram for all asphaltic materials made from petroleum asphalts. Tars too are about the same.

Properties of Hot Application Bitumens

Penetration	Suitable for work
10/20	Where hard material of very high melting point is required. Largely used in paint manufacture, repairing leaky roofs, roofing felts, etc.
20/30	Hard grade. Where the use of hard bitumen is indicated. Roofing: pipe jointing and coating of underground pipes; electrical insulations; damp-proofing; expansion joints in cement roads.
30/40	Medium hard grade. Standard grade for grouting in uniform size aggregate work; pre-mixing; surface painting—where a heavy application is considered necessary.
60/70	Sometimes used in lieu of 80/100 penetration in very hot localities where a slightly harder grade is required.
80/100	Standard grade for surface paintings; pre-mixing; grouting in graded aggregate dense surfaces and seal coats.
180/200	Very soft grade. For light surface dressings or in very cold climates; pre-mixed carpets.

15. CONSTRUCTION OF BITUMINOUS PAVEMENTS OR BITUMINOUS MACADAMS

The term bituminous macadam generally refers to grouted or precoated stone aggregates laid on a prepared road surface and consolidated as a base course over which a wearing course (or surface course) is superimposed.

Preparation of Base—The base on which bituminous macadam is to be laid should be prepared, shaped and conditioned to the specified grade and cross-sections as explained earlier under "Preparation of Base for Laying Bituminous Courses". The surface should be thoroughly swept and scraped clean free from dust and foreign matter. All depressions exceeding one-third the proposed finished thickness should be patched in advance.

GROUTING

Pouring a binding material in a liquid state on to a consolidated surface of road metal so that the binder penetrates the interstices until every stone is covered with the binder, is called grouting. This reinforces the structure of the road by binding the stones together and strengthens it to withstand heavy traffic. Of all the methods of road construction devised this is the simplest since it does not require any special tools and plant, nor skilled labour. The impact and shear forces set up by heavy bullock-carts and similar traffic often break up the structure of a water-bound macadam road.

When the binder is allowed to penetrate to the full depth of the consolidated stone layer, it is called Full-Grout while, when it penetrates to only half the depth or less, it is known as Semi-Grout. On most district roads in India a 75 mm consolidated and grouted thickness is quite sufficient provided there is at least 130 mm of good hard metal (including soling) in the base. In the semi-grout the consolidated metal thickness usually is about 50 mm. Semi-grout can be used on roads where the traffic is not very heavy.

Where the old road surface is scarified and new metal added, it is watered heavily while rolling so as to bring the "hoggin" about half way up the new metal. In this way the bottom layer of the metal will be firmly bound by the hoggin whilst the top metal will be bound with the binder.

Both hot and cold binders can be used for grounting. Emulsions which are more fluid than hot binders penetrate between the faces of stones better, especially in cold climates. Performance of emulsions on the other hand is much affected by the weather immediately after application; rain, in fact, may wash the emulsion away if it falls before the emulsion has "broken".

The road metal should be rolled only very lightly (with a 10-tonne roller) just sufficient to give an even surface conforming to the required camber. Ordinarily, three or four trips of the roller would be quite sufficient. A limited quantity of water may be sprinkled on during the rolling but not so much as to bring up any slurry from the bottom. A layer of stone dust or sand is spread underneath the new metal to avoid drainage of the binder below the metal to be grouted. Gravel as road metal is not suitable for grouting work.

If the old water-bound macadam surface is sound and it is proposed to grout it, the old surface should be swept clean with wire brushes so as to remove all blindage and dust, and expose the metal surface to a depth of about 20 to 25 mm. (This will not be a full-grout work). If the existing surface is unsound, it should be scarified and re-metalled as explained under "Re-metalling."

The surface of the road must be perfectly dry and clean of any dust before the application of a hot binder. Bituminous material heated to the specified temperature is uniformly sprayed over the surface longitudinally along the road, which should preferably be done under pressure. After the binder has been applied and while it is still hot or, as the spraying advances, small hard stone chippings should be spread on the road as "blindage" to fill the surface voids between the large stones. A thorough rolling should be given with a 10-tonne roller soon after blinding and when the asphalt is still hot and before it has set, so that the chips could wedge into the interstices as firmly as possible. A drag broom behind the roller (or hand brooming) greatly improves the finish.

During consolidation the surface should be tested with a straight edge for irregularities, and depressions should be filled in by painting the surface with hot binder and blinding with chippings, and high spots or projections removed by brushing. Traffic is then allowed on the road about 12 hours after final rolling.

It is very important with this type of surface to see that too much binder is not used as the extra binder has a tendency to work down towards the sides making them very "rich". and thus liable to form ruts under cart traffic and corrugations under motor traffic. The surface will also "bleed". Too much binder also results in a slippery surface.

Grouting with Emulsions

As emulsions are not suitable for heavy traffic conditions they are used only for light traffic roads and for which a 5 cm grouted thickness is considered sufficient. Where, however, hot asphalt application is not

practicable on a heavy traffic road, a greater thickness of graded metal should be made. In the case of cold emulsions the compacted surface of the road should be damped with water to aid penetration before the application of the binder. After the emulsion has been sprayed and before it "breaks", small hard stone chips should be spread evenly over the surface and lightly rolled to force the chippings into the road surface. A seal coat may be applied over this surface if necessary.

When only half width of the road is to be done at a time, the opposite half width which is in use by the traffic, should also be swept clean of dust in the same manner as the half portion which is being grouted, and kept watered while the work on the the first half width is in progress, so that the dust is not carried over to the new work. The pouring of the binder shall be done parallel to the length of the road and shall begin at the crown and work towards to the outer edge. A 15 cm strip along the crown should be left dry when the first half width is done so as to obtain a proper bond with the second half width, by repicking that when the second half width is taken in hand.

A grouted surface is considered generally less reliable than one made with pre-mixed material with graded aggregate. Grouting is not now much in favour, and is employed where equipment for pre-mix construction is not available, or in very heavy rainfall areas.

Specification for Built-up Spray Grout (Based on IRC: 47-1972)

Built-up spray grout consists of a two-layer composite construction of compacted crushed coarse aggregates with application of bituminous binder after each layer for binding, finishing with key aggregates at the top of second layer so as to yield a total thickness not exceeding 75 mm.

Grading of Aggregates

Sieve Designation	Coarse Aggregate per cent by weight	Key Aggregate passing the sieve
5o mm	100	—
25 mm	35 — 70	—
20 mm	—	100
12.5 mm	0 — 15	35 — 70
4.75 mm	—	0 — 15
2.36 mm	0 — 5	0 — 5

A Priming coat or Tack coat is given over the prepared road surface. (See under "Priming Coat", Tack Coat" and "Preparation of the Existing Road Surface") Graded aggregates are spread in two separate layers at the rate of 5.0 cu. m per 100 sq. metres per layer and rolled with either three wheel power roller of 8 to 10 tonnes or an equivalent vibratory roller Rolling should be stopped before voids in the aggregate layer are closed to such an extent as to prevent free and uniform penetration of the binder. Binder is applied over the rolled surface at the rate of 125 kg per 100 sq. metres. Second layer of coarse aggregates are applied immediately after the application of the binder, rolled, and binder applied. Immediately after the second application of the binder, key aggregates at the rate of 1.3 cu. metres per 100 sq. metres of surface are spread and rolled. Three or four trips of the roller will be sufficient.

The quantity of binder will be about 20 per cent more if tar is used and about 50 per cent more if cold emulsion is used, than straight run bitumen.

PRE-MIXING

The process of mixing road metal with a binder off the road and then placing and consolidating the mixture on the road, is called pre-mixing.

From the point of view of both stability and economy the premix method is superior to the grouting method. Premixing permits of a denser mode of construction as graded aggregate are used which are well consolidated producing a compact surface, and at the same time uses much less binder and the danger of rutting or waving under traffic due to excess of binder is avioded. Grouting method has the advantage that it does not require any costly mixers which are necessary for premixing works and the output of the work with grouting can also be much more.

A dense premixed carpet is particularly suitable for heavily trafficked roads and a consolidated thickness of 2.5 to 7.5 cm is generally prescribed according to the intensity of traffic on the road.

Only just sufficient binder should be used as any excess binder will only act as a lubricant instead of a binder and will encourage the stones to slide one on the other producing faults in the surface. But each particle must be thoroughly coated all over in addition to the binder absorbed by the pores.

Edging to give lateral support to the metal:

Before laying the premix grooves are cut along both the edges of the road, the measurements out-to-out of the grooves to be the exact final road width desired. Brick-on-end edging is provided in these grooves with the top of the edging projecting above the existing road (or soling) surface equal to the proposed consolidated thickness of the carpet to be laid.

Where carpets of small thickness are to be laid, the grooves may be lined with bricks laid flat and length-ways. Where the edging bricks need support, stone metal or shingle with about 50 per cent of clay may be rammed outside the edging for a width of about 25 cm or more. These bricks may be left in place after completion of the work; if removed, the grooves should be filled with pre-mixed metal and hand rammed. Supports can also be made of stone sets, cement concrete blocks, or timber, etc. The bricks at the edges of the road or any kerbs, should be painted with hot bitumen before mix is laid by the sides.

Aggregates for Pre-Mixing Works. The nature of the aggregates as well as their grading affect the quality of the macadam. The aggregate should consist of crushed stone, crushed gravel, hard and tough, of fairly cubical or angular shape. Gravel is not suitable for pre-mix work. Where uncrushed gravel must be used, a greater proportion of fine aggregate, not less than 30 to 35 per cent passing 3 mm sieve must be added in the mix. Graded angular aggregate should be used which will make a

dense and water-proof surface. A dense surface should be made in rainy regions. Rounded aggregate and fine sand make the surface smooth and slippery especially during rains, therefore, coarse sand should be used. When sufficient of fine aggregate is not used it requires a seal coat to make the surface water-proof. A dense carpet can also be made with large size aggregate and 50 per cent coarse sand (2:1) which will stand heavy cart traffic. Stability of bituminous mixtures increases as density of the mix increases.

The size of aggregate depends upon thickness of the layer. Large size stones contribute markedly to stability.

"For the base course, the largest size of stone shall approximate to, but not exceed, three-quarters of the final thickness of the consolidated course for asphalt carpets and two-thirds of the consolidated thickness for tar carpets. The proportion of such size shall be not less than 40 per cent, not more than 60 per cent, of the total stone content. For the wearing course or top course, the largest size of stone shall be not more than one-half nor less than one-third of the final thickness of the consolidated course and the proportion of such size shall be not less than 40 per cent, nor more than 60 per cent of the total stone content." (British Standard Specifications).

Mixing. Pre-mixing should preferably be done in a hot mix plant which may be either a batch type or a continuous one. If this is not available for small jobs, use drum mixers or concrete mixers. First mix the stone with approximately two-thirds of the total quantity of the binder required for the whole batch, add sand and finally the balance of the binder. Stone aggregates and sand should not be mixed together with binder. If mixed together it will usually be found that the larger stones are not well coated as the fine aggregates absorb most of the binder. With aggregate containing more than 10 per cent of material passing a 3 mm mesh sieve it is advisable to screen out the fine material before mixing. The fine material may then be added after the larger stones have been coated.

Mixing shall be thorough to ensure that a homogenous mixture is obtained in which all particles of the aggregates are coated uniformly. After about 15 seconds of dry mixing, the heated binder should be distributed over the aggregate at the rate specified.

For hand mixing, a barrel mixture can be made of sheet iron of size that of a tar drum in which three blades are welded inside and of the shape somewhat like a screw, and a removal lid is provided on the circumferential surface. This drum is supported on a stand about 120 cm high, and a rotating handle is fixed on one end of the drum for mixing. Mixing for small jobs can be done in the original tar drums of which a portion of the side is cut out and long bamboo sticks are fixed on both the ends to work as handle. One such tar drum barrel can be used for mixing about 2 cu. metres of aggregate per day. For small jobs mixing can be done on a cemented or sheet iron platform with shovels or spades. Gravel is best pre-mixed with shovels.

Much bigger outputs and better coating of the aggregate is obtained in hot weather and when the aggregates have been exposed to the sun.

Pre-Mixing with Emulsion—The process is the same except that the mixing should be done as quickly as possible to avoid any possibility of the emulsion "breaking" while in the mixer. Mixing is done just long enough to coat all the aggregates, and the mixer is then immediately empetied. Aggregates should be properly graded, no filler is used. Mechanical agitation is likely to cause the emulsion to "break" prematurely. Spreading should be done as soon as possible after mixing, say within 10 to 15 minutes, using forks, rakes and shovels, immediately after the tack coat of emulsion (where applied) breaks. Rolling should be done after the emulsion has broken and the water content has evaporated, usually any time up to 24 hours, depending on weather conditions.

Compaction must be limited to one or two passes of the roller, merely to attain the required shape and smoothness of riding surface, since further mechanical disturbance after "breaking" has started will only cause the binder to be stripped from the stone. If this happens the mix will never bind satisfactorily.

Over the compacted surface, coarse sand or stone dust should be spread and lightly rolled. The surface should not be opened to traffic until "breaking" is completed, usually in about 24 hours.

Pre-mixing for small jobs with emulsion may be done by filling perforated buckets with aggregate and dipping the bucket into tanks containing emulsion.

Cutbacks and emulsions are not generally used for carpets more than 2.5 to 4 cm thick.

Filler

Where no fine sand has been used in a bituminous mix, a small quantity of an inert material such as limestone dust, any stone dust (quarry dust), coal dust, slag dust, fly-ash or even cement can be added to the mix which makes the binder to flow more freely, thus giving rigidity and body to the mixture. It tends to reduce the brittleness of a mix in cold weather where the quantity of the filler can be considerably increased. The function of filler varies to some extent with the type of surfacing with which it is used. In mixes of open or medium texture, stability is increased and resistance to weathering improved. In mixes of close texture, temperature susceptibility is reduced (for dense surfacings-filler/binder mixtures have lower temperature susceptibility than straight binder of the same viscosity) and deformation minimized.

As regards the quantity of fillller, it depends upon a number of factors such as the quantity of aggregates and the binder and the type of filler. For the exact quantity required it has to be determined by field experiment. If the amount of filler is too much the mixture becomes too rigid and stiff for workability, weak and brittle which might lead to early cracking. The quantity of filler may be about 50 to 60 kg/cu. m of the aggregate if stone dust or fine sand is used, 38 to 45 kg/cu. m if fly-ash is used, 18 to 22 kg/cu m if hydrated lime is used and 25 to 30 kg/cu. m if cement is used. With limestone less of fine material is required than with granite. As large a quantity of filler may be used as the mix will carry and yet be workable.

The grading of the filler may be — The whole to pass through 600 micron (No. 30 ASTM or No. 25 BS) sieve, at least 90 per cent to pass 150 micron (No. 100 ASTM or BS) sieve, and not less than 70 per cent to pass 75 micron (No. 200 ASTM or BS) sieve — (IRC:29). Filler shall be supplied in dry state in bags or other suitable containers, and shall be protected from weather, dampness etc. When aggregates get thoroughly coated during mixing, filller is added and further mixing done until the filler is completely absorbed. No filler is needed where emulsion is used as binder. Filler is also required with "sand carpets."

Indication of Mix Defects

Some of the possible defects in the asphaltic concrete mix are:—

(1) Excess bitumen—is usually indicated by a sloppy mix which slumps too readily even in the lorry.

(2) Lack of bitumen—such mixes have a dry lean appearance lacking the shining black lustre of a well-coated mix. They may be difficult to spread, and transverse cracks may appear under the roller which do not readily heal up.

(3) Over-heated mix—excess of blue smoke from the mix usually indicates over-heating and its temperature should be checked prior to laying. After laying, such a mix can often by detected by a brown, dull burnt surface appearance.

(4) Mix too cold—such mixes are usually stiff and difficult to handle; the large particles may also not be properly coated. Spreading will be difficult with a tendency for the mix to tear under the screed.

(5) Excess moisture—mixes with too much moisture often have an appearance similar to those with excess bitumen in that they are sloppy and tend to slump in the lorry. The fact that moisture is the cause of the trouble can be detected by steam rising from the mix.

(6) Poor mixing—this will show up, particularly when the mix is laid. Some areas will appear lean and brown and others shiny and black (where there is excess bitumen).

(7) Poor grading—if there is excess of coarse aggregate, the mix will appear rich, though workability will be poor, and the appearance when laid will be rough open-textured. Excess of fine aggregate on the other hand will show up by the mix appearing lean and too finely graded when laid.

ROAD CARPETS

Carpets are laid in single courses or double courses and are classified as open graded or dense graded. Two-course carpets have a wearing course (or top course) provided on top of base course. Single course carpets are usually limited to a thickness of 7.5 cm. The medium and close textured (or dense) carpets 2 cm to 5 cm thick are laid as single courses (as wearing courses) on existing roads which are structurally sound and of reasonably good shape but in need of a new running surface. Thick carpets are generally laid in two courses. The total thickness of two-course paving may very from 6 cm to 12 cm according to the traffic and the condition of the road bed. The top course can be 1.5 cm to 4 cm thick, but it is generally kept about 1.5 cm thick and made as a seal coat.

The function of the top course is primarily to water-proof the pavement and to provide a better running surface. Two-course pavement is preferable over weak foundations. The support value of the base largely governs the minimum thickness required.

Although single-course work is chosen generally for speed of construction and low initial cost, it is more difficult to attain an even running surface and a uniform texture with it, than with two-course work.

The cost of a 2.5 cm thick carpet is only slightly, if at all, above that of a two-coat surface dressing, and the surface becomes more stable as excess of binder which results in bleeding and corrugations in two-coat surface dressing is avoided.

Open-textured and Close-textured (Dense) Bituminous Carpets

In open-textured surfaces, failure is most likely to be due to the weathering of the binder, whereas dense surfacing tend to fail by waving and deformation. Open-textured road surfaces are more liable to fail by disintegration than close-textured. Open-textured construction is more suitable as a base course than as a wearing course, and is not, as a rule, sufficiently durable. The incorporation of fine aggregate increases the durability of surfacing by closing its texture.

Whereas thin surface dressings and open-graded pre-mix carpets would, by and large, suffice for medium to heavy traffic, for very heavy traffic and heavy snow precipitation areas, as well as at locations like bus stops and roundabouts, consideration ought to be given to the provision of dense asphaltic concrete in single or multiple courses, so as to render the surface more stable and waterproof. Close-textured wearing courses are most suitable for heavy traffic.

When a roughened surface is required, in a two-course carpet, gritting should be done on the first course with chippings of either 20 mm or 12 mm single size, with 2 to 3 per cent filler, pre-coated with 2 to 3 per cent bitumen.

Asphaltic Concrete consists of a mixture of coarse aggregate, fine aggregate, and mineral filler, mixed with binder formed into a dense carpet used as wearing course, 25 to 50 mm thick.

Carpets with tar are made close textured type by adding a substantial amount of fine particles (sand) with the stone because tar cannot stand much of weathering as explained earlier. Cut-backs and emulsions are not generally used for carpets more than 25 to 40 mm thick.

Joints in Bitumen Roads. Longitudinal joints and edges are constructed parallel to the centre line of the road, and these joints are offset by at least 150 mm from those in the lower course. All joints are cut vertical to the full thickness of the previously laid mix and the surface painted with hot bitumen, before placing fresh material. At shut-downs and end of day's work, transverse joints are formed by rolling over the edge and then cutting back a vertical joint at full depth. For joining new work with old work, cut back the end portion of the old work by about 60 to

90 cm at 45-deg. to the section of the road up to the bottom. Paint the cut surface with hot binder and put on the new mix. Ram the joint with hot iron rammers.

Priming coat. (Also see under "Glossary of Highway Engineering Terms".)

A coat of low viscosity binder (cutback bitumen, tar or cold emulsion) is applied as a primer over an existing road surface (generally a water-bound macadam) if it is dusty, porous and absorptive, to develop a bond between the existing road and the surface application, as a bitumen will not adhere to a dusty and porous surface. Roads built with laterite, kankar, brick metal, moorum, etc., (water-bound macadam) are absorptive. The function of a primer is to penetrate the existing surface, fill the pores, bond together any loose material, and in general, produce a firm impervious layer to serve as a foundation for the material to be superimposed.

Priming coats are not given on cement concrete or dense bituminious surfaces, but a tack cost is applied instead. A coat of primer is not usually required where surfacing is to be done with tar.

Tack and priming coat are not given together

The choice of a bituminous primer, as to its viscosity and quantity, depends upon the porosity characteristics of the surface to be primed. The more porous the surface and cooler the weather, the lower should be the viscosity. Conversely, if too much material is being absorbed, a more viscous grade should be used. A more viscous primer should be used on open textured roads and a thin primer on close textured roads. The amount applied should ordinarily be absorbed in 24 hours. The primer should be applied uniformly by means of a sprayer.

No bituminous primer shall normally be applied on a wet surface, or during a dust storm, or when the weather is foggy or rainy.

The surface to be primed should be swept clean free from dust, and for best results, be dry, Large irregularities, pot holes, depressions, etc., should be repaired prior to priming. The primer should be allowed to soak (cure) into the road for 24 to 48 hours.

In the case of very dry and dusty surface it is often sound practice to moisten the road slightly before application of the primer.

When the priming has been completed (cured), any spots showing surplus bituminous material should be blotted up by sprinkling coarse sand over the portions, to any spots still showing porosity, a little more primer may be applied

Normally cutback bitumens of 0 or 1 grade or road tar of RT-1 grade are used as primers. The quantities may be as follows on water-bound macadam surfaces:—

75 to 145 kg/100 sq. metres if cutback is used, which may be 73 to 98 kg for low porosity, 98 to 122 kg for medium porosity and 122 to 146 kg for high porosity road surfaces.

120 to 150 kg/100 sq. metres if tar is used
100 kg/100 sq. metres if emulsion is used

Tack coat. A tack coat is given over the old road surface if it is smooth or of cement concrete, and on bituminous base courses if the surface is dry, hungry and dusty, before laying a thin premixed carpet, to ensure a proper bond between the base and the superimposed mat or carpet which must be made to adhere to the old road surface. Tack coat is not normally specified for carpets 5 cm thick and above. The tack coat should be applied just ahead of and keeping pace with spreading of the premix, only long enough that it shall not have cooled and hardened before the mat is placed on it.

Tack coat consists of application of a single coat of low viscosity liquid bituninous material and which may be of bitumen, tar or emulsion, the quantity depending on the texture of the surface and the viscosity of the binder to be used. Lesser quantity is required on smooth surfaces and under light sand carpets while more is needed under carpets of stone aggregate and on open-textured surfaces. A tack coat must not be applied over a surface which is rich in binder from accumulation of surface dressings. Where a tack coat is considered necessary it should be applied thinly and uniformly, by a sprayer or a pouring can.

The surface of the road on which tack coat is to be applied should be throughly swept and scraped clean of dust before applying the binder. The cleaned surface of the road should be moistened with water before the application of an emulsion tack coat to help it penetrate into the surface to some extent and also help to lay the dust. When the surface consists of moorum, kankar or similar material, it is advisable to give the surface a priming coat. If, however, the surface is good water-bound macadam a bond can be ensured by brushing the surface and making it slightly rough so that the carpet may bond into the interstices, but in that case a priming coat is essential.

Normally road tar RT-3 or 80/100 penetration bitumen are applied. The quantities may be as follows per 100 sq, metres of road surface:—

Using hot bitumen	*Using tar*	*Using emulsion*
(a) On untreated water-bound macadam surfaces		
75-100 kg	120-150 kg	100 kg
(b) On existing black top dry surfaces		
50-75 kg	70-100 kg	100 kg

Application of a tack coat is not necessary when a premixed corpet is to be laid soon after the provision of a bituminous base/levelling course.

Spreading and Rolling. The premixed material should be spread on the road surface as soon as possible after mixing in cold weather and rolling started soon after a length of about 60 metres has been laid and the mix has cooled down a little, but during hot weather tar macadam may be exposed to air for a period not exceeding 2 days with advantage. Any macadam which has become set should be rejected.

Bituminous macadam shall not be laid during rainy weather or when

the base couse is damp or wet or when the atmosphere temperature in shade is 16 deg.C. or below.

The premixed material is spread with rakes to the required thickness and camber or distributed evenly with a drag spreader, making a 20 per cent allowance for shrinkage during rolling, and is consolidated with 6 to 12-tonne power rollers, preferably of smooth wheel tandem type. (Some engineers prefer to give first rolling with a light roller, say of 4 tonnes, so as to prevent shoving up in front of the roller wheels, before rolling with the heavier roller.) The edge line and thickness are marked by wooden scantlings spiked on the base true to grade and levels, or with edge supports described earlier. (Also see under "Rolling" at page 18/26.)

The roller should proceed on the fresh material with rear or fixed wheel leading so as to minimise the pushing of the mix, and each pass of the roller spould uniformly overlap one-third of the track made in the preceding pass. If possible a set of rollers should be used moving at a speed not exceeding 5 km per hour.

Base course and top course (wearing course) should be rolled separately. Base course if well laid can be allowed to carry traffic for sometime so as to become compacted prior to receiving the wearing course. The rolling on the base course need not be so rigorous as on the top course so that the top course may join well with the base course when laid over it. While the rolling is in progress the surface of the road should be checked for correct levels, surface evenness and shape by means of a wooden template. Any high spots or depressions should be corrected by removing or adding fresh premixed material. Fat spots should be cut out and refilled and tamped. Rolling should then be continued until the entire surface has been rolled to compaction and all the roller marks elimated. Too much rolling of coated chips is injurious. The wheels of the rollers should be kept moist to prevent the coated material from adhering to the wheels and being picked up. In no case should fuel/lubricating oil be used for the purpose. Rollers should not stand on the newly laid material while there is a risk it will be deformed thereby. Traffic may be allowed after completion of the final rolling when the mix has cooled down to the surrounding temperature.

Where an emulsion or cut-back has been used for a premixed carpet, the pre-mix should be left to dry out after mixing for 2 to 3 hours. When the coated aggregate has dried out sufficiently to allow of its being handled without stripping it should be laid on the road and left unconsolidated for about 24 hours or more for the moisture to dry out thoroughly.

Seal Coat is applied on open textured bituminous road surfaces for sealing the voids to render the surface water-tight and strengthen the macadam, reduce porosity and tendency to disintegration. Seal coats are of two types:

Type A: Liquid seal coat (for high rainfall areas) comprising an application of a layer of bituminous binder followed by a cover of stone chippings, and lightly rolled. It is more or less like a renewal coat of surface dressing. The quantity of binder in terms of straight-run bitumen is 98 kg per 100 sq. metres of the road surface. Stone chippings used are of 6 mm size (all passing through 10 mm sieve and retained on 2.36 mm sieve). The quantity used is 0.9 cu. metre per 100 sq. metres.

Type B: Premixed seal coat (for low rainfall areas) comprising a thin spread of premixed fine grit or coarse sand. The quantity of binder used for premixing is 68 kg straight-run bitumen or 98 kg tar per 100 sq. metres of road surface. Stone grit used is passing 1.7 mm sieve and retained on 1.18 mm sieve. The quantity used is 0.6 cu. metre for 100 sq. metres of the surface.

The seal coat is applied immediately after the laying of bituminous course. Before application of the seal coat the road surface should be cleaned free of any dust or other extraneous matter. The seal coat surface is lightly rolled with a 6 to 9-tonne smooth wheeled roller, preferably a tandem roller.

Softer grades (less viscous) of binders are used for seal coats. The quantities of aggregates may be increased up to about 20 per cent for heavy applications. (Also see Table "Choice of Grade of Binder for Different Types of Road Conrtruction.")

Opening to Traffic. In the case of Type B seal coat (where premixed aggregates have been used) traffic may be allowed soon after the final rolling when the premixed material has cooled down to the surrounding temperature. But in the case of Type A seal coat (surface dressing work) traffic shall not be permitted to run on a newly surface dressed area until the following day. In special circumstances, however, the road may be opened to traffic immediately after rolling with resticted speed up to 16 km per hour, until the following day.

AGGREGATE GRADATION FOR PREMIXED BITUMINOUS CARPETS

Sieve Designation	Semi-Dense Carpets: 25 mm Compacted Thickess	Semi-Dense Carpets: 20 mm Compacted Thickness	Asphaltic Concrete, 25 to 40 mm Compacted Thickness: Grading 1	Asphaltic Concrete, 25 to 40 mm Compacted Thickness: Grading 2
	Per cent by weight passing the sieve			
20 mm	100	—	—	100
12.5 mm	75-100	100	100	80-100
10 mm	60-85	75-100	80-100	70-90
4.75 mm	35-55	35-55	55-75	50-70
2.36 mm	20-35	20-35	35-50	35-50
600 micron	10-22	10-22	18-29	18-29
300 micron	6-16	6-16	13-23	13-23
150 micron	4-12	4-12	8-16	8-16
75 micron	2-8	2-8	4-10	4-10
Binder content	5.5 per cent by weight of the total mix		5 to 7.5 per cent by weight of the total mix.	

The binder considered is straight-run bitumen of a suitable grade.

These mixes are suitable as wearing courses.

Asphaltic concrete as wearing course is of single layer 25 to 50 mm thick. For compacted layer thickness of 25 to 40 mm any of the two

gradings could be used, but for layer thickness of 40-50 mm, only grading No. 2 may be used.

These mixes need filler. See under "Filler" for details.

AGGREGATE GRADATION FOR 75 MM AND 50 MM COMPACTED THICKNESS OF BITUMINOUS MACADAM (PRE-MIXING)—IRC

Sieve Designation	75 mm Compacted Thickness: Grading A	75 mm Compacted Thickness: Grading B	50 mm Compacted Thickness: Grading A	50 mm Compacted Thickness: Grading B
	Percent by weight passing the sieve			
63 mm	100			
50 mm	90-100		100	
40 mm	35-65	100	90-100	
25 mm	20-40	70-100	50-80	100
20 mm	—	50-80	—	70-100
12.5 mm	5-20	—	10-30	—
10 mm	—	—	—	35-60
4.75 mm	—	10-30	—	15-35
2.36 mm	—	5-20	—	5-20
75 micron	0-5	0-4	0-5	0-4

The binder (straight-run bitumen) shall be 3.5 and 4.0 per cent by weight of the total mix for aggregate grading No. A and B respectively. The optimum binder content is bassed on the surface area of the aggregate.

AGGREGATE GRADATION FOR MIX SEAL SURFACING 20MM THICK PREMIX DENSE CARPET

Sieve Designation	Per cent by weight passing the sieve: For Type A	For Type B
12.5 mm	—	100
10 mm	100	70-100
4.75 mm	40-85	20-40
2.36 mm	5-20	5-20
75 micron	0-4	0-4
	Quantites of Aggregates and Binder Per 100 sq. metres	
Aggregate	2.7 cu. m	2.7 cu. m
Binder	220 kg	190 kg

Binder for pre-mixing

(a) Using hot bitumen/cutback:

52 to 56 kg/cu. m of aggregate 12 mm size and above

56 to 64 kg/cu. m of aggregate 10 mm size

115 kg/cu. m for stone grit

128 kg/cu. m for coarse sand (Carried over to page 18/78)

TABLE 1 — Pre-mixed Bituminous Open-Textured Road Carpets

Aggregates Required per 100 sq. meters of Road Surface

Nominal size of aggregate	Aggregate in cu. metres for consolidated thickness of carpet in cm											
	7.5		6		5		4		2.5		2	
50 mm	6 0 *	—		*		*		*		*		
40 mm		6.6	5.4	5.4								
25 mm		2.4	2.1		4.5	4.5						
20 mm	—	—	—	—	1.5	—	3.3	3.3	—	—	—	—
12 mm							1.4			1.8		1.8
10 mm									3.0		2.4	0.9
6 mm	3.6			2.7		2.1		1.7		1.2		

Seal Coat

	7.5	6		5		4		2.5		2	
(i) For liquid seal (high rainfall areas)— stone chippings 6 mm nominal size for blindage	1.5	1.5	—	1.5	—	0.9	—	0.9	—	0.9	0.9
(ii) For pre coated seal (low rainfall areas)—	2.4	2.4	—	1.8	—	1.2	—	medium coarse sand or fine grit 0.60 to 0.75 cu. m			
Binder for liquid seal for spraying on carpet	140 kg per 100 sq. metres of surface							98 kg per 100 sq. metres			

(i) The binder should be fluid enough after spraying on the road to 'wet' thoroughly both the surface and the cover aggregate when the latter is spread. Too hard a grade of bitumen will not maintain the necessary fluidity.

(ii) Less tar is required for dry climates.

(iii) Lesser quantities of binder and smaller sizes of aggregate are for dense and compact surfaces, and for lighter traffic.

(iv) If the base to be treated consists of a porous surface, a primer should be employed prior to surface painting.

TABLE II — Pre-mixed Bituminous Close-Textured (Dense) Carpets

Aggregate Required per 100 sq. metres of Road Surface

Nominal size of aggre-gate	Aggregate in cu. metres for consolidated thickness of carpet in cm													
	7.5	6	5		4		2.5	4		2.5			2	
50 mm	4.38	—								For light intensity traffic such as garden paths and drives. Also for providing a thin wearing course over existing cement concrete roads. (Also called bituminous sheets.				
40 mm	2.92	3.48	—											
25 mm		2.32	2.88											
20 mm			1.92	4.80	2.28									
12 mm					1.52	3.60	2.10	1.56	1.5	0.99				
10 mm							0.90	1.04	—	0.66	1.80	1.95		
6 mm											0.60		2.25	
Medium coarse sand	3.65	2.90	2.40	2.40	1.90	1.80	0.95	2.50	1.65	1.65	0.60	0.90	0.30	1.95

(v) As far as practicable, surface painting shall be carried out only when the atmospheric temperature in shade is 16 deg. C. or above.

(b) Using tar:
66 to 80 kg/cu. m for aggregate
118 kg/cu. m for coarse sand

(c) Using cold emulsion:
96 kg/cu. m for stone grit

Notes on Tables

The size of aggregate given are "nominal" sizes according to IRC classification.

The quantities given are only approximate for estimating purposes (which in fact, are very variable and depend upon many factor). 5 per cent should be added to the aggregate and 2½ per cent to the binder for wastage.

Table I—Open-Textured Carpets

(i) Add filler where necessary—*See under* "Filler".

(ii) Crushed rock and graded aggregate should be used as far as possible. Where uncrushed and rounded river gravel or shingle are used, add coarse sand

Seal Coat—Surface dressing (gritting) or seal coat is required over open-textured carpets. (a) Carpets marked with *need not have a seal coat on top or a seal coat of coarse sand may be given according to the type of surface desired. (b) Fine grit is wholly passing IS sieve 1.70 mm and wholly retaind on 180 micron. 6 mm chippings is passing 10 mm sq. mesh and retained on 2.36 mm mesh. (c) For lighter applications the quantities of chips and binder may be reduced by about 20 per cent. (d) In case an emulsion is used, the quantities of stone grit and binder will be half.

Table II—Dense Carpets

The quantities for aggregate are stones before mixing with binder, coated stones will increase slightly in bulk. These carpets will need a tack coat and no seal coat except "dusting" with stone dust after rolling. (c) Excess of sand should not be used as that will tend to separate the stones and prevent their interlocking. Filler may be added where necessary. (d) 1 part of sand to 2 of aggregate will give a dense mixture while 1 part of sand to 4 of aggregate will give a lean mixture(e)The aggregate should not contain more than 15 per cent by weight of material passing a 75 micron IS sieve (200 BS). Excess of fines (material finer than No. 200 BS sieve) is likely to cause trouble. With the quantity of binder prescribed for fine material, if the mixture gets too "fat", additional fines should be added.

Fine aggregate increase the stability and strength of the mix by reducing the voids in the coarse aggregate. It also enables a mix to carry a higher content of binder which give the mix greater durability.

Thin Carpets

Thin carpets of thickness 20 mm, 12 mm or even up to 6 mm can be made of sand pre-mixed with bitumen or tar. The sand to be fine gritty sand, all passing a 170 micron mesh sieve, filler is added to the mixture. The carpet is laid on the road suface with a float and rolled with a light roller of about 4 tonnes weight. A very light sprinkling of stone dust or cement is given on the surface just before the final rolling to give it a non-tacky finish.

A sand carpet has little inherent strength of its own and is therefore suitable for use only on a subgrade of adequate bearing power or on a properly prepared base. Advantage is that it protects the subgrade from the effects of heavy rains.

Open-textured carpets may have a high proportion of 20 mm or 12 mm aggregate with about 5 to 20 per cent of fine stone grit. Medium-textured carpets may have graded aggregate from 12 mm or 10 mm down, and fine-textured carpets 6 mm down stone grit or sand.

If after preliminary compaction (light rolling) pre-coated chippings of size 12 to 20 mm are sprinkled over before the mixture has set, and embedded into a sand carpet by light rolling, pneumatic tyred traffic can be allowed on it.

Sand or fine grit carpets are useful for surfacing of tennis courts garden paths, drives, railway platforms, etc. These mixtures can also be laid on doubtful and soft surfaces in preference to surface dressing.

A cheap carpet suitable for footpaths etc., can be constructed as follows:

The subgrade to be of gravel, brickbats or kankar, well consolidated and cambered. 12 mm hard stone chippings, about 2 cu.m per 100 sq.m for a 20 mm thickness are uniformly spread over the surface, well watered and consolidated with a light roller to press the chippings into the surface. Dry coarse sand is sprinkled over the chippings at the rate of about 0.3 cu.m per 100 sq.m. Cold emulsion is applied over the surface at the rate of 200 kg per 100 sq.m after it has been moistened with water. Coarse sand is then spread at the rate of 0.6 cu.m per 100 sq. m over the emulsion after it 'breaks'. Surface to be rolled again. A light seal coat with emulsion can be given over this surface after about a fortnight, if necessary.

Sand carpets should not be laid on grades steeper than 1 in 20 as they are apt to become slippery in hot sun or with excess of binder or when wet. Any mixture containing a high percentage of fines becomes unstable with only a small excess of binder. If pre-coated chippings of size 12 to 20 mm are sprinkled over a fine sand carpet before the mixture has set, as detailed before, it will remedy slipperiness.

Thin carpets are particularly useful in restoring good riding qualities of a bad road to a considerable extent although a thin carpet will follow any unevenness that may exist on the old surface.

Fine sand is more suitable for heavy traffic as large grains are more readily crushed than fine and resistance to displacement of binder is also increased for increase in area of contact between individual particles. With light traffic proportion of coarse grains can be increased with advantage However, the quantity of fine sand should not be in excess of the percentage mentioned above. Impurities of any type, clay, loam or such fines as oxides, and organic matter in any form in sand are undesirable. Mica in excess is also harmful. Crushed limestone should not be used.

Pat Test for Sheet Asphalt or dense carpets—Select small sample of hot mix and place it atonce upon a sheet of unglazed manilla paper, resting upon a flat board. Fold the paper over the sample and press heavily with the flat of a wood paddle. Strike the paper a sharp blow with the paddle, open the paper and remove the sample. If the stain is medium dark, bitumen content is about right. If it is very dark or sloppy, bitumen is excessive. If it is light and dry, bitumen is insufficient. If only the imprint of single sand grains appear, the amount of filler is deficient. If the space between sand grains is filled in, angregate grading is good.

Quantities of Materials Required for 20 mm thick Open-Graded Premix Carpet for 100 sq. metres (IRC)

(i) Aggregates for Carpet	
(a) Stone chippings — 12 mm size	1.83 cu.m
(b) Stone chippings — 10 mm size	0.91 cu.m
(ii) Binder for Premixing — bitumen or cutback	
(a) For 12 mm aggregate at 52 kg/cu.m	95 kg
(b) For 10 mm aggregate at 56 kg/cu.m	51 kg
	146 kg
Ditto. using tar of grade R.T. 3 or R.T. 4 For all aggregate at 72 kg/cu.m	197 kg

Seal Coat

Premixed seal — medium coarse sand or fine grit 0.6 cu.m 68 kg bitumen or 98 kg tar R.T. 3

Liquid seal — coarse aggregate 0.9 cu.m and bitumen 98 kg

Tack coat

Using straight-run bitumen
On WBM surface 73 to 98 kg
On black top surface 49 to 73 kg

Using tar

Priming coat on WBM surface 122 to 147 kg
Tack coat -- black top surface 73 to 98 kg

16. SURFACE DRESSING OR SURFACE PAINTING

Painting or spraying a road surface with a thin layer of a binding material (bitumen, tar, etc.) followed by a covering of stone chippings or coarse sand which is then lightly rolled, is called surface dressing. Surface dressing over an existing black top surface is often referred to as a *seal coat.*

The main functions of surface dressing are:

(i) To provide a dust free wearing (or riding) surface over a base course such as water-bound macadam. (ii) To prevent the entry of water into the road structure and the sub-grade. (iii) To protect the water-bound macadam by preventing the removal of the binding material between the stones by the action of pneumatic-tyred traffic. A water-bound macadam surface though slowly crushed under solid tyres (cart wheels) does not disintegrate but the fast moving motor traffic rocks the metal pieces and sucks out the fine blindage and loosens up the surface which gradually extends downwards. Surface dressing provides a thin cushion between the wheels of traffic and the road metal thereby protecting the road metal to a large extent from disintegration commencing from the surface. (iv) To prevent disintegration of an old bitumen surface dressing showing signs of wear. (v) To provide a smooth riding surface due to its flexibility. (vi) To resist the abrasion and wear of traffic.

Surface dressing is not a cure for a defective water-bound macadam surface. If the water-bound macadam is not properly consolidated, it will ravel and rut; if the metal is soft, it will crush under the bullock-cart traffic; if the water-bound macadam surface is uneven and rough before surface dressing is done, it will remain uneven and rough after surface dressing. Surface dressing does not improve the riding qualities of a road but will reproduce the irregularities of the old surface (except for a short period). It only retards disintegration of the surface and does away with the dust nuisance of a water-bound macadam surface.

Surface dressing may be applied in one or more coats, according to the traffic conditions and condition of the road surface, with a hot or cold binder and may be done on a water-bound macadam surface, stabilized gravel or soil stabilized road. Ordinarily single coat heavy dressing will meet all average conditions, but where the existing road surface has become uneven, wavy or has otherwise lost its shape and minor pot holes have developed, a *carpet* will be more suitable. A carpet will produce a non-skid surface and there will be no bleeding or corrugations which often follow two-coat work not done very carefully. Several coats of paint at comparatively short intervals or repainting before there had been any appreciable reduction in the thickness of the existing coat or coats by the action of traffic, should be avoided.

Preparation of the Existing Road Surface for Dressing

It is very essential to examine the water-bound macadam carefully before deciding to surface dress it. (See Patching of Potholes under "**Preparation of Base for Laying Bituminous Courses.**") If the existing surface shows any depressions, potholes, ruts or irregularities exceeding 12 mm or has a steeper camber than 1 in 60, it must be re-conditioned. If the surface is not so bad as to warrant resectioning, it should be patch repaired a week or two before the surface dressing is to be started, and left open to traffic. Stone metal of the old untreated road surface should be exposed to a depth of 6 mm to 12 mm to give the binder a better opportunity of penetrating and binding the road. Care should, however, be taken to see that the stability of the road is not disturbed and the stone metal is not loosened.

It is most important that the surface of the road shall be thoroughly cleaned before the application of a binder. It shall be swept clean and free from dust, dirt, caked earth and any other extraneous matter. The cleaning shall be done first with hard (or wire) brushes, then with softer brushes or base brooms and finally by fanning or blowing with gunny bags or sacks to remove all loose dust. The road must be absolutely dry before painting is commenced and this dry condition must extend right down to the earth subgrade (or for at least 6 cm in case of emergency).

Preparation of Newly Metalled Road Surface for Bituminous Treatment

If a newly metalled road has been well shaped and rolled smooth, it is advisable to allow traffic on it for some time say, for a year before giving a surface treatment. During that period the crust will have become firm and consolidated and soft material on the top will have disappeared to a great extent. Some departmental specifications prescribe that painting should not be delayed for more than 3 months after consolidation. But if surface dressing is to be done, it must be carried out before the road crust starts breaking up. Quantities of matrerials required for patch work:

Painting potholes with binder ... 3 liters/ sq. metre
Binder for pre-mixing ... 48 kg/cu. metre of stone

For estimating purposes the following quantities of road metal may be taken annually per kilometre of road length, and 3.66 metres (12 ft.) width of metal:—

Light traffic	12 to 15 cu. m
Medium traffic	17 to 25 cu. m
Heavy traffic	35 to 70 cu. m

For portions on gradients, add 25 percent extra.

When a water-bound macadam road is laid, black-topping is not generally provided immediately, because traffic is less in the initial periods. As the intensity of traffic increases black-topping of the road is undertaken.

Permitted Surface Evenness for Surface Dressing (IRC)

Surface evenness in longitudinal and transverse directions

Type of base on which surface dressing is applied	Longitudinal profile	Cross profile
	Max: undulations when measured with a 3-metre straight edge	Max: variations from specified profile when measured with a camber template
(1) WBM, penetration macadam or built-up spray grout	12 mm	8 mm
(2) Bituminous macadam	10 mm	6 mm

The longitudinal profile is checked with a 3-metre long straight edge at the middle of each traffic lane along a line parallel to the centre line of the road. The transverse profile is checked with a series of three camber boards at intervals of 10 metres. Undulations if any, should be corrected by addition or removal of blindage.

The underlying course (receiving surface) on which surface dressing is to be laid should be shaped and conditioned to the specified grade and camber.

Weather and Seasonal Limitations for Surface Dressing

Preferable, the surface dressing work with cutback bitumen or tar should be carried out only when the atmospheric temperature in shade is 16 deg. C. or above. In lower temperatures cutbacks should be used. Normally no bituminous material should be applied when the surface or the aggregate is damp, when the weather is foggy or rainy, or during dust storm. (See also "Work Under Wet Conditions" in the pages following.)

Rate of Spread of Binders

The rate of spread of a binder is probably the most important factor affecting the quality of a surface dressing. Too much of binder can lead to bleeding, resulting in a slippery surface in wet weather; too little can cause the loss of blindage aggregate which will be flung off the road by fast moving pneumatic-tyred traffic. The quantity of binder required depends on the following factors:—

(i) The nature and condition of the existing road surface: open-textured and porous surfaces, being absorptive, require more binder, and also the surfaces appearing dry and deficient or lean. Less binder is required on existing road surfaces which are rich in binder. A thicker film of binder and of low viscosity is desirable on roads that are being dressed for the first time.

(ii) Climatic condition: Heavy rainfall areas warrant a heavier application of binder than necessary in low rainfall areas.

(iii) The traffic intensity: More intense the traffic, lesser the quantity of binder required, as the compaction of the surface is increased. Higher viscosity binder is required on roads carrying more intense or fast moving traffic so as to resist more effectively the disruptive forces of such traffic.

(iv) The size and shape of the aggregates to be used. Large size of aggregates require more binder. Rounded aggregates (gravel) require more binder than angular or cubical stones. Flaky or flat stones require less binder than cubical stones.

The binder should neither be too much nor too little but just right all over the road. A relatively small difference in the rate of spread of the order of 50 kg/100 sq. metres can have a market effect on the results. Due care should therefore be taken to ensure the binder being applied uniformly at the correct rate. As the spraying operation is not susceptible of very accurate control, many variations of the binder content might be expected.

Weak spots may be caused by local penetration of most of the binder into a loosely compacted water-bound macadam leaving almost no binder on particular portions of the surface. In an open-textured and porous old road surface the binder will penetrate into the pores and there will be less remaining on the surface to hold the chippings. If the binder content becomes excessive, deformation will result. The use of smaller chippings is helpful in these cases.

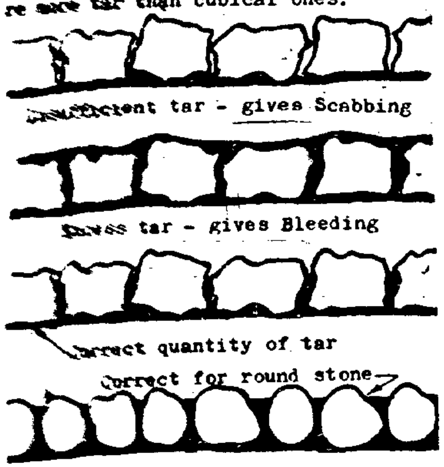
The grade of binder to be used would depened upon the climate conditions. The suggested binders are straight-run bitumen 80/100, road tar grade RT-3 or RT-4 (which are most commonly used in India) or a cut-back.

Method of Application of Binder: *(Tar or Bitumen)*

Heating:

All the tar drums should be carefully examined before discharging into the heaters to see that the contents have not been mixed with water. Great care should also be taken to see that no water enters the tar boiler otherwise the tar will froth up and overflow. A full drum must always remain suspended over the boiler so that the contents will never be reduced to less than one-fourth volume, otherwise it will cause a rapid rise in temperature.

Diagram showing how rate of spread of tar is related to the size and shape of chippings: Rounded chippings require more tar than cubical ones.



The heater should have a capacity in keeping with the daily output required and should be so operated that the temperature of the binder discharged to the road surface is within the range prescribed. A suitable thermometer (reading up to 300 deg. C.) should be provided at the site.

For small jobs tar can be heated in drums with a portion of the side cut out. Bitumen cannot be thus heated as it has to be raised to a much higher temperature which may not be possible.

The binder should be applied evenly (to the clean dry surface) by means of a pressure sprayer. An even and uniform distribution of the binder should be ensured from the calibration chart showing the relation between pumping presure, height of nozzle above the road surface and the amount of binder of a given grade that would be deposited per minute at the specified temperature.

The application of a binder is more efficiently and quickly done with pressure sprayer than by hand pouring. For hand pouring specially constructed pouring cans with wide mouths and of known capacity for a definite area are used. To obtain correct and even distribution of the binder the road surface may be divided into rectangles of known area, each suitable for the contents of one pouring can.

Pumping must be steady so as to maintain a constant pressure otherwise there will be uneven distribution of the binder. The rate of application is controlled by maintaining a constant reading in the pressure gauge. Pumping may only cease when there is nothing but hot air coming out of the nozzle. Spraying should in all cases be carried out parallel to the centre line of the road and never across the road. If sprayed across the road, unsightly overlapping will occur. Excessive deposits of the binder on the road caused by stopping and starting the sprayer, or by leakage, should not be allowed.

The operation is carried out by one man spraying the binder and two men equipped with long handled soft brushes. The binder is brushed evenly over the surface longitudinally immediately after pouring and excessive deposits due to overlapping or other causes reduced to minimum. Brushing shall always be done from the sides towards the crown. Rubber squeegees with long handles are also used for distributing the binder evenly on the surface. The binder should be brushed in one direction only as far as possible and not backwards and forwards.

The quantity of binder applied to the road shall be checked from time to time to ensure that an even and uniform distribution is being obtained and the height of the sprayer nozzle above the road surface fixed accordingly. To achieve an even application the lance should be rotated in an even circular motion with the spray jet at a fixed distance from the road surface. The circular motion obviates the transverse striations so often caused by swinging the lance from side to side. If hand pouring pots are used and two coats are to be done simultaneously, the line of distribution shall cross those of the first application at right angles. Brooms must be cleaned at the end of day's work.

At the end of days's work or when the spraying is to be stopped for any period, hot air should be passed under pressure through the delivery pipe and the nozzle for cleaning them of any residual material. Similarly, before starting spraying, hot air should be passed again to test the nozzle for cleanliness and free flow of the binder. Neglect of this may result into serious accidents. (Air device is fitted into boilers.) Labour employed on heating and applying binder should be provided with boots, putties and goggles.

Shape, Size, and Rate of Spread of Chippings

The aggregate should be of non-flaky, hard, tough, clean, crushed or broken rock, crushed gravel, roughly cubical is shape. A chipping that can be crushed by a boot-heel is unsuitabe. In general, harder stones will give a longer wearing life, although many dressings do not fail by the wearing away of the stone. It has been found that angular rather than rounded, sub-angular or flaky particles, give the best performance as this angular form provides a maximum surface area to assist adhesion with the binder.

A "single size" chippings are often desirable. If the chippings contain a range of sizes the smallest particles tend to prevent the larger ones making contact with the binder at the time of application. A small size particle lying between the larger ones will have its surface lower down and will not help in bearing the traffic loads; it will lie loose as it will not be pressed by the roller wheels. The bigger size of stones project above the surface of the smaller size and take all the traffic, get out and fail. However, it is preferable to use graded aggregate on rough textured and open surfaces and single size aggregate on smooth surfaces. The grading may be done by mixing 2 parts of the bigger size and 1 part of the smaller size prescribed (nominal size). Sufficient stone should be applied to give slightly more that shouder-to-shoulder cover.

Some engineers, however, consider that since nearly half of the chips used in surface painting crush down to sand and dust, it is more economical in most cases and frequently sound practice to use a percentage of sand in lieu of all chips only. The suggested proportions are 2 parts chips and 1 part sand. The chips must be applied first and rolled before the sand is spread.

Sand alone may be used for light applications if desirable for economy. A dressing of sand on an previously treated road will last for about six months under ordinary traffic. Sand renders the surface slippery in wet whether but forms a water-proof mat. Medium or coarse sand should be used.

Size of Chippings — The choice of size of chippings should depend upon the intensity of the traffic and the condition of the road. Small sizes, 10 mm or 6 mm are desirable where the surface of the road is very hard or where the traffic is relatively light. For the more heavily trafficked roads and when the road surface is relatively soft or rich in binder, or on rough surfaces, or where the problem is to deal with iron-tyred bullock-carts, larger size 20 mm are suitable. For a surface dressing on tightly bonded (smooth surface) water-bound macadam road, the size of the chippings should be 3 to 10 mm according to the surface. 10 or 12 mm (nominal size) crushed rock is considered most suitable for chippings for normal roads.

Surface dressings with smaller size of aggregate "flush" or "bleed" more frequently than those with a larger size.

Spreading of Chippings (or Gritting). — It is desirable that clean, dry chippings be used, properly screened. Best results are obtained when as little time as possible elapses between the application of a hot binder and chippings, and in no case should this interval exceed 15 minutes. The chippings should be uniformly distributed and precautions taken that the whole of the sprayed surface is covered uniformly without any accumulation of surplus material at any point. The chippings should be spread with a circular sweep of the shovel. Where different sizes of chips are to be used, spread the large size first and then the fines; this will give a smoother finish. If road is rutted, the bigger chips may be used in the ruts and smaller chips on the remainder. Hand brooming or light drag brooming should follow the application of the chippings prior to rolling. The excess of the chippings should be removed not less than 48 hours after application and should not be swept back on to the road. The chippings should not be dumped on the road, and should be cast parallel to the axis of the road and not across it.

When a rough surface is required, gritting should be done while the binder is still warm. The chippings should be either 20 mm or 12 mm single size.

Rolling Surface Dressings. — As soon as the binder has been covered and when it is still warm, and after the completion of light brooming, a suitable smooth wheeled roller should be passed over the whole surface and given a thorough rolling. The weight of the roller is governed by the character (hardness and size) of the chips the the under-surface. The smaller the size of chips the lighter the roller should be used. A few crushed chips do not matter but as soon as serious crushing commences to take place, rolling must be stopped. Do not over-roll and do not compact. The weight of the roller should preferably be restricted to 6 tonnes for 6 mm size of chips and 8 to 10 tonnes for bigger size. 6 to 8 trips of the roller should suffice for most of the works. Each pass of the roller should uniformly overlap not less than one-third of the track made in the preceding pass while rolling is in progress, additional aggregates should be spread by hand to make up irregularities. In places where traffic is light more rolling will be required than where it is heavy. Additional rolling may be necessary on the following day. Prefer tandem type roller for light rolling.

If coarse sand is used for blindage, rolling can be done with a hand or light power roller (4-tonne) during the warmer part of the day to embed the sand into the binder, but rolling on sand is not essential.

Although a roller is invariably used for the laying of every surface dressing, it is not essential. It is common experience that after the stone has been rolled by the iron roller a uniform appearance is obtained, but when the first rubber-tyred vehicle traverses the work the stone is disturbed and its wheel tracks are shown by the way each particle has been turned and lies in a reorientated position. It would not be unreasonable on this basis to dispense with the iron roller and to use rubber-tyred wheel rollers. The rubber tyres will give effective rolling without crushing.

Two-coat Surface Dressing

Two coats of surface dressing are generally necessary on untreated water-bound macadam roads subject to heavy traffic. A semi-grout with seal coat or a thin carpet should be preferred to two coats of surface dressing if cost is not prohibitive. Where the road is rutted or there are depressions more than 6 mm deep, two-coat dressing should be done.

The second coat may be applied 24 hours after the first coat or an interval of several months may elapse. The advantage of allowing an interval between each coat is that the first coat will have time to settle down under traffic and show up depressions which can be made up in the second coat, thus ensuring an even furface. When an interval is to elapse between each application, the method of construction is exactly the same as in the single-coat work. (See also IRC specifications at page 18/92.)

Where tar is used for two-coat work, the second coat is recommended to be applied immediately after the first coat, and before opening the road to traffic.

When the second coat is to be applied immediately after the first coat, the surface should be thoroughly cleaned and rolled lightly, (usually not more than twice up and down, just sufficient rolling to roll back any chippings that might have become loose and to bring the surface to a clean even finish) before application of the binder for the second coat. Simultaneous two-coat work should only be specified when a pressure sprayer is available as otherwise there is a possibility of excess binder staying on the surface and causing "bleeding". It is preferable to allow traffic for about a month's time.

Engineers are divided as to the quantity of binder and chips for the first and second coat in a two-coat work. Some recommend less of binder and more chippings for the first coat and more of binder and less chippings for the second coat, while others recommend vice versa. In fact, the specifications prescribed depend upon various factors as explained earlier. The tables show the quantities of materials usually recommended for the different types of works.

Some engineers recommend the use of medium coarse sand instead of stone chips for the first coat. This is especially beneficial where the water-bound macadam is loosely compected; sand forms a water-proof mat which stone chips do not and is more useful for rainy regions. Moisture remaining on the road surface for long will cause displacement of the stone chippings by gradually reducing adhesion between the binder and the stone chips. Cushioning formed by the sand mat increases the load bearing capaci · of the stone chippings of the second or wearing course. The sand surface will become smooth in about 2 to 3 months under traffic when the wearing course should be applied, and which should be applied within six months, and in any case before the rains start.

Renewal Coat is the coat of surface dressing applied when the original coat has sufficiently worn out but the road surface is smooth and not rutted, and is usually given before the surface start breaking up. The procedure of application is the same as for the first coat.

Opening to Traffic

Although traffic and weather together constitute the disruptive forces acting on the road surface, traffic itself is usually an essential agent in consolidating the newly laid surface dressing. In the early stages slow-moving traffic is particularly beneficial. Fast moving traffic, on the other hand, imposes horizontal forces on the road that tend to displace the stone in the surface particularly before the binder has hardened and the chips have acquired their interlocking mosaic. After this period any traffic that is relatively free from braking and turning continues to maintain the consoildation of dressing. The more intense the traffic, in fact, the closer becomes the compaction.

This goes to show that fast moving and heavy traffic should not be allowed on the road until the binder has set sufficiently to ensure that chippings will not be appreciably disturbed. This is particularly important with wet aggregate or in wet weather. Showers do not harm if traffic is kept off. Traffic should also be kept off the road during the hottest part of the day within the first 24 hours after laying and the road must not be used till the temperature falls. Even the highest viscosity tar will become fluid on the road on the first day. A surface dressing is in its most vulnerable condition when freshly laid, and keeping traffic off during the first few hours may, in certain circumstances, make all the difference between a good success and a bad failure.

Under normal conditions, it is generally recommended that when straight-run bitumen or tar has been used, traffic be allowed on the following day. If the road must be opened Immediately, speed should be limited to 16 kilometres per hour until the following day. Where cutback bitumen or emulsion has been used, the road should be kept closed to traffic until it has sufficiently cured to hold the cover aggregate in place.

Where pre-coated aggregate have been used for surface dressing, traffic may be allowed soon after rolling. Allow slow-moving traffic first for sometime.

Excess chippings thrown to the side of the road by traffic should be periodically brushed back over to the road. If the excess chippings do not adhere under traffic, they should be swept up and removed. If "bleeding" occurs subsequently due to excess of binder, it can be arrested by sprinkling coarse sand.

Treatment of Wet Aggregate

Provided the road surface has been thoroughly cleaned of all dust or other loose material by brooming the adhesion between the binder and the dry road is immediate and permanent. There is, however, no adhesion between sprayed hot binder and wet chippings until the chippings dry out by evaporation of the water. Rollings will press the stones into the binder, and provided they are not disturbed by traffic, they will dry out and adhere. Under poor drying conditions adhesion between the binder and the stone may not develop for some considerable time, and as the dressings cannot be isolated from traffic indefinitely, a large proportion of the stone may be flung off the road under the action of the fast moving and heavy vehicles. It is generally recommended that traffic be kept off new work until the stone has dried out.

For pre-mixing, in order to promote coating of cold and damp aggregates by tar or bitumen, it is necessary to add to the aggregate some reagent before the binder is incorporated in the mix. The reagents which have proved most successful are hydrated lime and quick-lime. Quick-lime causes a result similar to that of hydrated lime, but it is rather more effective in that it removes some of the water chemically from the stone and the resulting heat of reaction increases the surface temperature of the aggregate. Although quick-lime is particularly efficient, it has, of course disadvantages as regards handling. (For difference in quick-lime and hydrated lime see Section 12.) The aggregate is put into the mixer and the reagent added and thoroughly mixed. Then the correct quantity of binder is introduced and mixed until the aggregate is thoroughly coated.

It is not necessary to add any special reagent to tar and and the tar specified is usually of the same grade for use in wet aggregate mixes.

Work under Wet Conditions – The surface to be treated should be washed clean of all mud and water. If the road is wet, sprinkle fully freshly slaked white-lime powder (not kankar lime) at the rate of 25 to 35 kg/100 sq. metres of road surface according to the dampness on the surface, and spread the lime evenly, which forms into a slurry, with long handled brooms, so that the whole surface is covered with the slurry.

Wet stone aggregate are mixed with slaked white-lime powder at the rate of 10 kg of lime to 1 cu.m. of stone aggregate, and from 15 to 20

Surface Painting, Surface Dressing or Surfacing Roads

Quantities of materials required per 100 sq. metres of road surface

Binder				Stone Chippings	
Bitumen kg	*Cutback kg*	*Tar kg*	*Emulsion kg*	*Nominal size mm*	*Quantity cu.m*
		Single Coat Work			
		Light density rubber tyred traffic on water-bound macadam			
225	—	—	—	12	1.65
170—200	200—220	170—225	225	12	1.42—1.50
		Simultaneous Two Coat Work			
		Medium density rubber tyred traffic on water-bound macadam			
		1st coat			
170—200	200—220	200—225	—	12	1.40—150
—	—	—	200	12	1.35
		2nd coat			
100—120	120—150	120—150	100—130	10 to 6	0.90—1.10
		Repainting Old Painted Surfaces—Subsequent or Renewal Coat			
100—120	100—120	120—170	—	10 to 6	0.90—1.10
—	—	—	100—125	6 to 3	0.80—0.90

(i) Add 2½ per cent for wastage of binder and 5 per cent for wastage of aggregate.

(ii) The binder should be fluid enough after spraying on the road to 'wet' thoroughly both the surface and the cover aggregate when the latter is spread. Too hard a grade of bitumen will not maintain the necessary fluidity.

(iii) Less tar is required for dry climates.

(vi) Lesser quantities of binder and smaller sizes of aggregate are for dense and compact surfaces, and for lighter traffic.

(v) If the base to be treated consists of a porous surface, a primer should be employed prior to surface painting.

kg of lime to 1 cu.m. of sand. Aggregate and sand should be mixed independently with lime. If the previous road surface dries out after cleaning, no lime need be sprinkled over it. Mixing should preferably be done in a mechanical mixer and the binder sprayed with a pressure sprayer.

A tack coat can be applied over the lime treated surface where required under a thin carpet. Follow the usual sequence of operations either for surfacing or for pre-mixing.

There is, however, no completely satisfactory substitute for dry weather conditions. No bituminous material should normally be employed when the surface of the road or the aggregate is damp. The work should be deferred or discontinued when there is any likelihood of rain. Should rain fall on a newly applied coat, the road should be closed to traffic until it has dried.

Bituminous emulsion has the advantage over straight-run bitumens or cutbacks that moderate dampness of aggregate and road surface is not harmful. This allows the work to be carried out under a wider range of weather conditions. But it should be remembered that emulsion is suitable only for light traffic.

Estimating Quantities for Labour and Sundries

(a) Heating of Bitumen and Tar—Requirement of fire-wood:—

(i) 150 kg of fire-wood required for heating 1 tonne up to about 66 deg. C. (150 deg. F)

(ii) 250 kg of fire-wood required for heating up to about 120 deg. C. (250 deg. F.)

(iii) 400 kg of fire-wood required for heating up to about 180 deg. C. (350 deg. F.)

(b) Brushing Road Surface—per 100 sq. metres:—

(i) Water-bound macadam, kankar, laterite or other open textured surfaces 1½ mazdoors

(ii) Old bituminous or tarred surfaces ½ ,,

(iii) Preparation of surface for 2nd coat painting ½ ,,

(c) Heating and Spraying Surfaces with Tar Boilers: per 100 sq. metres:—

(i) Spraying 1½ mazdoors

(ii) Spreading stone chips 1½ ,,

Sundries required for road works

Baskets, buckets, pick axes, forks or rakes, ropes, wire brushes, soft brushes, gunny bags, empty tins and drums for water, tampers, pouring cans, road template or camber board, measuring boxes, weighing machine, gauge screens, thermometers, caution boards and barriers, watchman lantern, soap, etc.

Painting with Bitumen Emulsion

Emulsion is generally used for works under damp conditions and for roads with medium density rubber-tyred traffic, where two coats are given, and for light density traffic where only one coat may suffice. Emulsion is not suitable for heavy loading.

It is generally advisable to apply two coats of an emulsion rather than one, especially when the surface to be treated is open, *e.g.*, a new water-bound surface which has not been surface dressed before, as one coat will not deposit sufficient emulsion to fill the interstices between the stones. Single-coat surface dressing with emulsion is recommended only for renewal coats on black-top surfaces where only a thin coat of bitumen is required. It is important not to use too little emulsion, and if in doubt better to use more than less. There is not much risk of "fetting up" with emulsions, but a deficiency may bring about premature wear of the road surface.

Emulsion may be applied either by machine or by hand. Mechanical pumping causes premature "breaking" of the emulsion, and only vacuum or pressure pump are therefore suitable. For small areas hand methods are best. When sprayers are not available special pouring cans fitted with baffle mouth pieces are used. Pouring cans can also be made out of kerosene oil tins. Perforations are made on the upper half of the side opposite to the one to which a handle is attached so that when the can is filled with emulsion up to the bottom row of the perforations the contents will be about 9 kg (2 gallons). (See also under "Method of Application of Binder".) Brushing helps emulsion to "break" through agitation.

Chippings must be spread immediately after the emulsion has been poured and before it starts to break, i.e., changes colour from brown to black. But if coarse sand or chippings containing a high percentage of fines is being used, the breaking of the emulsion must be allowed to start before the covering material is spread.

Where the road surface is very dry and dusty, it should be very lightly sprinkled with water just before applying the emulsion.

Emulsions do not spread a thick layer of bitumen and cannot take bigger chippings than 6 mm for dressing on a previously treated surface.

According to IRC "Specification for Road and Bridge Work"

Binder	Stone Chippings	
(i) Single coat or the first coat of two coat surface dressing		
180 kg	1.50 cu. m	12 mm nominal size
(ii) Second coat of two coat surface dressing or renewal coat		
110 kg	1.00 cu. m	10 mm nominal size

12 mm nominal size is wholly passing through 20 mm sieve and retained on 10 mm sieve.

10 mm nominal size is wholly passing through 12.5 mm sieve and retained on 6.3 mm sieve.

Where surface dressing in two coats is specified, the second coat shall be applied immediately after laying of the first coat.

Specification for Bituminous Surface Dressing Using Precoated Aggregates (Based on IRC : 48—1972)

This consists of painting the prepared road surface with a binder, spreading precoated aggregates over it and rolling. It is more or less like the conventional surface dressing as described earlier except that a smaller quantity of binder is used. This may be done in single or two coats.

The size of the aggregates recommended is 12 mm nominal size (wholly passing 20 mm sieve and retained on 10 mm sieve) for the first coat and 10 mm nominal size (passing 12.5 mm sieve and retained on 6.3 mm sieve) for the second or renewal coat. The aggregates are precoated with 0.75 to 1 per cent of its weight of binder. The precoated aggregates are cured for at least one week before spreading on the road. The aggregates are preheated to 60 deg. C. for precoating except for cutbacks, and then mixed with binder heated to its application temperature.

After the road surface has been prepared, the binder heated to the appropriate temperature is sprayed uniformly over the dry surface.

Generally bitumen 80/100, tar RT-3 or RT-4 are used. Immediately after application of binder, precoated aggregates are spread uniformly so as to cover the surface completely. Before commencing rolling the surface should be broomed with a view to ensure uniform coverage of aggregates.

First Coat. The rate of application of the binder is 150 to 175 kg per 100 sq. metres of road surface for bitumen and 175 to 200 kg for cutbacks and road tars. Precoated aggregates of 12 mm size at the rate of 1.4 to 1.5 cu. metres per 100 sq. metres are spread uniformly over the coated surface.

Second or Renewal Coat. The rate of application of the binder is 94 to 108 kg per 100 sq. metres of road surface for bitumen and 108 to 132 kg for cutbacks and road tars. Precoated aggregates of 10 mm size at the rate of 0.9 to 1.1 cu. metres per 100 sq. metres spread uniformly over the coated surface.

Thin Sand-Asphalt Surfacings on water-bound macadam road with worn out single coat surface dressing of about 4 years old. (Extracts from IRC Road Research Bulletin No. 13.) Experiment carried out on National Highway—Delhi Mathura Road—which carries a heavy intensity of traffic of the order of about 5000 tonnes per day. Presents the case for the adoption of such surfacings where the aggregates are costly and good sand is locally available.

Any sand passing No. 4 BS sieve (4.75 mm) and reasonably coarsely graded was found suitable. The mixes were hot mixed and hot laid. Mixes were prepared in a 5-tonne capacity Spot mix in a central place. Sand was heated to 150-160 deg. C. in the hot bin while the binder was heated to a temperature of 160 to 175 deg. C. The heated sand and binder were mixed in the pugmill. The filler was added to the pugmill before the incorporation of the binder. The hot mixture was carried to the site by a trailer.

After cleaning the old road surface so as to be free of dust, etc., a tack coat of 75 kg/100 sq. m with the same binder as in the mix was sprayed on the surface uniformly.

The hot mix was dumped in the surface between two batons 25 mm high and 3 m long and spread manually with rakes and screeds to a uniform thickness of 25 mm loose. The compaction was done while the mix was sufficiently hot by 2 to 3 passes of a 8-10 tonne roller.

An evaluation of the various surfacings after a period of 3-4 years indicates that almost all the sand-bituman surfaces using 80/100 bituman have

Possible Causes of Unsatisfactory Asphaltic Concrete Surfacings

From : "Bitumen Road Construction" Burmah-Shell

Possible cause of difficulty \ Type of Difficulty	Mix slipping on base	Tearing of mix	Crushing	Long deep cracks	Fine haircracks	Pushing and wavy finish	Roller marks	Bad joints	Ravelling	Rough surface	Brown dead appearance	Fat surface
Lack of tack coat	X					X						
Tack coat not used	X					X						
Excess tack coat	X					X						X
Excess fines	X				X	X	X					
Excess coarse aggregate		X	X		X			X	X	X		
Lack of bitumen		X			X				X		X	
Excess bitumen	X					X	X					X
Bad proportioning	X	X	X		X	X	X	X	X	X		X
Poor batches in load						X			X	X	X	X
Excess moisture in mix	X					X					X	
Mix too hot		X					X				X	
Mix too cold		X						X	X	X		
Poor operation of finisher		X				X		X	X	X		
Inadequate rolling	X				X		X	X	X	X		
Over-rolling	X			X	X							
Rolling too hot	X				X	X	X	X	X	X		
Rolling too cold								X		X		
Rollers too heavy	X		X	X	X	X	X			X		
Unstable base	X	X		X	X	X				X		
Segregation in laying		X						X	X	X		
Finisher going too fast		X								X		
Mix laid too thick						X						
Traffic allowed on too early						X						

behaved very satisfactorily. The surfaces have been well kneaded by traffic, without any shoving or waviness.

The viscosity of the bituman used plays an important role. Generally mixes laid with high viscous binder such as 60/70 bitumen did not get compacted much under traffic and ultimately did not give good performance even though they gave high stability values in the laboratory tests. Because of the fact that mixes being stiff got pushed instead being kneaded by traffic with resulting waviness of the surface.

Filler—Silt as filler behaved extremely well. The only precaution to be observed is that it should be thoroughly dried and should be pulverised before using so that uncoated clods are avoided in the mix. Mixes with lime as filler behaved better than mixes with cement as filler.

Properly designed thin sand-asphalt could economically be substituted for conventional premix carpet and surface dressings especially where stone aggregates are not available within economical reach.

17. CEMENT CONCRETE ROADS

Foundation Under Concrete Pavements

It is very essential to have a good solid foundation of well consolidated and non-absorptive materials under a concrete road. A concrete slab, being a rigid type of construction, is liable to crack over a yielding and flexible base when loaded. The most important property required of a base or subgrade is its equal bearing power all over and not its actual bearing value. The load-carrying capacity of a concrete road structure lies mainly in the structural rigidity of the slab and the uniformity of subgrade support. It is therefore necessary to prepare the base in such a way that the concrete slabs are supported as uniformly as possible. A concrete slab will not overcome deficiencies in the subgrade or base except a little bridging action. Subgrade failure is a common cause of cracking and subsequent tilting or disruption of the slabs. (Base, sub base or subgrade together act as a unit for supporting the concrete pavement.)

Except for water-bound macadam, the base of concrete pavement should be made smooth before laying the concrete so as to reduce the co-efficient of friction between the concrete slab and the base. Concrete contracts while setting, therefore, a newly laid slab must have free movement while drying. There will also be subsequent expansions and contractions due to temperature changes and, as a consequence, the slab will tend to slide over the base. This horizontal movement of the slab is restrained if there is friction between the slab and the base.

The width of the base should project out at least 30 cm beyond the proposed edge of the concrete slab if kerbs are not set on the slabs.

The base under the concrete pavement must be properly levelled, hollow patches filled up and consolidated with hard core, cambered and cross-falls or longitudinal slopes given. There should be no soft spots present either in the base or the subgrade.

For concrete pavements the minimum modulus of subgrade reaction of the under-surface (base or subgrade) obtained with a plate bearing test should be 5.5 kg/cu. cm. Where the type of soil in the formation of the road is of this quality, no intermediate sub-base need be provided. The top 15 cm should be thoroughly compacted at the minimum moisture content to the exact profile of the road.

Where the natural subgrade is weak but it does not consist of highly expansive soil such as black cotton soil, and is not impregnated with deleterious salts, the thickness of the consolidated base need not be more than 15 cm. Where lime-puzzolana concrete or lean concrete is proposed to be used, the thickness then may be only 10 cm.

The base can be one of the following :—

(i) One layer of water-bound macadam over a layer of flat brick soling with joints filled with sand. (A minimum depth of 15 cm of metalling or 22.5 cm thickness of metalling and soling combined.)

(ii) Two layers of water-bound macadam. This may be of either stone, hard kankar, dense blast furnace slag, jhama or over-burnt brick aggregate, or any other granular materials which are not likely to soften under the action of water, well consolidated.

When a concrete pavement is laid over a water-bound macadam surface, the concrete slab is bonded with the rough and uneven surface of the water-bound macadam base. To obtain a good bond, the surface of this base should be well cleaned with wire brushes to a depth of about 15 to 20 mm so as to remove all loose material; the protruding metal will provide a good key. The cleaned surface of the water-bound macadam may be given a wash of cement slurry prior to laying concrete (2.75 kg of cement/sq. m).

10 cm thick concrete slabs laid over 15 cm well consolidated existing water-bound macadam crust have been found adequate to stand an axle load of 8100 kg (or single wheel load of 4100 kg) and heavy cart traffic, while 7.5 cm thick slabs have stood traffic intensity of 1400 to 1500 tonnes. Only some of the slabs developed cracks near joints and edges. Theoretically, 7.5 cm thickness is too small for this intensity of load but the endurance of these slabs may be due to the bonding of the slabs to the existing road which virtually increases the thickness of the slab, and also due to the higher-modulus of subgrade reaction of the thick and consolidated old road crust. These slabs are without reinforcement.

A kankar subgrade under heavy traffic is not satisfactory. Kankar disintegrates under heavy traffic and there is a certain amount of pumping action which slowly creates hollows, particularly near joints, and thus cracks begin to develop in the road slab. Experiments, however, have shown that where the concrete was well bonded into the existing kankar road it stood very well.

(iii) Well-graded soil — gravel mixtures.

(iv) Soil, moorum or laterite stabilised with lime or cement, where necessary, to give a minimum soaked CBR of 50 after 7 days curing. (This can be normally achieved with 3 to 4 per cent cement or lime.)

(v) A lean concrete mix of 1:4:8 or even 1:5:10, or lime puzzolana concrete. The 28 days compressive strength in the field of such concrete should be in the range of 40 to 60 kg/sq. cm.

(vi) Rolled or grouted cement concrete, as described in the pages following, can also be used for base course over clay soils.

No base course is required on subgrades of gravel, sand, or gravel-sand-clay which can be thoroughly compacted. Such subgrades should be well rolled and levelled and either sprayed with bitumenous emulsion and blinded before the concrete is laid, or covered with water-proof paper to prevent loss of liquid into it.

Where the natural subgrade soil is impregnated with deleterious salts in injurious amounts, the base should be laid over a capillary cut-off.

Where the natural subgrade consists of highly expansive soil such as black cotton soil, the base should be of the same specifications as given above, but should be laid over a 15 cm blanket course consisting of either (a) Black cotton soil stabilised with (about 4 per cent) lime; or (b) Local moorum, treated with lime (4 per cent); or (c) Local sand or non-plastic ungraded fine aggregate such as kankar, shales, laterite materials.

Where the action of salts is considered to be not very severe, sulphate resisting cements or Portland blast furnace slag cement with puzzolanic admistures such as burnt clay puzzolana may be used. Chemical attack on concrete in roads is not normally serious enough to warrant particular precautions.

To prevent the movement of moisture from the subgrade to road pavement, in areas of high water level, capillary cut-off comprised of sand, fine gravel or bituminised materials is required to be provided. The top of the base should not be less than 30 cm above the anticipated water level in the side drains.

Where the natural subgrade consists of highly clayey or silty soils (very weak and plastic) but not highly expansive soils (such as black cotton soil) the base should consist of the same specifications as given above, but laid over a 10 to 15 cm blanket course. To avoid surface of the pavement or its constituent layers going wavy on account of the combined effect of poor supporting powers of the subgrade and working up of the wet soil into the large voids of the water-bound macadam or stone soling during wet season, a blanket course is recommended in addition to the usual thickness of the base course. The blanket course may consist of either (i) Local soil stabilised with about 3 per cent lime or cement, to attain partial resistance to softening effect of water, or (ii) Local sand or non-plastic ungraded fine aggregate such as kankar, shales or any other granular material. The blanket course (sub-base) should be laid over properly compacted subgrade to give uniform support to the road slab.

The economy of providing a base under a concrete road slab should be weighed against the thickness of the slab. (See under "Design of Pavements", and CBR values). The provision of a base will not result in much increase in the strength of the road structure. A base has to be provided to form a working surface on clays, silts and sandy clay soils and other weak soils, and to provide a levelling course on roughly shaped formations. A layer of lean concrete may also be necessary on old macadam roads for re-shaping the surface. In such cases a base need only be thick enough to provide a smooth, hard working face over the formation. A subgrade of hard rock need also be covered with a layer of broken stone or gravel. When a base is considered necessary, it is seldom worth laying a thickness of less than 7.5 cm, and except on poor soils, a base thickness of 7.5 cm is usually adequate.

Concrete should not be laid on black-topped surfaces having soft spots caused by excessive bitumen or where thick premixed carpets have been rutted under traffic. In such cases the entire surfacing material should be removed up to the top of the compacted macadam surface.

The subgrade is prepared and checked at least two days in advance of concreting. The under-layer should be in wet condition but not muddy at the time the concrete is laid. If necessary, it may be saturated with water the night before or, not less than six hours, not more than twenty hours in advance of placing concrete. If the surface becomes dry, it may be sprinkled with water again. Or the surface be insulated with bitumen emulsion or covered with water-proof paper, where it is not intended to bond the concrete slab with the base. A clay must not be too wet when it is covered.

Smooth surface is compacted sand, gravel and clinker, stabilised soil, rough foundation covered with water-proof paper. *Rough surface* is water-bound macadam, soil gravel mix, rolled lean concrete, etc.

Water-proof Layer. It is desirable to lay a layer of water-proof paper whenever concrete is laid directly over soil subgrade. Such an arrangement reduces friction between the slab and the under-surface. The use of water-proof paper is particularly desirable for slabs laid in hot weather when considerable contraction may take place before the full strength of the concree has developed. A layer of about 20 mm of sand may be laid over the old road surface and wetted, or a coat of bitumen given over a treated surface before laying the slab so that there is no bond between the newly laid concrete and the old surface, and the slab is free to move. Another advantage of a water-proofing layer between the slab and the subgrade is, loss of cement paste to a porous base or subgrade is prevented. The interposed layer will also prevent any harmful salts present in the subgrade rising up and attacking the concrete slab.

Concrete road slabs can usually be laid in single layers if good aggregate is available to produce hard and dense surface and where the traffic is not very heavy or the thickness is not too great to produce any difficulty in ramming.

Where good hard agregate is not available or is not economical, the road can be laid in two layers using hard aggregate for the top course which can be 25 to 50 mm thick. The bottom layer can be of 1:2:4 and top layer 1:1½:3, or 1:2:5 if a roughened surface is desired. For top course rounded aggregate are not suitable but hard crushed stone should be used in proper proportions. The upper layer should be laid 6 to 12 mm higher than the profile to permit of its being rammed into position with the tamper.

Where the road is laid in two courses, the top course should be laid within 30 minutes after completion of the bottom course and before the bottom course has set so that a good bond between the two layers is obtained.

Slabs with thickened edges have no advantage over slabs of one uniform thickness, but on the other hand, are rather unreliable since the subgrades

cannot always be perfectly shaped and compacted to the exact sections of the slab to be laid over, thus giving possibility of settlement and cracking.

While designing intersections, corner angles less than 90 deg. should be avoided as slabs are particularly susceptible to corner cracking; stress in an acute-angled corner is higher than that in a right-angled corner and more so in the case of slabs less than 15 cm thick. The stress at a corner of 50 deg. is about double the stress at a right-angled corner and proportionately so with other angles. Where acute-angled corners are unavoidable, such corners should be strengthened either by increasing the slab thickness at these points, or by using heavy reinforcement or by both.

A plain concrete slab in which the dimension in any direction exceeds 4.5 metres is liable to crack. Where the slabs are reinforced longitudinally and transversely with equal amount of steel, widths can be increased up to 7 metres.

All the arrises of cement concrete slabs should be chamfered at 45 deg. for 38 mm width and rounded. All manholes and gullies should be surrounded by wooden boxes during concreting and filled round later.

Slab Thickness

There are many variables such as allowable flexural stress in concrete, co-efficient of subgrade reaction or the subgrade support, wheel load, temperature variations, co-efficient of sub-grade resistance, i.e., the friction between the slab and the subgrade, distance between joints, that must be considered in designing payment thickness and many of these variabies cannot be evaluated precisely, therefore, the design calculations give more or less approximate values, and reliance should be placed on previous experience.

Wheel loads cause flexural stresses in the concrete slabs and they fail by tension rather than by compression. Repetitions of wheel loads produce fatigue in the concrete. Wheel loads (dynamic) as they pass over joints create an impact effect and their static value is increased by about 25 to 50 per cent.

Stresses due to wheel loads are higher at the edges and corners of the road slabs than at the centre. The most satisfactory method of strengthening the edges forming the side of the road is to extend the slabs under the kerb. A 25 to 30-cm extension on each side of the road is adequate; this will prevent the occurrence of high stresses due to traffic running along the edge of the slab and will also provide a foundation for the kerbs. The slab surface under the kerbs should be roughened and provided with a concrete backing or the kerbs be securely fixed by short dowel bars fixed in the slab and keyed into the kerbs, to prevent their being displaced by traffic. This arrangement has the advantage that a vertical joint is avoided at a position where the risk of water reaching the sub-grade is greatest. Where such an arrangement is not practicable, the sides of the road slab can be strengthened by: either (i) thickening the outer edges of the slab, or by (ii) constructing a concrete beam under the slab at the sides of the road, or by (iii) providing extra reinforcement at the sides.

Where concrete slabs are less than 15 cm thick or where the slabs are on a poor foundation, it is advisable to strengthen their corners by means of "hairpin" reinforcement as described under "Reinforcement" in the following pages.

The following thicknesses are recommended for Concrete Road Slabs in IRC: 15-1970.

Average daily number of commercial vehicles (with laden weight over 8 tonnes) both directions anticipated to develop in the next 20 years	Thickness of slab
Not exceeding 200	15 cm
Over 200 and up to 350	17.5 cm
Over 350	20 cm

Description of subgrade	Thickness of base (cm)	Thickness of slab for traffic intensity (cm)					
		Very heavy	Heavy	Medium heavy	Medium light	Light	Very light
Very stable	None	23	20	18	15	13	10
Normal	8	25	23	20	18	15	13
Unstable	up to 15	28	25	23	20	18	15
Reinforcement min.	kg/sq. m	7.6	5.4	5.4	3.8	3.8	2.7

Very stable subgrade is undisturbed foundations of old roads, solid rock, compact well graded gravel. Unstable subgrades are susceptible to non-uniform movement and comprise of organic or highly plastic clays or peat. Also subgrades formed by embankments exceeding 1.5 metres in height and those in which the level of water table may rise within one metre of the foundation. (Also see under "Design of Pavements" — Description of Subgrades.)

For the design of rigid pavements, the stability of the sub-grade that is, uniformity of sub-grade support, is more important than the thickness of the pavement. Tests which measure the strength of the sub-grade soil are not of much value in designing concrete road slabs.

If a concrete road fails repair is often very costly, therefore it is advisable to keep a slightly greater thickness of construction than would otherwise be considered necessary. As increase in slab thickness will result in a percentage increase in structural strength of about double the percentage increase in the cost of the slab.

Reinforcement in Concrete Road Slabs

The normal form of rigid (concrete) pavement is without reinforcement. Because of its rigidity and beam-action it distributes the wheel load over a greater area of subgrade than does a flexible pavement of the same thickness, but it does not adjust itself to minor settlements that may occur beneath it.

Reinforcement is not usually necessary where the slab is laid on firm and well consolidated foundation. Reinforcement helps to carry over any weak places in the subgrade, and is usually provided under heavy loads only. Reinforcement does not materially increase the load carrying capacity of a properly supported concrete slab but it does increase the resistance of

the concrete to cracking and it controls any cracking that may occur by preventing the cracks from opening further. Where the slabs are provided with proper joints and are well supported, the reinforcement has little function. Reinforcement may therefore be used only on doubtful subgrades.

Experiments carried out at the Road Research Laboratory, Harmondsworth (England) have indicated that the maximum tensile stresses occur at the top of a slab at the corners and at the bottom elsewhere in the slab, and they consider it a better procedure to place the reinforcement at the top if it is laid in a single layer. But if corners can be reinforced, the main reinforcement may be at the bottom. Thin slabs reinforced at corners will take heavier loads. Another point in favour of the top reinforcement is that the sun shines on the top of the slab and not the bottom, the top tends to expand and produce tension while it is heated, when the bottom remains cooler. Reinforcement is placed about 5 cm below the top of the slab.

If the subgrade is weak, the reinforcement must be laid in the bottom and about 5 cm above the bottom line of the concrete, but if the subsoil is unreliable as regards its settlement, and temperature stresses are expected, a double layer of reinforcement should be provided. Reinforcement should be equal in both the layers. Slabs 15 cm thick or less need be provided only with one layer of reinforcement. There is no evidence to show that better performance is obtained with a double layer of reinforcement than with a single layer (of the same total weight). The use of two layers of reinforcement impedes construction, especially where the concrete is being spread and compacted by power-propelled machines. It is therefore recommended that a single layer of reinforcement should be used except where steel weighing 7 kg/sq. metre or more is required, or for positions stated above. It is not economical to use adequate steel to increase appreciably the structural strength of the slabs.

The amount of reinforcement per each layer is about 3 kg/sq. metre for moderate traffic, and about 5 kg sq. metre for heavily trafficked roads, with the greater proportion placed in the longitudinal direction.

Reinforcement for concrete pavements is most often of the bar-mat type or the welded-wire-fabric type. These are made up in sheets 10 to 20 cm narrower than the width of one lane and length according to spacing of the transverse joints so that reinforcement will stop short not less than 5 cm nor more than 10 cm of all edges and full depth joints in the slab, but it is often continued across dummy groove joints to serve the same purpose as tie bars.

Bar-mat reinforcing is often made up on the job, the bars being wired together at each crossing point. As far as possible the reinforcement should be in the form of flat mats. Where the reinforcement is delivered at the site in rolls of welded wire fabric, it should be seen that it does not take a curved shape when laid Spacing of bars across the width of the slab may be uniform, or more steel may be concentrated near the edges. Longitudinal bars of 10 mm diameter are generally spaced about 30 cm on centres. Transverse bars are often the same size as the longitudinal bars, but are spaced about twice as far apart, or may be 6 mm diameter spaced

50 cm. Where the transverse joints are at about the width of the slab, reinforcement should be equal on both sides.

"Hairpin" reinforcement is recommended at the corners. This consists of two lengths of bars, one bent with two legs at right angles and the other with the two legs forming an angle of 30 deg. Each leg is 1.25 to 1.52 metres long and 16 mm diameter, and hooked at ends. Another method is to bend two 2.44 to 3.05 metres long bars at 60 deg. Sometimes two straight bars are placed at the corner at right angles and a third bar at 45 deg. If the main reinforcement is not placed at the top then the corner reinforcement should be 5 cm from the top of the slab, otherwise at mid-depth.

The amount of longitudinal and transverse steel required per one metre width or length of slab can be computed by the following formula:

$$As = \frac{L f W}{2S}$$

Where:

As=area of steel in sq. cm required per metre width or length of slab (measured at right angles to the bars);

L=length of slab between joints or edges in metres;

f =co-efficient of friction between slab and subgrade, or subgrade resistance — value usually ranges from 1 to 2, with 1.5 as the average which is most commonly used;

W=weight of slab per sq. metre in kg— assumed as 2400 kg/cu.m, or 24 kg/sq. m per cm of slab thickness;

S =working stress in the reinforcing steel in kg/sq. cm—usually taken as half the yield stress (will be different for mild steel bars and cold drawn steel wire fabric).

The formula can be applied to both longitudinal and transverse steel reinforcements, which will be in the same proportions as the longitudinal and transverse dimensions of the slab.

Aggregates. The aggregate should be of the hardest and toughest variety available. In plain single-course or for the bottom course of two-course work, the nominal maximum size of coarse aggregate may be up to one-third the thickness of the slab, with 63 mm max. size. But the size most commonly recommended is from 40 mm down to 10 mm for the base course. It is considered that the coarse aggregate should not be less than 40 mm otherwise it would not take the load. For the top course the size should not generally be greater than half its thickness; 20 mm is the common size with 40 mm max., graded down to 4.75 mm. Requirement of surface texture may also sometimes effect the choice of size of coarse aggregate.

The aggregate should be as much cubical as possible in shape. Rounded aggregate in the surface, in course of time, tend to polish and produce a slippery surface.

The sand content should be as low as possible and should be lower for rounded than for crushed rock aggregate. Sand to all pass through 6 mm sieve and not more than 20 per cent shall pass through a 300 micron IS (52 BS) mesh sieve. Fine sand should be avoided as it will produce a smooth surface which is not desirable in road slabs.

Light and soft varieties of sandstones and limestones are not suitable for road work except for bottom courses where even brick ballast can be used if the traffic is not very heavy.

The usual mix proportions are 1:2:4, 1:2:3½, 1:1½:3 according to the strength or wear resistance desired. Aggregate gradations are given in the section "Reinforced Concrete".

Some gradings of coarse aggregates successfully used:

(a) 40 to 25 mm —2 parts; 20 to 10 mm — 1 part

(b) Mixed: 40% — 40 mm, 60% — 20 mm (nominal size)

(c) Mixed: 50% — 40 mm, 50% — 20 mm (nominal size) in proportions 1:2:3½

For gradation of coarse aggregates from 5 mm to 40 mm, the aggregates are collected in two size ranges, one below and the other above 20 mm size. When grading is from 20 mm to 50 mm the materials are collected in the size ranges of above and below 25 mm.

The concrete mix should be such that gives a minimum cube compressive strength of 280 kg/sq. cm on field specimens after 28 days. Where a heavy duty wear resistance is desired (such as steel tyred traffic), a concrete with a minimum field compressive strength of 420 kg/sq. cm at 28 days shall be adopted for wearing course.

The minimum cement content for the mix corresponding to 280 kg/sq. cm shall not, however, be less than 6.5 bags of 50 kg nett/cu. m of concrete and for mix corresponding to 420 kg/sq. cm, shall be 8 bags of 50 kg nett/cu. m of concrete. (IRC: 15-1970)

The *slump* of the concrete mix for pavements compacted by mechanical vibrations should be more than 25 mm and that by manual compaction (hand tamping) not more than 50 mm.

Forms

As far as possible side forms shall be of mild steel channel sections with a broad base (at least 200 mm) and depth equal to the pavement thickness. These shall have square ends and be provided with rigid locking device to ensure continuity of line and level. The forms should be supported firmly by stout stakes or pins about one metre distance, to hold them in position on a uniformly firm foundation.

Wooden forms, where proposed to be used, shall be free from warps and twists. Such forms shall have a minimum base width of 10 cm for slab thickness of 20 cm and 15 cm for slabs over 20 cm thickness. The thickness of wooden forms shall be at least one-third the depth and not less than 7.5 cm. Wooden forms are capped along the inner upper edge with 50 mm angle iron, well recessed and kept flush with the face of the form. These forms are held in position on the site by wooden stakes or steel pins 20 mm diameter suitably placed to prevent lateral displacement.

Wooden forms should be used for curves of less than 45 metres radius. For smaller curves, side forms of brick in clay plastered can be used.

Formwork shall be provided for all expansion, contraction, construction and longitudinal joints. Forms should be set in advance to a length sufficient for at least one day's concreting. Line and level pegs should be fixed with a dumpy level at 30-metre intervals on straights, at all tangent points, and at about 6-metre intervals on curves. Level pegs should also be fixed at all changes of gradient. All forms should be treated with oil on the inside face to prevent concrete from adhering to them. Forms should not be removed till after 48 hours of concrete laying.

Camber

Camber or cross-fall may be either curved or straight. It is difficult to obtain a curved surface with work done by vibrating machine. A cross camber of 1 in 60 to 1 in 72 may be given. In dead level stretches of road longitudinal cross-falls have to be provided at the edges to drain water into the gullies. (See under "Camber or Cross-fall").

Laying Concrete in Road Slabs

Two methods for laying are used — *(i)* Alternate bays and *(ii)* Continuous strips. The first method consists of laying a series of alternate bays, the intermediate and adjacent bays being laid after an interval of at least 48 hours. In the second method the slab is laid continuosly in strips between longitudinal joints. Ends of slabs should be painted with bitumen before the intermediate bays are filled in. When pavements are laid in series of bays their dimensions vary in lengths from 9 to 10 metres with 15 metres maximum, and widths not more than 6 metres. On steep grades the slabs may be laid in bays of about 4.5 metres lengths. Size of the slabs is fixed according to the joints — transverse and longitudinal. In alternate bay method fewer expansion joints can be provided.

Joints in Concrete Pavements

There are two categories of joints for concrete pavments: *(i)* Transverse joints, and *(ii)* Longitudinal joints. Joints are made at the time the pavement is constructed. Joints are formed by placing within the concrete strips of metal or wood, or impregnated fibre, of a thickness of the joint required, embeded close under the pavement surface.

Expansion joint spacing is designed based on the maximum temperature variation expected and the contraction joint spacing is based on the allowable tensile stress in the concrete during the initial curing period. The gap (or groove-width) to be left between adjacent slabs depends upon the spacings of the joints (length of the slab) and the temperature range to be expected. Greater the distance between the expansion joints, the greater is the width required of the gap for expansion. Wide joint gaps cause discomfort to the traffic and it is also difficult to keep them properly filled in when tye maximum opening occurs. It is therefore considered groove width should not be more than 25 mm.

Transverse Joints have two main classes: *(a)* Expansion joints, and *(b)* Contraction joints. Construction joints are also required but they are replaced by either *(a)* or *(b)*. These joints are made at right angles to the road length and extend the entire width of the pavement.

Contraction and Expansion joints are made in concrete pavements to keep the stresses caused by changes in the volume of the concrete due either to shrinkage during hardening and drying, or to temperature chages, and thus prevent the formation of cracks and ultimate failure of the slab. Shrinkage may be caused either by the initial heat of hydration of cement and subsequent cooling, or by changing moisture conditions through the depth of the slab as it dries out. Temperature changes may be seasonal or daily and there will usually be a difference in temperature between the top and the bottom of the slab.

Transverse Construction Joints should be made at the end of each day's run, or where unavoidable interruption of more than 30 minutes occur in the concreting operations. As far as possible construction joints should be located at the site of expansion or contraction joints and formed so as to make a slab at least 3 metres in length. Such joints are generally plain butt and extend to the full depth of the slab. When concreting is stopped only for a short while and before the concrete has hardened, the new and the old concrete should be thoroughly sliced together, otherwise the exposed face of the joint should be painted with bitumen or given a coat of limewash before concreting the ajdacent bay.

Transverse Expansion Joints permit increase in the length of a slab as its temperature rises. It has been explained in the section "Reinforced Cement Concrete" under 'Expansion Joints' that total expansion or contraction that would occur in a 30-metre length of a concrete structure with a rise or fall of 10 deg. C. will be about 9 mm. Expansion joints should be so spaced that they will permit thermal expansion over a range of temperature from the lowest at which any slab in a complete pavement is placed to the maximum liable to be attained. Roads constructed in winter months should have expansion joints at closer intervals than those constructed in summer months. There is complete separation between the abutting slabs. These joints are constructed at right angles to the centre line of the pavement and extend to the full depth and width of the pavement. Such joints are usually provided with dowel bars.

Expansion joints should also be provided at intersections of pavements with structures or other pavements. The joints filler may be assumed to be compressed up to 50 per cent of its thickness and therefore the expansion joint gap should be twice the allowable expansion in the concrete, the usual width is 20 to 25 mm.

Transverse Contraction Joints. The expansion joints discussed above allow the concrete to expand, but it is also neccessary to have contraction joints at intervals in between the expansion joints to localise and control transverse cracking of the slab indiscriminately when it shrinks during curing or with a fall in temperature. Contraction joints are plain butt joints (like the expansion joints) with full-depth break in continuity of the slab, or "dummy" groove joints. These are most often the dummy type.

Spacing of Joints

No definite rules for the spacing of transverse joints can be given as the spacing depends upon many variable factors such as co-efficient of thermal expansion of the concrete—temperature variations, the temperature during placing, the frictional resistance of the subgrade to the movement of the slab, the thickness of the slab, and the amount of reinforcement (if provided). Reinforced and thicker slabs need joints at longer intervals.

Safe spacing of joints increases with increase in surface roughness of the foundation in case of expansion joints, and decreases in case of contraction joints.

Joints are the weakest part of the road structure and the commonest cause of bad riding. The chief faults in joints are : (*a*) Difference of levels between two adjoining slabs; (*b*) Badly rounded arrises: (*c*) Depresson of the edges owing to the unskilled use of rounding tools; (*d*) Extruding filler. Another major cause for weakness at the joints especially the construction joints where the work is stopped for the day is that at the end of the day the work is usually hurried and not carefully done. Also the contractors are apt to scrape up all the unused concrete, including concrete that has partially set, and utilize it in completing the job. These faults will not only cause bad riding but will make for excessive wear and damage at the joints and reduce the life of the road.

Spacings of Joints in Concrete Road Slabs

Expansion Joints (25 mm wide)

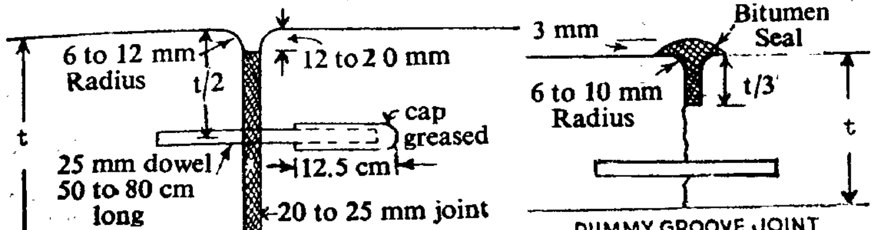
Period of Construction	Degree of foundation roughness	Max: Expansion joint spacing (m) Slab thickness (cm) 15	20	25
Winter (Oct.-March)	Smooth	50	50	60
	Rough	140	140	140
Summer (April-Sept.)	Smooth	90	90	120
	Rough	140	140	140

Contraction Joints

Slab thickness (cm)	Max: Contraction joint spacing (m)	Weight of reinforcement in welded fabric (kg/sq. m)
Unreinforced slabs		
10	4.5	—
15	4.5	—
20	4.5	—
Reinforced slabs		
10	7.5	2.2
15	13.0	2.7
20	14.0	3.8

I.R.C. Specifications

Weight of reinforcement is of welded fabric. Where the reinforcement is used in the form of mild steel bars, equivalent section areas corresponding to the sectional areas of the welded wire fabric should be employed.



DUMMY GROOVE JOINT

Dummy groove joints are made similar to other joints except that the break in continuity of the slab is only about 1/4 to 1/3 of the slab thickness and the lower solid portion which forms plane of weekness cracking during shrinkage of the concrete. Dummy groove joints have advantage over plain butt joints that they permit continuous construction and provide certain amount of anchorage below the groove which is useful for the transfer of loads across the joint.

Grooves left for dummy joints are only 6 to 10 mm wide according to the expected temperature ranges and the spacings of the joints (extending to 1/4 to 1/3 of the slab thickness).

Tongued and Grooved joints are sometimes made instead of butt joints to prevent one slab rising relatively to the adjacent slab and to enable transfer of load to take place, but the efficiency of these joints is doubtful. Tie bars are also fixed in these joints as in butt joints.

Transverse joints should always, if possible, be made at right angles to the edges of the pavement and continued straight across the carriageway. It has been observed that where transverse joints have been staggered on either side of longitudinal joint, sympathetic cracking has often occurred in line with the joint in the adjacent slab. But it is not a 'must'. Staggered transverse joints overcome the structural weakness caused by the intersection of four joints (four edges) and have been tried at some places with success where they were staggered about 25 to 30 cm.

Longitudinal Joints. Spacing of longitudinal joints is determined by the lane widths to be provided in the carriage-way. Where hand tamping or vibrating hand screeds are used, the width of longitudinal joints should be limited to 4 metres. Longitudinal joints are generally of the plain butt type. The faces of the old concrete slabs should be painted with bitumen before placing of fresh concrete. These joints should be keyed with deformed tie bars as explained under "Load Transfer Device", to hold the two slabs together.

Load Transfer Devices are usually provided at transverse expansion joints and sometimes at contraction joints of concrete road slabs to prevent the full effect of a wheel load from coming on to the end of a slab as the load approaches or leaves the joint to prevent relative movement and cracking of edges under impact and also to maintain vertical alignment of the adjacent slabs at the joint. Such devices are also used where concrete pavement slabs are laid on subgrades of doubtful or low bearing value such as peat, clay, fine grained soils and made ground, which are likely to sink under load. Load transfer devices are also required in roads of heavy traffic. The devices used for achieving load transference from slab to slab are dowel bars, dowel plates, or tongue and groove joints. Dowel bars are the most commonly used.

Warping Joints (also sometimes called hinged joints) are provided to relieve stresses induced due to warping. They are a form of contraction joints in which opening of the joint is prevented by carrying through the slab reinforcement or by inserting tie bars. Longitudinal joints with tie bars fall in this class of joint. If expansion and contraction joints are properly designed and constructed there is no need of providing warping joints in addition.

Contraction butt joints are used in alternate bay construction. Both faces of the concrete slabs should be coated with bitumen to prevent bonding. Contraction joints may be provided with smooth dowel bars if the spacing of such joints exceeds 6 metres.

Forming Gaps or Grooves for Joints. A (temporary) strip of wood or metal is embedded in the concrete during laying to form gaps or grooves for joints of thickness according to the widths required. Joints can also be formed by inserting the web of a 'T' — iron section with oiled surfaces; with handles provided at both ends. Such gaps are usually 20 to 25 mm wide full-depth break for expansion joints Also see under "Dummy Groove Joints" After the slab has been consolidated and finished the strip is removed. The arrises of the joints are carefully rounded (bull-nosed) to a radius of about 6 to 10 mm about half an hour after the final tamping of concrete when it has become rather stiff but before it has hardend enough to be unworkable (and after the gap forming strip has been removed). An edging tool made of steel plate may be used the vertical limb of which extends to the required depth. Final edging normally succeeds final belting or brooming. As soon as a joint is finished and before it is sealed, it should be tested with a 3-m straight edge so that it extends on both the sides of the joint. Arrises should be wire brushed before sealing to remove surface laitance and promote good adhesion of the sealing compound. It is advisable to use a mix slightly deficient in coarse aggregate near joints and edges to avoid risk of honeycombing at these positions.

"Sawn" joints are also used; these being cut 8 hours or more after placing of the concrete. This gives more regular riding surface across the joints.

Dowel Bars are usually smooth mild steel round bars of short lengths placed half-way down the depth of the slab, whose one-half length is bonded into the concrete of one slab and the other half is arranged to slip back and forth longitudinally in the other slab as expansion and contraction occurs. IRC recommends 25 mm diameter dowel bars of length 50 cm to be spaced at 20 cm for 15 cm thick slabs and at 30 cm for 20 cm thick slabs. In runways where the slab thickness is 40 to 50 cm, these bars may have a diameter of 40 mm and length 60 cm Dowels nearest the edges of the slabs should be placed closer together say, 25 to 30 cm from the ends.

The sliding or free end of the bar is fitted into a cap (also called sleeve or ferrule) of mild steel 3 mm thick plate or G.I. pipe of suitable diameter which gives a clearance of 3 mm, and about 12.5 cm long. Such dowel bars permit the joint to open and close but hold the slab ends on each side of the joint as nearly as possible at the same level. A thin coating of bitumen is given over the dowel bars.

Dowel bars prevent any one panel rising relatively to its neighbour and partly prevent warping and curbing. The load due to traffic wheels coming on to the end of each slab is partially transferred to the other slab thereby reducing the stresses.

Dowels should be distributed over the full length of the joint. Great care must be taken during construction to keep the dowels in straight alignment otherwise there will not be proper joint action — holding in position devices should be used.

Dowel bars do not provide any substantial advantage when thickness of the concrete slabs is less than 15 cm. It is also considered that dowels are expensive and lack in durability. Frequently, the dowels appear to bind in the concrete socket and become ineffective.

Dowel bars are not ordinarily provided for dummy contraction joints except when the joint spacing exceeds 6 metres for unreinforced slabs, but are provided in case of full depth construction joints for added safety.

Tie Bars are used across longitudinal joints, and sometimes at transverse joints, whenever it is necessary or desirable to tie together the two slabs or to ensure two adjacent slabs to remain firmly together in case opening of longitudinal joints is anticipated. The area of steel required per metre length of joint may be computed from the following formula:

$$As = \frac{bfw}{S}$$

where:

As = area of steel in sq. cm required per metre length of joint; b = distance between the joint in question and the nearest free joint or edge in metres; f= co-efficient of friction between pavement and the subgrade (usually taken as 1.5); W=weight of slab in kg/sq. metre; S = allowable working stress of steel in kg/sq cm (1400).

The length of any tie bar should be at least twice that required to develop a bond strength equal to the working stress of the steel. Expressed as a formula, this becomes:

$$L = \frac{2SA}{BP}$$

where:

L = length of tie bar (cm); S = allowable working stress in steel (kg/sq. cm); A=cross-sectional area of one tie bar (sq. cm); P=perimeter of tie bar (cm); B = permissible bond stress in (i) deformed tie bars—24.6 kg/sq. cm, (ii) plain tie bars — 17.5 kg/sq. cm.

For tie bars either deformed bars are used or they are hooked at the ends to have good bond in both the slabs. (Function of tie bars is different from those of dowel bars Tie bars are not for sliding.) The maximum diameter may be limited to 20 mm, and they should not be spaced more than 75 cm apart. The calculated length L may be increased by 5 to 8 cm to account for any inaccuracy in placement during construction. (IRC: 58-1974).

Typical Tie Bar details for Central Longitudinal Joint of Two-Lane Rigid Highway Pavements with a lane width of 3.5 metres

Slab thickness (cm)	Dia. (mm)	Max: spacing (cm)	Min: length (cm)	
			Plain bars	Deformed bars
15	8	38	40	30
	10	60	45	35
20	10	45	45	35
	12	64	55	40
25	10	30	45	35
	12	45	55	40
	14	62	65	46

IRC: 58-1974

Details of Tie Bars for Central Longitudinal Joints of Two-Lane Concrete Highway Pavements (IRC: 58-1974)

Slab thickness	Dia. (mm)	Max: spacing (cm)	Min: length (cm)	
			Plain bars	Deformed bars
15	8	38	40	30
	10	60	45	35
20	10	45	45	35
	12	64	55	40
25	10	30	45	35
	12	45	55	40
	14	62	65	46

The maximum diameter may be limited to 20 mm and they should not be spaced more than 75 cm apart. (IRC: 58)

Joint Fillers and Sealers for Concrete Roads

For filling joints in concrete pavements, either pre-moulded or poured types of fillers may be used. Pre-moulded fillers are compressible materials and consist of either bitumen-impregnated cellular rubber, cork, fibre board, or a strip of soft knot-free wood. Such a filler should be of thickness of full width of the joint gap — generally 20 to 25 mm, and 20 to 25 mm less in depth than the thickness of the pavement slab (extending up to the subgrade), thus leaving a slot below the top surface which is later sealed with a sealing compound. Holes for dowel bars, where provided, should be accurately bored or punched out in the filler board. During casting of the slab the pre-moulded joint filler is placed in position against the finished end of the concrete slab.

After the curing period is over, the joint portion above the filler board is cleaned thoroughly and the joint is filled with hot applied ceiling compound.

The most commonly used fillers are mixtures of asphalt of a penetration that, in the locality where used, will neither "bleed" in summer nor crack in winter (usually 20/30 penetration) and saw dust or chopped hemp or coir. After any of these materials is filled in the joint, heated asphalt is poured in. Sand is usually mixed with the asphalt and is also dusted over it after the joint has been filled in and before traffic is allowed on the road.

Other joint fillers used are:

(a) Sand 60 per cent; asphalt 30 per cent; saw-dust 7 per cent; cement 3 per cent.

(b) 80 kg of hot bitumen; 1 kg cement; 0.25 cu. m of coarse sand.

Joints should not be sealed while the concrete is still green or when it is damp. Joints should be filled up before the rains start and during winter, when the joints are widest, and should not protrude more than 3 mm on the surface.

The sealing compound shall be heated until it is fluid enough to pour easily into the joint. Rubberised compositions shall not be heated above 180 deg. C. The hot sealing compound is taken out in a pouring kettle having a spout and poured out into the joint opening in such a manner that the material does not spill over the exposed surface of the concrete. An extruding filler or excess material over a joint which cause jar or bump to the passing vehicles, can be trimmed off with a sharp-edged tool which may be heated to facilitate its cutting action. While the joints may be sealed flush with the adjacent pavement surface in summer, in winter they may be filled to a depth of 3 to 4 mm below the surface.

A primer is used to improve the adhesive bond between the sealing compound and the concrete, a low viscosity bituminous paint or a cut-back bitumen may be used. IRC recommend the following:

(i) Bitumen 180/200 penetration, 66 per cent by weight — blended hot.

(ii) Light creosote oil, 14 per cent by weight, blended hot or cold.

(iii) Solvent naptha, 20 per cent by weight, blended cold.

The bitumen is melted and fluxed with just sufficient creosote oil to render it fluid when cold. Solvent naptha is then added to bring the mixture to the consistency of paint. The primer is applied the thinnest possible. The primer, after application, is left for sometime till it feels "tacky", when the sealing compound is applied. Soon after the primer is applied, the joint is covered with 10-15 cm wide paper strips so that no dust is deposited on the primer. These particular primers are not suitable if a sealing compound is used.

Dry faces painted with kerosene oil also facilitate adhesion of molten bitumen. Bitumen emulsions shall not be used as primers.

When joints require replenishing of the sealing material, all loose and foreign matter should be removed from the joint by scrubbing with hand brooms of stiff fibre or wire brushes. Where the existing sealing compound in old joints has cracked, it can be removed with a raker. Primer shall be applied to the cleaned concrete faces as described above. Resealing is preferred in winter when the joints have opened up to the maximum.

Compaction: Tamping and Screeding

The surface of the concrete slab is brought to the specified contour or cross-profile by means of a heavy tamper or screed (also called strike board). A tamper is generally made of wood 7.5 cm wide and 23 cm high shaped to the surface cross-section of the slab. It is shod with a steel plate 3 mm thick, fixed to the bottom. The tamper is provided with two handles to enable tamping operations to be performed by two men in standing position. A tamper should weigh not less than 10 kg per running metre, and not more than 50 kg. Plain wooden sections will serve widths up to 4.6 metres, and if of greater widths, they will have to be braced with steel bars for rigidity. A tamper is made 45 to 60 cm longer than the proposed width of the pavement for resting on the forms. Where the pavement is to be laid in two bays, the tamper is made for the half road width and shaped accordingly. In two-course constructions a notched tamper will have to be made to consolidate the bottom course.

The tamper should rest on the side forms and should be drawn ahead with a sawing motion in combination with a series of lifts and drops alternating with lateral shifts of about 25 mm. At transverse joints the tamper should be drawn not closer than one metre towards the joints and should then be lifted and set down at the joint and drawn backwards away therefrom. In alternate bay construction the corners of slabs should receive special ramming with small rectangular hand rammers. Consolidation with tamping should be carried out up to the time when the mortar in the mix just works up to the surface under the hammering. Three to four hits of tamper should be enough to bring up the mortar. Too much tamping should be avoided. The loose concrete when laid on the road should protrude about 25 mm above the side forms. A tamper can also be made of steel of channel cross-section.

Hand Floating: After the screeding or tamping has been completed the surface should then be floated with a wooden float board 75 cm long and 7.5 cm wide. The float should be operated by a man sitting on a bridge made of a 4 to 5 cm thick wooden plank spanning the slab. The float should be worked with the long dimensions parallel to the centre line of the road, and it is drawn back and forth in slow stretches about 60 cm long and advancing slowly from one side of the slab to the other. This will produce a uniform even surface on the concrete free from transverse waves. When a length of about say, 10 metres has been tamped and finished it should be checked thoroughly with a straight edge and any differences of level over 3 mm corrected immediately before the concrete has taken its initial set. The floated surface should then be finished by belting or brooming.

Smoothing Board is generally made of hard wood about 4 to 5 metres long and 15 to 20 cm wide, supported on a frame fitted with plough-handles. This is sometimes used after the concrete has been tamped to produce a smooth close-knit surface and to remove any waviness that might have been caused by careless tamping. It should be used with a sawing movement, transversely and forward at the same time.

Straight Edge is used to check the finished pavement surface in longitudinal direction. It is made of hard-wood with steel plate at the bottom, 1.5 to 3 metres in length, 10 cm in width with two handles.

Where compaction is done by vibrations, the vibrations in the vibrator screed shall be of a frequency not less than 3500 impulses per minute. Internal vibrators should also be used for compaction of portion of the slab along the edges, corners and joints, and when slabs more than 20 cm thickness are laid. Internal vibrators and vibrating screeds have been described in the Section "Reinforced Concrete"

Too much compaction is bad. If excessive vibrations are given to a mix, cement mortar comes to the top making the surface weak which gets abraded under traffic exposing the aggregate making the surface rough and shabby.

Belt finish. Canvas belts are used for finishing the pavement surface before the concrete has hardened. The belt is made of two-ply stout canvas 25 cm wide and 100 cm longer than the width of the concrete slab, with wooden handles fixed at each end. It is worked with a combined longitudinal and transverse movement until a smooth surface is obtained but not so overworked as to cause laitance or an excess of fines to be brought to the surface. Final belting is done after the water-sheen has disappeared but before the concrete has set. Belting produces a gritty, non-skid surface. Brooming may be done instead of final belting.

Surface Finish

A smooth concrete surface is not suitable for animal drawn vehicles. To obtain an exposed aggregate finish - skid resistant rough surface, immediately after the surface has been smoothed off, it should be lightly brushed with a soft hair or rubber broom so as to remove all laitance and surface water, etc. After an interval of 12 to 24 hours (depending on the temperature) when the concrete is sufficiently hard to prevent displacement of the aggregate, but not yet hard enough to prevent the removal of a certain amount of mortar, the surface should be well brushed with a hard fibre brush to expose the aggregate. Work is helped if some water is added while brushing. It will leave the stones slightly "proud" on the surface, thus giving a non-skid smooth riding surface. Scorings should not, however, be over 1.5 mm in depth. A hessian cloth drawn over the surface will also make the surface non-skid.

The brush broom is 45 cm long and 7.5 cm wide, with a handle which should be at least 30 cm longer than half the width of the slab. At least four brooms should be provided on the job. The brooms are drawn transversely across the slab standing either on platform bridging the slab or across half width from either side of slab.

Curing

After the concrete has sufficiently set, it should be covered with wet gunny sacks or canvas, or wet hay, for about 24 hours and must be protected from drying by the direct rays of sun or high winds. After 24 hours the concrete should be kept wet either by ponding or by a cover of wet sand or earth not less than 7.5 cm thick and continued for about 14 to 21 days. Curing can also be done by proprietary membrane curing compounds such as calcium chloride or sodium silicate, or by spraying the surface with cold bituminous emulsion immediately after the initial set has taken place. Such methods are very useful in water scarcity areas. Covering the surface with water-proof paper is very effective. (See also under "Reinforced Concrete")

Correcting Slippery Surfaces. A concrete surface which has become polished and slippery by traffic is a source of danger particularly at steep gradients and turns, and especially when wet. Skidding properties can be removed by the application of dilute hydrochloric acid to the surface leaving it for a few minutes, scrubbing the surface with stiff brushes and finally washing with fresh water. A solution of 1 part of concentrated hydrochloric acid and 1 part of water is applied at the rate of 1 litre of solution to about 4.5 sq. metres of surface. More than one treatment may be given if necessary.

Hardening Concrete Surfaces—See Section 8. See page 18/16

Opening to Traffic

No traffic should be allowed on the finished surface till after 28 days of its completion, where ordinary cement has been used, and 7 days where rapid hardening cement is used. (Where hardcore has been consolidated for the sub-grade or soling has been provided it is advantageous to allow traffic on it, if practicable, for about a month.)'

Repairs to Concrete Roads

Damage to a concrete road may be caused by abnormal loading, excessive expansion, defective drainage or washouts in the subgrade, or inadequate support owing to its settlement. Access of water to the subgrade through joints and other cracks, is the commonest cause of failure in a concrete road. Before repair is carried out, the actual cause of the damage must be determined and necessary steps accordingly taken to cure any basic defect in the subgrade drainage system or design of the pavement.

The replacement of the damaged portions of the concrete slab is only necessary when the damaged area is displaced or when the broken pieces are too small to distribute the load on to the subgrade without settlement or rocking.

Scaling and Crumbling of the surface of the concrete slab is generally due to poor materials or workmanship. The most satisfactory and convenient remedy is bituminous surfacing or patching. No doubt, bituminous patching is very unsightly.

Joints, cracks and small pot-holes should be filled in with hot bitumen with coarse sand after they have been thoroughly cleaned. All dirt, loose mortar and dead filler must be removed using a wire brush or compressed air jet. Fine cracks do little harm and are best left alone unless it is apparent that they extend right through the pavement or their sides are weak, in which case they should be opened out, otherwise no attempt need be made to fill them with bitumen which will not penetrate into hair cracks. Those wide enough to admit grit or debris should be thoroughly cleaned and sealed.

Shallow patches not exceeding 12 mm can be repaired by coating the surface with a cutback asphalt or emulsion, after they have been cleaned and well brushed, and covering with coarse sand or finely crushed stone screenings. Ligger depressions can be patched with premixed bitumen and stone grit of size 3 mm to 10 mm after the surface has been cleaned and primed.

Where the slab is badly cracked, it should be broken into small pieces, compacted thoroughly, and grouted with bitumen.

Patches or pot-holes exceeding 25 mm in depth may be repaired with concrete. The damaged portion should be cut out to a rectangular shape. The faces of the old concrete should be thoroughly cleaned and wetted, and brushed with neat cement grout. The new concrete should be, as nearly as possible, of the same mix as the old one and mixed to the stiffest consistency that will permit of thorough tamping and ramming

The place to be patched should be completely filled with concrete, slightly proud (say about 6 mm) of the existing adjoining surface, and tamped. After an hour or so this concrete should be retamped to take up any initial shrinkage. The patch should be thoroughly cured before traffic is allowed on it. A cement concrete mixture gauged with lime (1 cement: 1 lime) may also be tried.

Where a cement concrete road slab shows signs of wear all over in the form of cracks of varying stages because of its inability to withstand the heavy repetitive wheel loads, it is best to put on a thin carpet of asphaltic concrete for strengthening the existing road. As a result of experiments carried out by The Central Road Research Institute, New Delhi, the following conclusions were arrived at, as reported in the IRC Bulletin No. 14.

(i) For a traffic of the order of about 3000 tonnes per day, a 6 in. (152-mm) thick water-bound macadam topped up with either 3/4 in. (19 mm) premix with seal coat, or a $1\frac{1}{2}$ in. (38 mm) asphaltic concrete, gave satisfactory performance. A tack coat between the base concrete slab and water bound macadam layer is a must.

(ii) A 4 in. (102 mm) layer of well graded asphaltic concrete is minimum as an overlay. For heavier loads the thickness of asphaltic concrete may be increased.

Where the slabs have settled due to defective subgrades, the same can be remedied by grouting under pressure through a series of holes drilled not exceeding 40 mm diameter, with cement and sand grout of the consistency of thin cream, or bituminous mixture. This will fill up the voids between the subgrade and the slab and will also raise the slab up.

Joining Concrete Roads with Bituminous Roads. Adhesion between bitumen and concrete is very poor. The best method is to lay a double course of stone setts laid on concrete, since bituminous material will adhere to stone and the concrete can be made to do so by grouting. To ensure a good bond the pavement slab should be roughened where necessary by narrow grooves cut in a herring-bone pattern.

Foot Paths

A finished thickness of 50 mm cement concrete 1:2:4 slab (cast-in-situ or pre-cast) laid over 50 mm of lean concrete, say, 1:6:12, over well consolidated subgrade, and sand filling, will meet most of the requirements of foot-paths. The top slab may be laid while the base is still green. On heavily trafficked paths and cycling tracks, the thickness of the top slab may be increased to 63 to 80 mm. Kerbs or rounded 'edgings' may be provided at the ends (see under "Kerbs"). A thickness of 80 mm for the common foot-paths and 100 mm for heavily trafficked paths and cycling tracks is usually recommended in England and U.S.A.; 50 mm is for the most lightly trafficked paths.

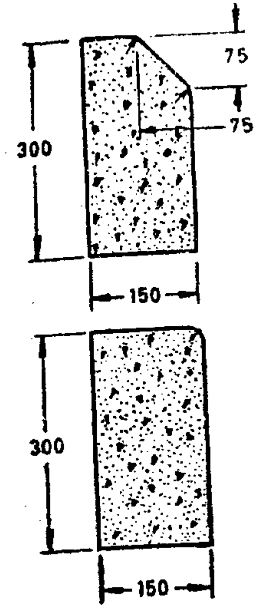
The pavment can be constructed in bays or in panels, or continuous with dummy joints at 1.83 to 2.44 metre intervals. For a 50 mm thickness the bay or panel should have no side longer than 1.83 metres, while 3.66 metres (12 ft.) should be the maximum length for the thickest slab. Sometimes pre-cast square slabs (of sides according to the width of the foot-path) are made which are laid cross-wise with their diagonals perpendicular to the length of the path, and the spaces left in between filled with half-square pieces. About 1.5 mm gaps are left at the joints which can be filled with bitumen. In this case no dummy or other joints will be necessary. Where the concrete is laid in situ, the surface of the foot-path can be made non-skid by pressing expanded metal pieces over it while it is still green, or the surface broomed as explained under "Concrete Road Slabs."

See also under "Cycle Tracks & Foot-Paths)"

Road Kerbs

Kerbs may be laid directly on the top of road slab resting about 15 to 23 cm in preference to at the sides. Risk of failure of the slab at the edges due to excessive loading at the edges is reduced. Where kerbs are laid by the side of the slab they are bedded in concrete to form the abutment of the road, and require longitudinal joints between the slab and the kerb. Such kerbs can then be used as side shutters for the pavement concrete. Kerbs should have joints in continuation of the transverse joints of the road slab. Precast kerbs give excellent service. Minimum size is 15x15 cm and 23 cm below the road level, edge rounded 5 cm, 1.8 metres in lengths, 1:2:4 concrete. Instead of kerbs, rounded edgings made of precast concrete, 7.5 cm thick and of full height of the footpaths above the slab top may be built. (See also under "Bridges"). A concrete channel is formed in front of the kerbs to guide the flow of storm water; see under "Roadside Gullies or Inlets". Concrete kerb and channel are usually cast as one piece. The height of the kerb (side of the piece) may be 15 cm and the width of the channel 45 cm.

Kerbs are made usually of heights (or vertical faces) 75 mm, 150 and 250 to 450 mm, top edges rounded with sloping (or battered) faces to prevent scratching of vehicle tyres. The smaller kerbs should be so designed that vehicles can ride over in case of emergency to avoid accidents.



CONCRETE KERB

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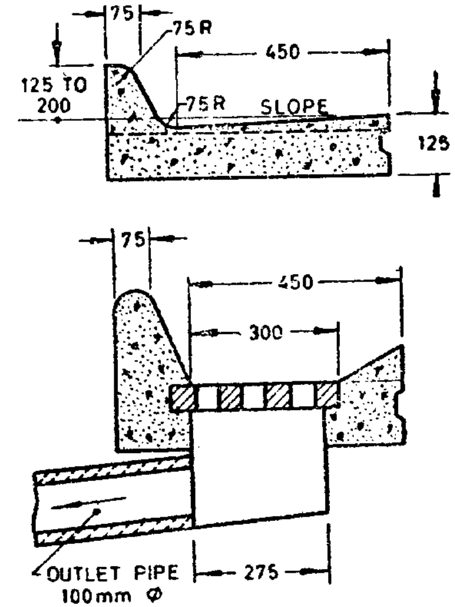


COMBINED KERB AND GUTTER

All dimensions in millimetres

Wheel Tracks or Trackways—These are known as **Creteways** when made of concrete.

A width of 60 cm (for each track) is quite satisfactory (with 45 cm min.) for both fast and slow moving traffic keeping to the path, but widths up to 85 cm are adopted. Distance between centre to centre of the track varies between 145 to 165 cm depending on the prevailing axle length. A 15 cm thick 1 : 2 : 4 concrete slab will be required for trackways on stabilized soil or well consolidated hard natural surface, 13 cm on medium base, and a 10 cm thick slab on well rammed kankar or stone. These should be able to carry heavy cart traffic. Other general requirements regarding preparation of the sub-grade should be followed. Continuous method of laying the concrete slab is preferable to alternate bay method. Provide expansion joints 12 mm wide at 14-metre intervals with butt joints 4.5-metre centre to centre. Trackways are not likely to receive much maintenance attention as they would be provided at remote places, therefore, joints at closer intervals are recommended.

Cement Grouted Macadam

(Also see under "Comentation and Grouting Method" in the Section on "Reinforced Concrete".)

Coarse aggregate consisting of broken stones, gravel or over-burnt brick-bats of size 63 to 40 mm is laid on a prepared and compacted sub-grade to the thickness required (adding 20 per cent extra thickness for consolidation) between the forms and levelled by means of a template to the camber required. If a little rolling is done it will compact the aggregate, reducing voids and thereby reduce grout consumption. Care should, however, be taken that no stone is crushed. A cement-sand grout in the proportions of 1 part of cement to $1\frac{1}{2}$ to $2\frac{1}{2}$ parts of sand with about 32 litres water per bag of cement is poured over the surface. The grout must have complete penetration to the full depth, which depends upon the fluidity of the grout and the size of the voids in the aggregate. Coarse sand grout is suitable for use with large aggregate and fine sand grout with small aggregate. Mixing of grout can be done in a tilting drum concrete mixer or a Grout-mixer. For the same fluidity more mixing water is required with fine than with coarse sand; with increase of water the grout becomes more fluid within certain limits only, but with excess of water the rate of flow is retarded especially with coarse sands. Therefore, correct amount of water for a particular mixture is essential. 'Wetting' agent can also be added to facilitate penetration. The sand used is coarse sand all passing 2.36 mm (No. 7 BS) sieve and only 5% passing 150 micron (No. 100 BS) sieve. If large size of aggregate has been used with much of voids, a layer of 20 mm shingle or broken stone is spread over the surface before grouting.

The grout shall be poured until all voids and interstices are filled but there should be no excess of grout on the surface. At no time shall the grouting be ahead of the tamping by more than 2.5 metres. Some engineers prefer to roll the surface after grouting as the rolling after grouting produces a smooth surface and brings up sufficient grout to cover the aggregate, but rolling should be delayed as long as hardening of the grout will permit.

In this type of work, poor aggregates such as chalk, soft sandstones, brick ballast can be used. This method is useful for base course under concrete pavements, light traffic roads or paths. There is a saving in cement from 20 to 25 per cent in addition to the saving in cost of stone, labour and time, as compared to normal construction methods. A top finish of hard stone can be given as described below.

For a smooth finished wearing coat, immediately after grouting, a concrete mix. of 1 : 2 : 3 with graded aggregate of 6 to 20 mm with minimum amount of water, should be placed on the surface and tamped with a tamper and finished with a wooden float or a belt as required. Hand tamping produces little compaction but levels the surface well. The thickness of the wearing coat is from 12 to 40 mm. Care should be taken to see that a perfect bond is secured between the grouted base and the premixed top. Or alternatively, after the grouting operation stone chipping of size 6 to 20 mm can be sprinkled on the surface and forced into the surface mortar by tamping. Where a smooth finished surface is not essential as in garden paths, the surface may be finished with a long handled broom type brush.

110 cu. m of coarse aggregate and from 400 to 550 bags of cement are used for 100 cu m of base course work. Joints should be made as for the premixed cement concrete roads and same procedure followed for curing.

Rolled Concrete

A mixture of pre-mixed lean concrete is laid on a prepared sub-grade between side forms and rolled with a light roller of 3 to 6 tonnes capacity,

preferably of the tandem type. Proportions of concrete mixes vary from 1:7 to 1 : 20 with sand contents varying from 2 to 4 times that of cement; the usual proportions being 1 : 2 : 8 with a water, cement ratio of 0.60, and proportions of 1 : 2 : 10, 1 : 3 : 10, and 1 : 5 : 10 with water-cement ratio of 0.64, 0.68 and 0.75 respectively. The maximum size of aggregate is not more than 50 mm for a 10 cm thick slab and this permits a slighly larger size of aggregate than with the common method. The pre-mixed concrete is laid rapidly on the sub-grade and rolled before initial setting of the cement starts; a 15 metre length of concrete should be laid within $\frac{1}{2}$ hour of mixing. Mixing is done in concrete mixers. The roller should not be of a heavy weight. A heavy roller will produce corrugations on the surface. The level of the loose concrete is kept 20 mm to 40 mm higher than of the final required thickness to allow for the compaction. Expansion joints are not required in a rolled concrete. Reduction of cement and sand contents reduce the shrinkage properties. Transverse construction joints are, however, made. As it is not possible to compact the concrete fully at the ends of transverse joints, dry metal is laid within 45 cm of both the edges instead of the mixed concrete and roller passed over. This portion is filled with premixed concrete after both the sides have been rolled, and hand tamped. Where the pavement is laid in two layers, the joints are staggered.

This type of work makes a good base course for a concrete or bitumen pavement for heavily trafficked roads and one course pavement for light traffic roads. Rolled concrete can also be adopted for improving or upgrading an existing water-bound macadam. There is a great saving of cement, sand and time for lying over the conventional method of tamping or vibrating. Compaction by rolling gives a greater flexural strength which is an advantage in resisting soil movements, especially clay soils, and that is done at much less cost for equal strength. Abrasive resistanee and crushing strength of the top layer of a road pavement have to be high but the lower layer may be an inferior type and this is taken advantage of in the above types of low cost constructions.

Cement Bound Macadam (*Sandwich method*)

The method of construction is to spread a 50 mm thick layer of single size stone of size between 50 to 40 mm (some engineers prefer to use graded stone between to 50 mm to 20 mm) and give it one or two runs of 6 to 8 tonne roller. Cement mortar of 1 : 2 or 1 : 3 and of the consistency of a brick-layer's mortar is spread over the surface (previously moistened) 40 to 25 mm thick and on this another 50 mm layer of similar stone is spread. The whole is then rolled again working from the sides towards the crown. The roller should preferably be of the tandem type. During rolling the mortar gets pressed into the voids of both the layers and as rolling proceeds, excess mortar works to the surface which is brushed uniformly over the surface. Rolling and brushing continues until a uniformly sealed surface is obtained. If stones are picked up by the roller wheels they may be slightly damped. This will make a smooth surface and if a rough surface is desired, as on steep gradients for animal traffic, surplus mortar should be removed by brushing. If the road is proposed to be used for pneumatic traffic, a top course of rich concrete mixture (1 : 2 : $3\frac{1}{2}$) about 25 to 40 mm thick of hard stone aggregate may be given. In this case the rolling on the base course should be stopped as soon as morter comes on the top. This method gives a saving of about 20 to 25 per cent of cement and 10 per cent of stone over the usual method of construction in addition to the saving in time, labour and supervision. Utility is the same as for the other types of low cost cement concrete roads described above. The two 50 mm layers of stone will consolidate to 90 mm and will need 110 cu. m of stone per 100 cu. m of consolidated thickness.

18. GRAVEL, KANKAR, BRICK AND SOIL STABILIZED ROADS

Gravel Roads

Gravel roads are a layer of compacted gravel (or crushed rock) graded from fines to pebbles containing binding stuff (clay) in the fines. Sandy and gravelly materials which meet the grading requirements given earlier and under "Soil Stabilised Roads" in the pages following have a high bearing value and provide an excellent base under a bituminous surface. The size and grading of the gravels should vary from 50 mm at the bottom to 12 mm at the top. If grading is not possible as suggested above the following proportions may be taken:

25 mm to 20 mm	15 per cent
20 mm to 6 mm	75 per cent
Below 6 mm	10 per cent

Washed gravel is devoid of the fines needed to bind the material, and will require the admixture of pulverized clay, about 5 to 10 per cent, to act as a binder. Or, alternatively, the proportion of fines passing a 75 micron (200 BS) mesh sieve should be about 10 to 15 per cent and sufficient to fill the voids in the gravels. The sand content in the fines should be at least twice as great as the clay content.

Gravel roads are generally built in two courses, foundation course and surface course. These types of roads are generally built in shallow trenches; where not practicable, feather edge types are used, with desired cross slope for the pavement surface. A thickness of about 15 cm is required for light traffic and about 30 cm for heavy traffic, but such roads are suitable for only light traffic. Compaction to be done in layers not exceeding 10 cm in thickness, with rubber-tyred rollers or a 6-tonne smooth-wheeled roller.

Kankar Roads

The kankar should be hard, tough and free from earth and sand. A good specimen of kankar available in the south will show a brownish fracture. "Bichwa" kankar as available in the Punjab and U.P. should show a bluish surface on fracture. For road work the kankar is broken to gauge varying from 20 to 63 mm and the largest size is used for the bottom layer and the smallest for the top. It is gauged as follows:-

(*i*) The whole passes a 80 mm sq. mesh screen.

(*ii*) Not more than 20 per cent is retained on a 63 mm sq. mesh screen.

(*iii*) Not more than 10 per cent passes a 25 mm sq. mesh screen.

(*iv*) No quantity passes a 12 mm mesh screen.

Soling may be of bricks or kankar. The kankar is generally spread in 15 cm layers. Two parallel mud walls, 20 cm wide and 15 cm high are made of well puddled clay along the outer edges of metalling to confine the metal and prevent its spreading under the action of rammers. Templates (made of wood and of the road cross-section) having bottom member of a depth equal to the unconsolidated thickness of metal, should be placed at distances not exceeding 15 metres, truly horizontal to ensure that both sides of the road are dead level.

After the kankar has been spread it shall be flooded with water till the water fills all the interstices between the kankar. Ramming shall be effected by not less than 16 men and a leader on a 3.66 metre road, and by not less than 12 men and a leader on a 2.74 metre road. The gang shall be formed into two rows close together and shall first ram the haunches working parallel to the road over a width of 1 metre on each side. They shall then ram the central portion of the road working at right angles to the road. All ramming shall continue until the surface has been thoroughly compacted and no marks are left by the action of the rammers. During the whole process of ramming the surface shall be kept well watered. The time allowed for ramming shall not exceed three working days per 200 metres of road length. Not more than 200 metres of the road should be under operation in any one part of the road at one and the same time.

Consolidation of kankar metal is now done by light weight road rollers and has been quite successful.

When traffic is allowed on a newly consolidated road it shall be spread over the full width, and is generally done by placing tree branches over the track.

Repairing Old Kankar Roads

If the thickness of the crust exceeds 10 cm the whole of the old surface shall be picked up to a depth of 40 mm and the roughened surface raked true to template. If it is 10 cm or less the surface shall not be picked up but shall be scored across with criss-cross lines. The criss-cross lines shall not exceed 25 mm in depth and shall be 23 cm apart. If the old surface is badly rutted the higher portions shall be excavated to such a depth as will ensure, when the excavated material is filled up in the depressions, that the depth of the loosened surface is uniform. The old metal shall be screened and good metal used up again. Other general specifications given for water-bound macadam roads should be followed.

Surface dressing on kankar roads with tar or bitumen has been quite successful. It can take much heavier traffic and increases the life of the road considerably.

Brick Pavements

Pavements in town streets for light traffic can be made with bricks laid flat or on-edge over 50 to 75 mm of rammed ballast or lime concrete well consoidated. Bricks may be laid while the concrete is still green. The soil should also be well rammed and brought to camber or proper levels. If bricks have to be laid on a previously placed or existing hard surface, a sand cushion of 25 mm is laid under the brick pavement. The sand used should be coarse sand. Bricks are laid with their length along the width of the road with joints evenly spaced, parallel and at right angles to the centre line of the road, adjacent layers breaking joint, with a 23 cm wide profile at right angles to the length of the street, at about 2.5 metre intervals. Edging is made with bricks laid parallel to the road. At curves the bricks should be laid in radial courses transverse across the roadway, allowing at the outside of the curve a joint space between each course not exceeding 12 mm. Where the joints exceed 12 mm, the bricks should be laid in more than one course leaving a space in between and in this intervening space the bricks should be laid longitudinally at right angles to one of the transverse courses at each successive closure. At crossings double diagonal or herring-bone method should be adopted.

The bricks should be set on a bed of cement mortar and joints filled up flush with mortar. Joints should be as fine as can be laid. Or alternatively, the joints are filled with bitumen and dusted with sand. After laying the pavements should be thoroughly rolled with a tandem roller of 3 to 5 tonnes weight. After final rolling the pavement should be tested with a straight edge laid parallel with the kerb, and any depressions exceeding 6 mm should be corrected. Camber should also be checked for drainage. Portions of the pavement inaccessible to roller should be tamped by a hand tamper applied upon a wooden board.

Bricks for pavings must be well burnt or slightly over-burnt and hard, and need not have 'frogs'. Bricks can also be laid in header and stretcher bond or herring-bone bond. Herring-bone bond gives a smoother riding surface. A paint coat of bitumen or tar with chips or a light premix carpet can be laid over brick pavements to give a smooth riding surface; the same treatment can be given where the surface becomes uneven by traffic or settlement of the sub-grade. It is advantageous to blind the brick pavements occasionally with sand or earth as it will preserve the bricks and also provide a smoother riding surface.

See also under "Brick Soling" and "Brick Metal".

SOIL STABILISED ROADS AND EARTH ROADS

(This subject has also been dealt with in Sections 6, 7, 17 and in earlier articles of this Section.)

The three main constituents of soil — sand, silt and clay, rarely occur in nature in a pure or "stabilized" form but have limitless combinations of proportions. By "stabilized soil" is meant that the different constituents of soil are mixed in such a proportion that the properties of the resultant soil produced are more resistant to weathering, and the load-carrying capacity is considerably increased and the soil is maintained in a high state of stability. Most soils, to be stable, require the addition either of fine or of coarse material so that the proportions of particles of different sizes fall within certain limits. A large number of soils when well compacted and at a suitable moisture content have good load bearing properties but become unstable if their moisture content is increased. To achieve ideal mechanical stability it is necessary to have a well proportioned coarse material (having a particle size distribution giving a high dry density), together with some fine binding material such as clay. Shear strength of a graded cohesionless soil is greater than that of a soil where the particles are of a uniform size. Numerous methods have been prescribed for determining the proportions in

which materials of known sizes must be mixed to produce a specified size distribution. Mix-in-place method has usually to be adopted.

As a rough guide, an intimate and compact mixture of the following will make a stabilized soil:-

Clay ...	... 5 to 10%	It will usually be sufficient to have 70% sand and 30% clay (or silt and clay together). The percentage of clay and silt mixed is equal to the percentage of voids in the sand.
Silt ...	... 10 to 20%	
Total sand	... 70 to 85%	
(Coarse sand)	... 15 to 25%	
	By weight	

From 45 to 60 per cent sand should be retained on a 250 micron (No. 60 B.S.) sieve. The clay content should be more in dry areas and less in wet tracts.

If aggregate is also used, the proportions may be:-

Silty clay ...	... 18%	The aggregate is 20 mm down to 4.75 mm well graded. Sand is 85 per cent passing 2.36 mm (or No. 7 BS) sieve and none passing 75 micron (or No. 200 BS) sieve.
Sand ...	... 45%	
Aggregate (graded) ...	37%	

The aggregate may be 40 mm down to 10 mm. The maximum size of aggregate may be even up to 75 mm, well graded up to fines (only small percentage of fines).

Another proportion recommended is:-

35 per cent of stone graded from 10 mm to 40 mm. The material passing 10 mm might show 90 per cent passing 4.75 mm sieve, 65 per cent passing 2.36 mm sieve, 50 per cent passing 300 micron (No. 52 BS) sieve and 15 per cent passing 75 micron (No. 200 BS sieve).

Soils of the following properties are also considered satisfactory for constructing earth roads:-

	Base coarse	Wearing coarse
Clay	less than 5%	10 to 18%
Silt	9 to 32%	5 to 15%
Sand	60 to 80%	65 to 80%

A higher proportion of the finer sizes is required in the surfacing than in the base to assist in the retention of moisture necessary for cohesion. Before a soil can be improved it is very essential to determine its clay contents.

Where aggregate are used for stabilisation, it has been suggested that weak aggregate is to be preferred because it will break down under compaction to give a size distribution more closely approaching that required for maximum dry density. Adequate compaction is essential for mechanical stabilization; it is important to ensure that.

In the case of heavy clay soils, *e.g., black cotton soils*, improvement can be had by spreading sand on the top of the soil, which is not mixed with the soil, and blending is achieved by the combined action of traffic and weather. A layer of 12 mm to 40 mm of coarse sand or cinders or slag, just before the rains also improve the riding qualities of a clay soil.

Soil Stabilisation in Water-logged Areas

Where the sub-soil water level is high, the sub-grade cannot be rolled with a heavy roller. (Sheepsfoot roller is not used.) Where the bearing capacity of a soil is low, with a heavy roller the sub-grade would be lifted in ripples as the roller passes over it creating boggy conditions. In such cases the sub-grade should be rolled with repeated numbers of rollings with a light roller, say about 1½ tonnes. Measures like rolling ballast into such a sub-grade to increase its bearing capacity have been described earlier.

Earth Roads—The camber and grade of an earth road is maintained by means of a *road-drag* which is dragged on the road surface when necessary. Road drags are made of wooden pieces braced together and drawn by bullocks or horses.

Roads in Sandy Tracts

One or two layers of sand stabilised with bituminous materials (penetration grade bitumens, cut-backs or emulsions) laid directly over a sandy sub-grade can be made as base course. A 10 cm thick base course (A loose thickness of 15 cm is compacted to a 10 cm thickness) with a wearing course of 20 mm thick premix carpet has been found to be satisfactory for carrying normal traffic up to 150 vehicles per day (vehicles exceeding 3 tonnes loaded weight) and also occasional heavy traffic of up to 1500 vehicles per day. Hot sand-bitumen mix with a filler (say, locally available kankar dust) has been found satisfactory. For the sand the percentage passing 75 microns sieve shall be not more than 10 per cent by weight.

To impart greater stability to the establised sand mix coarse aggregate can be incorporated in the mix up to a maximum of 25 per cent by weight of the mix. The coarse aggregates should be either gravel or kankar, clean and free from fines and clayey matter, and the size be such that the whole of it should retain on 4.75 mm sieve with the maximum size not being more than 1/3 of the compacted thickness of the layer to be laid.

The final finishing of the berms as well as providing the proper cross slope or superelevation shall be done after the sand-bitumen work has been completed. In the case of sandy subgrades, the carriageway should be marked with planks of depth equal to the proposed loose thickness of the sand-bitumen layer. The subgrade shall have the required camber or superelevation of the finished road surface, and it shall then the sprinkled with water and lightly rolled. Grooves 8 cm wide and 5 cm deep shall be dug at the edges on either side so as to give a thickened edge. This will give additional strength to the edge.

The following measures can be adopted in order to provide a firm temporary road surface:—

(*i*) 30 cm layer of good soil on top of sand.

(*ii*) Where much good earth soil is not available, a good road can be made by a 15-cm thick layer of rolls of jungle brushwood with 15 cm to 20 cm of good earth on top. Brushwood bundles are laid diagonal to the road surface.

(*iii*) Spread sarkanda, long grass, or some similar stuff in a layer from 8 cm to 15 cm thick loaded with good earth at both ends.

(*iv*) Wheel tracks of concrete or bricks: Concrete tracks have been described earlier under "Wheel Tracks or Trackways" under "Concrete Roads." For brick tracks the bricks may be laid flat or on edge directly on the consolidated sub-grade or on made foundations according to the traffic conditions. The joints between bricks may be left 6 mm and sanded within 25 mm on the top and then grouted with hard bitumen or tar. General principles of laying as given under "Brick Pavements" should be followed. Size of tracks has been given under "Concrete Trackways".

Roads in Kallar Tracks

A tolerable kachha road can be made if 30 cm of kallar soil is removed and 15 cm of earth is put over 15 cm of good sand.

Sand-Clay Roads

A surface course not less than 25 cm thick built of sand-clay mixture in the proper proportions will give good service under medium traffic, provided that constant blading is carried out to maintain the correct shape. A sand-clay road has only about half the load-carrying capacity of a gravel road. It does, however, provide a suitable base on which a more durable surface can subsequently be laid.

To prepare an admixture of sand and clay soils the percentage of clay to be added to sand should be equal to the percentage of voids in the sand.

A rough guide to the proportions of the components is: 25 per cent clay and 75 per cent sand with proper grading. The grading should preferably be within the following limits:

Passing 2.36 mm (No. 7 BS) sieve	60 to 100%
Passing 600-micron (No. 25 BS) sieve	30 to 60%
Passing 75-micron (No. 200 BS) sieve	9 to 25%

Mixing of clay and sand may be done either in a machine for which any paddle or pan type of concrete or bituminous mixer can be used, or on the road site. The materials should be mixed dry in the first instance. Clay must be pulverized which can usually be done with a tractor-drawn harrows or agricultural rollers. Where mixing has to be done at the road-side, the sand is spread over the subgrade and the pulverized clay brought over and spread on top. The materials are mixed by blading back and forth, five turn usually being sufficient.

The subgrade is made true to grade and cross-section, and free of bumps, depressions and ruts. It is then moistened and rolled after which the dry mixed material is spread over it to the desired width (where mixing has been done in a mixer).

Wet mixing, where unavoidable, may be done either in the mixing plant or on the road.

Compaction is done by pneumatic-tyred rollers, smooth-wheeled rollers, or by carefully regulated rubber-tyred traffic.

Stabilization of Black Cotton Soil and Moorum with Lime

The soil from the borrow areas shall be collected after removing the top 7 to 10 cm of soil crust which is usually very hard. It shall then be placed on the subgrade in loose thickness of 20 to 23 cm, and pulverized by mechanical means. Fat lime (freshly slaked at site) shall be spread over the loose soil at the rate of 4 per cent of the dry weight of soil. (This can be worked out by finding out the approximate weight of dry soil per cu. m.)

The soil-lime mixture shall be mixed thoroughly with a rotary device failing which by means of manual labour. The soil in small stacks shall then be brought to optimum moisture after allowing to remain overnight, the wet soil shall be thoroughly mixed with rotavators. This shall then be compacted with 8 to 10 tonnes power roller resulting in a consolidated crust of about 15 cm. (IRC : 15-1970)

Salts in Soil—Presence of detrimental salts in harmful quantities have also to be determined. Salts of sodium sulphate or sodium carbonate are considered detrimental to soil stabilization as they make the pavements soft and fluffy during winter. A 6-in. layer of any sharp (pure) sand interposed under a road crust of stabilized material will prevent the rising of salts in a salt effected area.

If the local soils are fine grained and suitable soils for mixing with them to produce a well graded stable soils are not available within economic distance, insoluble binders such as cement or bituminous materials are mixed in small percentages to stabilize the road crust. Such a crust has, however, no appreciable abrasive resistance and a (black-top) wearing surface has to be provided.

Stabilization of a soil can be judged by mixing the soil in correct proportions, or with cement or other stabilizer, and compacting the mixture at a moisture content thought to be suitable for rolling into a mould which is let to dry. If the mould hardens satisfactorily the soil mix is likely to be suitable for stabilization.

To prepare a site for stabilization work, the top soil is removed, drainage provided and the ground shaped up to its formation level. The sub-grade is thoroughly compacted before the stabilized soil is superimposed on it.

Soil Stabilization with Binders

Cement, bituminous materials and sodium silicate are generally used as stabilizers or binding agents for natural soils.

Stabilization with Bituminous Materials

Earth roads can be improved by spraying the dry soil surface with a stabilizer such as asphaltic and cut-back bitumens, oils, tars or emulsions, in small quantities. This process is successful in hot and dry climates but soils with high moisture content cannot be stabilized by this method. Best results are obtained on well graded soils. The road is first shaped by grading and the binder is then usually applied in two or three equal distributions totalling about 450 litres per 100 sq. metres so that the binder penetrate about 12 to 25 mm into the soil. The applications are separated by about a week. The more porous is the road surface the more viscous the binder and greater the rate of application. It is preferable to blind the surface with a light dressing of coarse sand and then lightly rolled.

The main use of bituminous stabilized soil for roads has been in the construction of bases for lightly trafficked surface-dressed roads. Tar and bitumen are equally suitable; more careful control is required with tar than with bitumen.

Site Reconnaissance and Investigation of Soil Conditions

The general topography of the land will give some indication as to

whether the soil conditions are likely to be variable or not. A change in the vegetation over quite a small area may indicate an important change in the sub-soil. A single line of borings at intervals of about 100 metres along the centre line of the road alignment or a double line offset 15 or 30 metres on each side will usually be sufficient. The depth of boring should be about 1.5 metres below the existing level or finished formation level, whichever is the lower, with occasional deeper borings. For high embankments, boring should be to a depth about equal to twice the height of the embankment if there is a possibility of soft material underlying it. Investigation of the depth of sub-soil water level is very important.

Consolidation or Compaction

(See also under "Earthen Embankments and Dams" in Section 17 (and also this section) and "Rolling" described earlier.)

Consolidation is a process in which the soil is compressed under load, voids are reduced (by the expulsion of water from the pores and expulsion of air from the voids) and the soil particles are packed closer. Compaction improves the properties of the soil, its shearing strength and bearing capacity are increased and its ability to absorb water is decreased. The compaction of a soil is measured in terms of its dry bulk density or the amount of solid matter in a unit mass. Increased compaction results in increased dry density until the volume of air remaining in the soil is so reduced that further compaction produces no substantial change in the volume. Too much of rolling disturbes the structure of the soil.

To achieve high density the soil should be compacted in thin layers. One of the most important factors is the moisture content of the soil. Greatest efficiency is obtained from the roller when the moisture is at its optimum value. It is inadvisible to compact cohesive soil sub-grades below their optimum content in cases where they are likely to be subject to the ingress of moisture. A light roller on a dry cohesive soil may merely ride over the top and little alter the soil structure. In general, the heavier the roller the better the compaction.

Any filling if adequately compected need not be left to "weather" before the pavement is laid over it.

Base course and sub-grade of cohesive soils are rolled with sheepsfoot roller. For dry clays the sheepsfoot roller gives the highest densities. The rolling is finished off with a flat three-wheel roller 5 to 6 tonnes in weight. On an average about 60 trips of the roller (sheepsfoot) are required to consolidate a 3.66 metre (12 ft) width of the road. Wearing course is rolled with a 5 to 6 tonnes flat roller. One roller is capable of doing 200 metres of subgrade, base course and wearing course per day.

On some of the cohesive types of soils, compaction is best effected initially by a pneumatic tyred roller, followed by a smooth-wheeled roller to give the final shape and finish. Power rammers are suitable for small areas.

CYCLE TRACKS AND FOOT-PATHS

Cycle Tracks

Separate cycle tracks may be provided when the peak hour cycle traffic is

400 or more per hour on routes with a traffic of 100 motor vehicles per hour. When the number of motor vehicles using the route is more than 200 per hour, separate cycle tracks may be justified even if the cycles traffic is only 100 cycles per hour.

The width of cycle tracks should normally be 3 metres and should in no case be reduce to less than 2 metres for up to 2000 cycle per hour, increasing in multiples of 1 metre per 1500 cycles more.

As far as possible, a cycle track should be so aligned that the radii of the horizontal curves are not less than 10 metres. Where the track has a gradient steeper than 1 in 40, the radii of horizontal curves should not be less than 15 metres. Vertical curves at changes in grade should have a minimum radii of 200 metres for summit curves and 100 metres for valley curves.

Gradients steeper than 1 in 30 should normally be avoided. Only in exceptional cases, gradients of 1 in 20 and 1 in 25 may be allowed for lengths not exceeding 20 metres and 50 metres respectively.

It is desirable that cycle tracks should be provided on both sides of a road and be separated from the main carriage-way or foot-paths by grass verges or berms of width not less than 1 metre, which may be reduced to 50 cm under exceptional circumstances. Should it not be practicable to obtain sufficient space for the verges, the cycle tracks may be separated from the carriage-ways and foot-paths by kerbs or fencing.

It is desirable that a cyclist should have a clear view ahead of not less than 25 metre, and on gradients of 1 in 40 or steeper, not less than 60 metres. The minimum head-room provided should be 2.25 metres. The minimum width of a cycle track at an under-pass should be 2.5 metres instead of 2 metres in the open. The head-room (or vertical clearance) should also be increased to 2.5 metres instead of 2.25 metres. The standard of lighting on cycle tracks should be equal to that of the main carriageway.

Full width cycle tracks should be provided over bridges also. Where the cycle track is located immediately next to bridge railing or parapet, the height of the railing or the parapet should be made 15 cm heigher than required otherwise.

Cycle tracks may be (i) adjacent to the carriageway at the same level, (ii) adjacent to the carriageway but at a higher level, or (iii) separated from the carriageway by a verge at the same or different level. It is desirable that the cycle tracks should be as wide as possible, the the verge separating the cycle tracks should be as wide as possible, the minimum being one metre. Under exceptional circumstances, on streets with inadequate road land, verge could be reduced to 0.5 metre.

Side-walks or Foot-paths

Minimum width should be 1.5 metres. In industrial and business areas, it should be 4 m for main streets and 2 m for minor streets. In residential areas, the foot-paths should have a width of 3 m in main streets and 1.5 m in minor streets. If space is available sidewalk should be provided on either side of the road and should be raised above the general carriageway level. The sidewalks should be sloped adequately, ranging from 1 in 40 to 1 in 30 which will be sufficient to drain away the rain water.

19. DESIGN OF PAVEMENTS

The design of a pavement is governed by the traffic density, *i.e.*, the number of vehicles using the road during peak hours, and the maximum wheel load. The daily traffic volume is about 8 to 10 times the maximum hourly volume Character of the traffic and the speed of vehicles are also major considerations. Rate of traffic increase per annum is also to be considered and which may vary from 10 to 20 per cent in a developing country like India.

Density of traffic is sometimes defined as the volume of traffic per unit width of carriageway.

Intensity of traffic is expressed as so many tonnes of traffic per unit width of carriageway per day.

Land Width is the total width required to accommodate roadway, berms, drains and width reserved for future developments. Minimum land width is exclusive of land required for borrow pits, while desirable land width includes land for borrow pits.

Recommended Land Widths for Different Classes of Roads (IRC)

Road Classification	Plain & Rolling Terrain				Mountainous & Steep Terrain	
	Open areas		Built-up areas		Oper areas	Built-up areas
	Normal m	Range m	Normal m	Range m	Normal m	Normal m
National & State Highways	45	30-60	30	30-60	24	20
Major District Roads	25	25-30	20	15-25	18	15
Other District Roads	15	15-25	15	15-20	15	12
Village Roads	12	12-18	10	10-15	9	9

Road land width is also called "right of way" and is the land acquired for road purposes.

Desirable Land Widths for Roads in Urban areas :

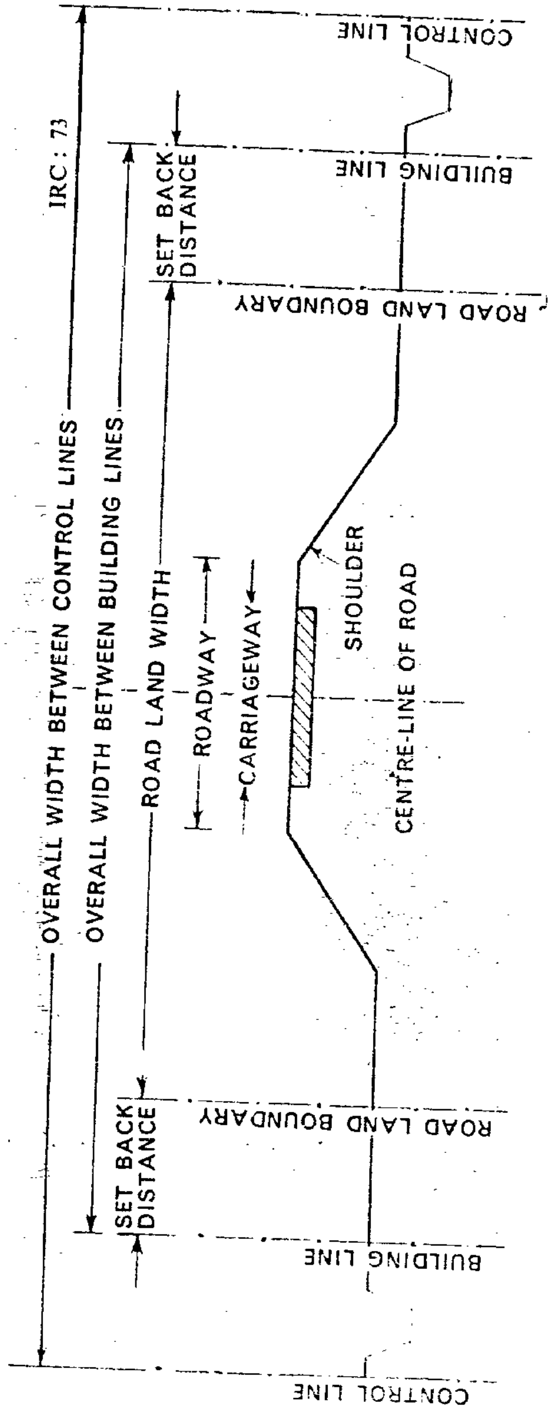
Arterial Roads	50-60 m
Sub-arterial Roads	30-40 m
Collector Street	20-30 m
Local Street	10-20 m

(a) Distance between Building Line and road boundary (set-back) should be 3 to 6 metres.

(b) In order to ensure proper Sight Distance, it may be necessary to acquire additional land over that indicated in the Table.

Formation width is carriageway plus shoulders. It is the top width of embankment or the bottom width of cutting excluding the side drains.

In open country roads, earth berms need to be provide of width 1.8 m (min:) on each side.



Recommended Standards for Rural Highways

Road Classification	Plain & Rolling Terrain	
	Open areas	Built-up areas
	Overall width between Building Lines	Distance between Building LIne and road boundary (set back)
	Metres	Metres
National & State Highways	80	3-6
Major District Roads	50	3-5
Other District Roads	25-30	3-5
Village Roads	25	3-5

For mountainous and steep terraines distance between Building Lines and Road boundary (set-back) is 3 to 5 metres

Minimum Width of Carriage way (Pavement) in Urban Areas

Single lane without kerbs	3.75 m
Two lane without kerbs	7.0 m
Two lane with raised kerbs	7.5 m
Three lane with or without kerbs	10.5 m
Four lane with or without kerbs	14.0 m
Multilane width per lane	3.5 m
Village roads	3.0 m

Urban Roads in Plains (Developed areas)

Urban roads other than expressways (National Highways) are classified as:

Arterials - A street primarily for through traffic, usually a continuous route

Sub-arterials - As above but offering somewhat lower level of traffic mobility.

Collector Streets - Those collecting traffic from local streets and feeding it to the arterial and sub-arterial streets or vice-versa.

Local Streets - These are intending primarily to provide access to abutting property and normally do not carry large volumes of traffic.

Width of Roadway (Formation) for Single-Lane or Two-Lane Roads in Plain & Rolling Terrain

Road Classification	Roadway Width
1. National & State Highways (single or two lanes)	12.0 m
2. Major District Roads (single or two lanes)	9.0 m
3. Other District Roads	
(i) single lane	7.5 m
(ii) two lanes	9.0 m
4. Village Roads (single lane)	7.5 m

Width of Carriageway, Roadway, and Right-of-Way for Hill Roads

Class of Road Classification	Carriage-way width m	Shoulder width m	Roadway or Formation* width	Road Land Open area	Road Land Build-up area
(a) National & State Highways					
(i) Single-lane	3.75	2 × 1.25	6.25	24	20
(ii) Two-lane	7.0	2 × 0.9	8 8	24	20
(b) Major District Roads	3.75	2 × 0.5	4.75	18	15
(c) Other District Roads	3.75	2 × 0.5	4.75	15	12
(d) Village Roads	3.00	2 × 0.5	4.00	9	9

Based on IRC: 52-1981 & 70-1977

The width of separate Parking Lane, wherever provided, may be kept 3 m.

The carriageway intended for one line of traffic movement is generally called a *traffic lane*.

Width of pavements at (railway) level-crossings approach roads up to a length of 30 metres measured outwards from the gates :

Single-lane—6.71 m Double-lane—7.32 m

Increase of width is required on approaches to towns and industrial areas to provide for increase in traffic.

Notes : The roadway widths given above are exclusive of the parapet (usual width 0.6 m) and side drains (usual width 0.6 m). These should be provided in addition to the shoulders as necessary.

*In hard rock stretches, the width of shoulders may be reduced by 0.4 m on either side on double-lane roads and 0.2 m in other cases.

On roads subject to heavy snow-falls, where regular snow clearance is done over long periods to keep the road open to traffic, roadway width may be increased by 1.5 m for District Roads and Village Roads.

The full roadway width should be carried through on culverts, scuppers and causeways.

The widths given above are meant for undivided highways. Over and above the required right-of-way, the minimum set back for building line should be 5 m in normal cases and 3 m in exceptional circumstances. Additional land may be acquired at locations involving deep cuts, high fills and unstable or landslide areas.

Lay-bys. On important roads lay-bys should be provided at intervals to enable vehicles to draw off the road for temporary parking of break-downs and repairs. Bays should also be provided for bus stops. A space of about 30 m × 3.0 m is sufficient for lay-bys.

Traffic Counts-Equivalency factor for vehicles

	Vehicle type	Equivalency Factor	Since the road traffic is
1.	Passenger car, tempo, autorikshaw	1.0	Composed of different
2.	Cycle, motor cycle or scooter	0.5	types of vehicles,
3.	Truck, bus, tractor-tailer unit	3.0	it is converted
4.	Cycle rikshaw	1.5	into equivalent
5.	Horse drawn vehicle	4.0	passenger car
6.	Bullack cart	6-8.0	unit for counting

Road Capacity

	Type of Road	Capacity
1.	Single lane, 3.75 m carriageway with earth shoulders	1,000
2.	Single lane with hard shoulders 1.0 m wide	2,500
3.	Two-lane, 7 0 m carriageway	10,000
4.	5.5 m carriageway	5,000
5.	Four-lane divided highway	20,000—30,000

*It is traffic count-equivalency factor for vehicles per day (both directions).

"Commercial vehicles" inclde all such vehicles except light delivery vans and three-wheeled vehicles. Roads carrying heavy indivisible loads should be designed as for very heavy traffic.

Portable (electronic) recording devices have been developed which are laid across the roadway and they measure axle weights, spacings and speeds of vehicles in motion.

Three-lane roads offer only limited advantage over the two-lane roads, and are unsafe where the passing sight distance cannot be provided. Four-lane roads should invariably be provided with median strips.

Basic (max.) traffic capacity in town streets considered in 1300 passenger vehicles per hour of "green light", per traffic lane of 3 75 m width, all moving in the same direction. Traffic capacity of a two-lane road (at peak hours) should not be more than 1500 passenger vehicles per hour, regardless of direction, all moving at a speed of not less than 50 km/hr. It is however not economical to design a road so wide as to be congestion free every hour throughout the day.

As traffic volume increases, top speed decreases, (road) capacity also decreases with decrease in speed. The maximum capacity of a two-lane road is cut to almost half when vehicles speed is decreased to 16 km/hr. from 50 km/hr At 50 km/hr. the minimum spacing of vehicles centre to centre is about 25 metres where as at 16 km/hr. it is 14 metres.

For counting equivalents of trucks or slow moving vehicles to passenger vehicles, see under" Traffic Counts-Equivalency for vehicles".

Basic capacity of a traffic lane is the maximum number of passenger cars that can pass a given point on a lane or roadway during one hour under the most nearly ideal roadway and traffic conditions that can possibly be attained. Counting is done at street intersections and street width is taken from kerb to kerb.

Thickness of Pavements Design

There are two main types of pavement constructions, *viz.*, Flexible (bituminous) pavements and Rigid (concrete) pavements. The function of pavement design is to design the thickness and type of construction that will distribute traffic loads to enable the subgrade (and base or sub-base layers) to support these loads. No sound theoretical and fully acceptable method of pavement design has yet been evolved although there are several methods of design in use which are wholly or partly empirical. The thickness design of a pavement involves the interplay of several variables such as the wheel load and its impact effect, volume and character of the traffic expected to use the road, bearing capacity of the soil on which the road is to be built. The individual effect of many of these factors is difficult to evaluate mathematically with any precision.

When a wheel load is applied to the pavement, the intensity of stress reduces as the depth increases from the top of the surface. Thickness of pavement is increased for heavier loads and a larger total volume of traffic.

CBR (California Bearing Ratio) Method of Design. Of all the available methods of pavement design the CBR method has been found the most reliable practical means of evaluating the strength of the subgrade (bearing capacity of the soil) and construction materials, and of estimating the required thickness of pavement to satisfy a given loading. It is the most convenient and widely used of all the methods. Though devised for the design of flexible pavements, the method is also used to determine the total thickness of a concrete pavement and granular base or sub-base. It gives a reasonably accurate estimate of the required thickness of construction. But, the CBR method is considered to give extravagant thickness for roads which have to carry only light traffic.

CBR is a measure of the load carrying capacity (resistance to direct penetration) of any soil or granular material, which is expressed as a percentage of the load carrying capacity of a standard crushed rock specimen (which is taken as of 100 per cent value) determined by a penetration test. This is an arbitrary figure : a surface having a CBR of 100 per cent is one in which a load of 1360 kg has to be exerted to drive in a cylindrical flat plunger with a base area of 19.3 sq. cm to a distance of 0.250 mm at the rate of penetration of 0.125 mm per minute.

Computation of Traffic for the use of CBR

Distribution of traffic over the carriageway-the traffic is considered in units of commercial vehicles.

(i) Single-lane roads (3.75 m width) : The design is based on the total number of commercial vehicles per day in both directions multiplied by two.

(ii) Intermediate width roads (5.5 m width) : The design should be based on the total number of commercial vehicles per day in both directions multiplied by 1.5

(iii) Two-lane single carriageway roads : The design is based on 75 per cent of the total number of commercial vehicles in both directions.

(iv) Four-lane single carriageway roads : The design should be based on 40 per cent of the total number of commercial vehicles in both directions.

(v) Dual two-lane carriageway roads : The design should be based on 75 per cent of the number of commercial vehicles in each direction. The distribution factor shall be reduced by 20 per cent for each additional lane.

Modulus of subgrade reaction (or co-efficient of reaction, or subgrade modulus) is a measure of load carrying capacity of a soil (resistance of a soil to penetration determined by loading a plate) used in Westergaard's formula or its derivatives, represented as "k" value. It is more or less like CBR value. It is a measure of the resistance of a soil to penetration or deformation under a load, expressed as kg/sq. cm/cm or kg/cu. cm. It is assumed that subgrade reaction at any point is proportional to the deformation. Values of subgrade modulus have approximate co-relation with CBR values. (Westergaard's formulae being rather complex and beyond the scope of this work, have been omitted).

The strength of a soil depends upon its density, cohesive strength and the moisture content.

Moisture content is the most important factor in the case of clayey soils which are most likely to suffer by water absorption. It is therefore, important to ascertain the wettest condition in a given case and the basis of design should be the strength of the subgrade in the condition. Testing the compact soil for bearing strength at 100 per cent satuation is essential for areas subject to water-logging and floods. As the governing factor is the bearing strength after full soaking, a soil which does not prove satisfactory after soaking can be improved by suitable admixture of granular material. All clayey soils must be tested after full soaking whether or not the area is subjected to water-logging. The materials in successive layers downwards of a pavement crust must be progressively weaker.

A water-table less than 60 cm below the soil formation level usually means a poor foundation and drainage measures should be taken as described earlier.

In the case of sandy soils the detrimental effect of moisture is much less than in clayey soils. The strength of soil depends on its density, therefore, it is important to compact sandy subgrades to the maximum density possible.

The subgrades have the property of recovery if the stresses are not of a repetitive nature and do not exceed the safe bearing capacity of the subgrade. A certain amount of occasional and infrequent overstressing the pavement is not harmful. Therefore it will be uneconomical to design pavements for unusal heavy loads passing once in a way.

Each time a load passes, some deflection of the surface and th underlying layers occurs and the useful life of a road is shortene Repetition of loads has a destructive effect and repeated applications of a excessive load will lead to cracking and ultimate failure. An occasion overload of twice the design load will not be destructive.

Thickness of Construction for Flexible Pavements in Centimetres Based on CBR Values

No. of Vehicles per Day Exceeding 3 Tonnes Laden Weight		Approx. CBR per cent											
		3	4	5	6	8	10	15	20	30	40	50	60
0—15	Based on IRC:37	30	26	23	20	18	15	12	10	8	7	6	5
15—45		37	32	28	25	22	19	15	12	10	8	7	6
45—150		43	37	32	30	25	22	17	14	11	9	8	7
150—450		50	42	38	33	28	24	19	16	13	10	9	7
450—1500		57	48	42	37	32	27	21	18	14	11	10	8
1500—4500		62	53	47	42	36	31	24	20	15	12	11	9
above 4500		68	60	53	48	40	35	26	23	17	14	12	10

Bearing Values of Soils (Sub-grades)

Description of Soil	*Classification of soil*	*Bearing values* kg/sq. cm*	*Approx. CBR*
Soft and plastic clays—poor drainage	Very poor	0.70	3
Black-cotton soils	Very poor	0.90	4
Silty clays or heavy clays—uncertain drainage	Poor	1.05—1.20	5—7
Sand and clay—poorly graded	Fairly hard	1.40—1.75	10—20
Sand, sandy clay, gravel—fairly graded	Hard-good	2.10	25
Compact clays	Hard-good	2.43	30
Gravel or kankar with fines—well graded	Very hard	2.80—3.60	40—80

*Bearing value is measured as the pressure which applied to the subgrade uniformly over a circular area 90 cm dia. causes a settlement of 2.5 mm.

Notes for Table

(*i*) The table should be considered operative up to single axle loads of 8200 kg (or 4100 kg. wheel load) and tandum axle loads of 14500 kg. Beyond axle loads of this magnitude, an increase over the thickness shown in the table will be necessary. Tyre inflation pressure ranges from 5.3 to 6.3 kg/sq. cm. for most commercial vehicles.

(*ii*) The CBR values are based on laboratory tests according to IS:2720.

(*iii*) The thickness given is the total thickness of the pavement (surface, base and sub-base) over any sub-grade or sub-base of known CBR. The thickness of different layers of sub-base, base or surfacing can be determined by repeated use of the table, each having an increasing CBR value as the surface is approached.

(*iv*) Where provision of a sub-base is found necessary the thickness should not be less than 10 cm.

(*v*) Normally no material with CBR value lower than 80 per cent should be used in base course construction. The minimum thickness recommended for base courses is 15 cm even on minor roads.

(*vi*) Where the traffic intensity consists of heavy vehicles in excess of 1500 per day, it may be worthwhile to go in for "bound" layer to at least 5 to 7.5 cm thickness in the top of the base course prior to laying of the wearing surface. (Well consolidated bituminous macadam, lean cement concrete, soil cement, lime-puzzolana concrete, etc., are bound layers). Where a 15 cm thick water-bound macadam is indicated, 10 cm thick base could be considered as equivalent.

(*vii*) When the wearing surface consists of thin surface dressing or open-graded premix carpet 2.5 cm thickness, the thickness of surfacing should not be counted toward the total thickness of the pavement.

(*viii*) It is better to improve the sub-grade rather than to provide a thicker pavement. If the sub-grade can be made strong enough to carry the full load, it may only be necessary to superimpose upon it a skin to keep it waterproof and to prevent abrasion.

(*ix*) For the design, the sub-grade strength must always be assessed at the maximum moisture condition likely to occur during the entire service life of the pavement and the CBR value determined accordingly.

Description of Subgrades—General Foundation Properties :

(*a*) Very stable subgrades are well compacted and undisturbed foundations of old roads ; well graded gravelly—sandy soils—they compact well, with negligible settlement if properly compacted ; solid rock. No base is needed on such subgrades except where a levelling course is required.

(*b*) Stable subgrades are gravel-sand-clay soils which can be well compacted and where no base is needed. Other sandy and stable soils inferior to the above may be provided with 7.5 cm base.

(*c*) Poor subgrades are soils very susceptible to non-uniform movement : loams, peat and plastic clays. Silty soils generally give trouble.

It is better to remove silt to a depth of about 45 cm from the road surface and fill to subgrade level with coarser materials.

Peaty soils and black cotton soils : these are very compressible and are subject to seasonal volume changes. They should be avoided whenever possible or replaced with sound coarse materials.

(*d*) Subgrades where the water table may rise to within 60 cm of the foundation : No base is needed on soils which can be thoroughly compacted, otherwise a 7.5 to 15 cm base is desirable.

Where a hard base course has been given, higher value for the subgrade should be adopted.

For poor soils with CBR values of 4 to 5, the following approximate thickness of pavements may be taken :

State Highways	47 to 52 cm
Major District Roads	38 to 46 cm
Other Roads	20 to 30 cm
Village Roads	10 to 15 cm

Suitability of Road Material as Regards Density of Traffic :

Per day per traffic lane—

(*a*) Gravel roads	50 tonnes of iron tyred vehicles, or 80 to 100 tonnes of pneumatic tyred vehicles.
(*b*) Water-bound macadam	1000 tonnes of combined iron tyred and pneumatic tyred vehicles.
(*c*) Bituminous surfaces (surface treated with bitumen)	1200 tonnes of pneumatic tyred and 500 tonnes of iron tyred or 750 tonnes all iron tyred traffic.
(*d*) Pre-mixed bituminous or Grouted	5000 tonnes combined traffic, or 1200 tonnes iron tyred.
(*e*) Semi-grouted	3000 tonnes combined traffic.

The above figures are only rough approximations and largely depend upon how the road has been built.

Rate of Coverage of Base Materials :

(*a*) 1 tonne of rubble stone 15 cm thick covers 4 sq. m.

(*b*) 1 tonne of stone of size 2.5 to 4 cm of compacted thickness of 8 cm covers 0·8 sq. m.

(*c*) 1 tonne of rolled ash 2.5 cm thick covers 3 sq. m.

20. ROAD GRADIENTS

The rate of rise or fall of a road surface along its alignment is called its gradient or grade and is expressed as a ratio of one vertical to so many horizontal—which is the distance measured along the length of the road. Thus, if a road ascends or decends one metre for thirty metres of its length, the gradient is said to be 1 in 30. Gradient is also expressed as a percentage—1 in 30 is 1/30×100=3.33 per cent.

Ruling gradient is the desirable gradient which in the normal course must not be exceeded and when designing a road profile the aim should be not to exceed it except in exceptional circumstances. *Limiting gradient* is steeper than ruling gradient and which has to be used in restricted lengths where keeping within the ruling gradient is not feasible. *Exceptional gradient* is the steepest permissible gradient which may be used in short stretches only in extraordinary situations For better grip, more flexible and more rough the surface, steeper can be the gradient

Long stretches of steep gradients should be separated by comparatively easier grades or level lengths 35 to 100 metres long and which should be given every kilometre or so.

Difference in rise over 2 kilometres length shall not exceed 100 metres in mountainous terrains and 120 metres in steep terrains. Efforts should be made to provide flatter gradients at high altitudes as the efficiency of an automobile engine falls with increase in altitude Short and frequent changes in grades result in bumpy surfaces

Terrains Classification

Type of terrain	Cross slope of the country
Steep	greater than 60 per cent
Mountainous or Hilly	from 25 to 60 ,,
Rolling	from 10 to 25 ,,
Level, Plain or Flat	less than 10 ,,

	Ruling gradient	Limiting gradient	Exceptional gradient
Motor Roads			
(*a*) Roads in plains	1 in 30	1 in 20	1 in 15
(*b*) Mountainous terrain and steep terrain over 3000 m height above mean sea level	1 in 20	1 in 17	1 in 14
(*c*) Steep terrain up to 3000 m height above mean sea level	1 in 17	1 in 15	1 in 12
(*d*) Absolute max : gradient for short lenghts not exceeding 60 m in one km road length	—	1 in 12	
Jeep Roads	1 in 15	1 in 12	
(*a*) Absolute max : gradient for short lengths		1 in 10	
(*b*) Gradient at curves		1 in 30	
(*c*) Max:sustained grade with safety in operation		1 in 20	

Maximum gradient at hair-pin bends or other sharp curves with inside curve of 8 to 15 metres shall be 1 in 40 and which should never exceed 1 in 20 and should be as flat as possible for at least 30 metres above the hair-pin bends, even if it is necessary to increase the gradient for a short distance beyond this length of 30 metres.

Recommended Gradients for Urban Roads

A gradient of 1 in 25 should be considered the maximum for urban roads. On roads carrying predominently slow moving traffic, the gradient should not exceed 1 in 50.

The desirable maximum gradient for pedestrian ramps in 1 in 10 and for cycle tracks 1 in 30.

Exceptional gradient should not exceed 100 m at a stretch. Successive stretches of exceptional gradients must be separated by a minimum length of 100 m having gentler gradient.

Junctions should preferably be located away from steep gradients, especially where the road is on a curve.

For kerbed sections minimum gradient for drainage should be 1 in 200, when drain is lined & 1 in 100 when drains is unlined.

Gradients on curves should be eased as far as practicable. The grade figures given above are, strictly speaking, applicable to the straight portions of the roads. Even on slight curves the gradient should be eased.

Gradients steeper than 1 in 20 will often create drainage difficulties, as water will tend to run down the road surface. However, a minimum gradient of 1 in 250 for black-top roads and 1 in 330 for concrete roads is desirable for the flow of rain water.

Heavy commercial vehicles need much flatter slopes for operation, which may not be more than 1 in 30. Easy gradient should be provided as far as possible and the standards of alignment and design of the road should not be lowered to economise cost.

On out of town highways with design speed of about 75 km/hr., it is considered that vertical curves should be at least 150 metres long.

Commercial Vehicles: Heavy trucks can maintain a speed of 32 km/hr. on sustained grades of 3 per cent. The steepest grades on existing paved highways or streets in the USA are 9 per cent to 12 per cent for highways and 30 per cent to 32 per cent for urban streets. An average commercial vehicle can ascend a continuous 17 per cent grade in low gear—use only for ramps, accesses, driveways.

Passing facilities (added lanes or long sight distances) should be provided on mixed traffic roads where grades cannot be reduced to 3 per cent.

Passenger Vehicles: Automobiles can normally operate in high gear on maximum substained grades up to 7 per cent, 6 per cent is the maximum sustained grade for safe operation of trucks and automobiles. On mountain roads in high altitudes and areas subject to frequent ice, snow, sleet and fog, the maximum safe substained grade is about 5 per cent for all vehicles (USA).

The absolute maximum gradient which can be used by *animal traction* is 1 in 12. This is to be used only in hill roads where unavoidable, for not more than one kilorreter at a stretch. A gradient of 1 in 10 can be given where absolutely essential for distances not exceeding 90 metres. On cement concrete roads if iron-tyred bullock carts in large numbers are expected, a grade steeper than 1 in 20 should not be provided.

Steep gradients should be avoided at approaches to road junctions, roundabouts, bridges, acute bends and where the movement of traffic is restricted. All changes of gradient should be gradually effected by means of vertical curves of ample length. The surface of the road should be rough finished on all steep gradients.

All changes of grades exceeding a difference of one metre in level should be joined by vertical curves.

Loads can generally be hauled up a grade of 1 in 100 to 150 without much increase in motive power.

In a gradient of 1 in 20 the reduction in the load carrying capacity is only 20 per cent, whereas in the case of a gradient of 1 in 15, the reduction would be as high as 50 per cent.

Pack-Mule roads in hills :

Animals can draw only the following percentages of weight on an incline :

1 in 10	...	25 per cent	1 in 40	...	70 per cent
1 in 20	...	40 „	1 in 50	...	80 „
1 in 30	...	60 „	1 in 100	...	90 „

Maximum gradient for bridle paths or pack mules is 1 in 7.5, with 1 in 6 in lengths not exceeding 90 metres ; rise per kilometer must not exceed 140 metres. Curves with radius of less than 3 metres should not have any rise.

Maximum gradient for pedestrian paths is 1 in 5 (prefer 1 in 7.5).

In USSR the maximum longitudinal gradients are taken in relation to the design speed, as follows :

Design speed, km/hr.	150	120	100	80	60	50	40	30
Gradient, per cent	3	4	5	6	7	8	9	10

In all cases it will be good policy to design highways with longitudinal grades not exceeding 3 per cent, unless this leads to a considerable increase in quantities and cost of the construction work.

Critical Length of Grades (according to US standards) :

Where feasible, upgrade should not be of a length that causes loaden trucks to reduce speed unduly. On this basis, the critical lengths of upgrades when approached by level or nearly level sections of roads, are as follows :

Upgrade per cent	3	4	5	6	7	8
Critical length of upgrade—m	490	330	240	200	170	150

21. THE GEOMETRICS OF HIGHWAYS

Geometrical design standards generally represent an acceptable compromise between the road users' requirements and the construction costs.

Stopping Distance is the distance covered by a moving vehicle from the instant an obstacle on the road ahead becomes visible to the driver and the vehicle is brought to a stop. Minimum safe stopping distance is sometimes known as *non-passing sight distance.*

Sighting Distance or Sight Distance or Visibility Distance is the length of a road ahead of the vehicle which should be visible to the driver to enable him to bring his vehicle to a stop in case of an obstruction on the road. (A driver can see at a distance of about 300 metres in line with his eye along a level road). The minimum sight distance should be based on the expected speed of the vehicles and the breaking distance required to stop and which should also include the driver's reaction time to the danger. The safe sight distance is dependent on the breaking time of the vehicle and is generally taken as being equal to twice the distance required for a vehicle to stop. Mechanical and human factors should be taken into account – the breaks may be defective, the driver may be drunk, and these factors will need more of stopping distance and reaction time. The safe stopping sight distance should invariably be provided throughout the length of all roads.

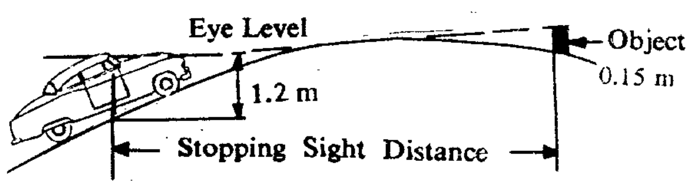
The Sighting Distance provided on roads should not be less than the safe Stopping Distance for the particular speed allowed on the road. Where the minimum stopping distance cannot be provided, warning boards should be fixed for reducing the speed.

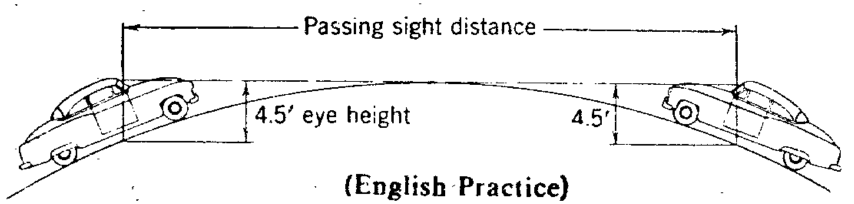
The eye level of drivers can vary from 1.05 m above the carriageway in a standard car to approximately 2 m in large eommercial vehicles.

For the purpose of measuring the stopping sight distance or visibility ahead, IRC has suggested the height of eye level of driver as 1.2 metres and the object height as 0.15 metre above the road surface. Hence stopping distance at a summit curve is that distance measured along the road surface at which an object of height 0.15 metre can be seen by a driver whose eye is at a height of 1.2 metres above the road surface.

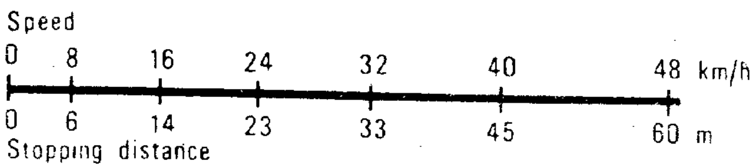
Sight distances, both vertical and horizontal, should be measured along the "eye level" (1.2 m) above the carriageway along the centre lines of both the rear side and outside lanes of the carriageway.

Vertical Curves





(English Practice)



Stopping Sight Distance and Overtaking (or Passing) Sight Distance for Various Speeds for Rural Highways

Design Speed km/hr.	Absolute min: Stopping Distance m	Min: Overtaking Sight Distance: single-lane carriageway two-way traffic* m	Min: Overtaking Sight Distance: two-lane (undivided) carriageway two-way traffic m	Min: Visibility Distance along major roads at intersections m
20	20	40	—	
25	25	50	—	
30	30	60	—	
40	45	90	165	—
50	60	120	235	110
60	80	160	300	135
65	90	180	340	145
80	120	240	470	180
100	180	360	640	220

* Opposing vehicles. Based on IRC : 66

(Absolute minimum sight distance is the safe stopping sight distance).

In single-lane roads where two-way movement of traffic is permitted, the minimum stopping sight distance should be equal to *twice* the stopping distance to enable both vehicles coming from opposite directions to see each other.

Restrictions to sight distance may be caused at horizontal curves or in vertical summit curves, by objects obstructing vision at the inner side of the road or at intersections.

Comparison of Design Criteria in Various Countries

	Driver eye height m	Object height m	Min: stopping sight distance of 120 km/hr. m	Min: stopping sight distance of 80 km/hr. m	Min: overtaking sight distance at 80 km/hr. m
Australia	1.14	0.23	238	122	488
Germany	1.0	zero	235	105	—
Finland	—	zero	250	120	—
France	1.0	0.15	230	105	—
Italy	—	—	210	90	—
Netherlands	—	—	250	130	—
U.K.	1.05	1.05	300	140	360
U.S.A.	1.14	0.15	287	137	549

"Geometric Road Design Standards" published by O.E.C.D., Paris.

A comparison of the design criteria adopted by various countries shows that there is fairly close agreement on eye height standards, but the object height adopted vary from zero to 1.05 m. The visibility of an object is a function of numerous factors.

Overtaking Sight Distance is the distance required for a moving vehicle to overtake and safely pass another vehicle moving in the same direction but at a lower speed. The minimum distance ahead that must be clear to permit safe passing is called the *passing sight distance*.

From actual tests made, a car travelling at 48 km per hour and passing a car at rest, requires 35.66 meters to turn on the traffic line and return to it. A car travelling at 96 km per hour and passing one travelling at 72 km per hour would require a distance of 143 meters. Hence for two cars coming from opposite direction at 96 km per hour a sight distance of 285 metres would be required.

Where for step gradients to the approaches a vertical curve cannot be avoided, the summit of the curve (hump) should be made horizontal for a length of about 30 metres.

Same visibility is recommended on horizontal curves as on vertical curves. The sight distance is measured along the centre line of the road.

Overtaking Zones. Minimum sight distance provision at each overtaking section should be equal to the overtaking sight distance. Overtaking zones of lengths not less than three times the distance given above should be provided at frequent intervals. Roads of 3.75 metres width and less where there are no berms such as canal and village roads, overtaking places of not less than 61 metres should be provided at frequent intervals.

Provision of Visibility on Curves

The visibility of horizontal curves should be checked for a vehicle proceeding along the inner nearside traffic lane. It is usual to assume that the driver's eye is situated 1.5 metres from the edge of the carriageway. The test sight is measured along the path of the vehicle.

Visibility across the inside of horizontal curves is very important. Lack of visibility in the lateral direction may arise due to obstructions like walls, cut slopes, buildings, wooded areas, high farm crops, etc.

The setback distance to give the desired sight distance on the inside of the horizontal curves on two-lane roads may be taken from the following table :

Sight distance is required to be considered for both horizontal and verticle curves. For horizontal curves the presence of obstacles to vision must also be considered. To provide visibility, hill sides have to be cut back at curves and bends. Whenever possible straight lengths should be provided on both the sides of bridge approaches. They should preferably be not than 30 meters in length, and should be as level as possible.

On a *vertical curve* the sighting distance represents the distance apart of two vehicles approaching from opposite direction when they just become visible to each other. In this case the height of the driver's eye above the road surface has to be taken into account in addition to the speed of the vehicle.

Minimum Set-back Distance "S" Required at Horizontal Curves for Two-Lane Carriageway for Safe Stopping Sight Distance "D"

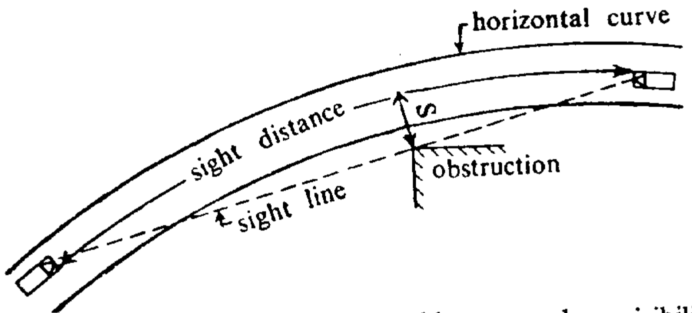
Radius of curve at road centre-line in metres	Set-back distance in metres					
	D 20 V 20	D 45 V 20	D 60 V 50	D 90 V 65	D 120 V 80	D 180 V 100
20	5.0	—				
50	3.2	7.0				
75	2.8	5.5	13			
100	2.5	5.0	11			
125	2.5	3.7	9.0	11		
150	2.5	3.6	7.0	9.8		
175	2.5	3.3	6.5	8.0	13	
200	2.5	3.0	6.0	7.6	11	
250	2.5	3.0	5.0	6.5	8.5	
275	2.5	3.0	5.0	6.0	8.0	17.5
300	2.5	3.0	4.5	6.0	7.5	15.5
350	2.4	2.8	4.0	5.0	7.0	13.0
400	2.4	2.7	3.5	4.5	6.5	12.9
450	2.4	2.6	3.0	4.4	6.0	11.0
500	2.4	2.5	3.0	4.0	5.5	10.2
550	2.4	2.5	3.0	4.0	5.4	9.5
600	2.3	2.5	3.0	4.0	5.0	9.3

D is safe stopping sight distance in metres, V is design speed of the road in km/hour.

"S"—the min: set-back distance (as shown in the illustration) to sight obstruction is measured from the centre line of the carriageway.

Minimum Set-back Distance "S" Required at Horizontal Curves for Single-Lane Carriageway for Safe Stopping Sight Distance "D"

Radius of curve at road centre-line in metres	Set-back distance in metres				
	D 20 V 20	D 25 V 25	D 30 V 30	D 45 V 40	D 60 V 50
14	3.4	—	—	—	—
15	3.2	—	—	—	—
20	2.4	3.8	—	—	
23	2.1	3.3			
30	1.7	2.6	3.7		
33	1.5	2.3	3.4		—
50	1.0	1.6	2.2	5.0	—
60	—	1.3	1.9	4.2	
80		—	1.4	3.1	5.6
100			1.1	2.5	4.5
120			0.9	2.1	3 7
150				1.7	·2.3



The alignment of hill roads should ensure clear visibility up to the min: stopping sight distance all along its length. It will be impracticable normally to provide visibility corresponding to the overtaking sight distance. Vision berms should be provided on the inside of horizontal curves up to a height of 0.6 metre. The width of these berms should equal the set-back distance minus the width of drain.

While designing for safe visibility distance, it should not be ignored that drivers' visual difficulty is magnified by, hazy visibility during the monsoons.

Lateral and Vertical Clearances at Underpasses for Vehicular Traffic

Lateral clearance is the distance between the extreme edge of the carriageway and any nearest projection. Vertical clearance is the height above the highest point of a carriageway to the lowest point of the overhead structure.

Lateral Clearance for Urban Roads

There should be a minimum lateral clearance of 1.0 m from the edge of pavement for a pavement without footpath, for arterial and sub-arterial roads, and 0.5 m for collector and local streets. For pavements with footpaths no extra clearance beyond the footpaths is necessary.

Lateral Clearance for Rural Roads

The full roadway width at the approaches should be carried through the underpass - the same roadway width as recommended for culverts in the Table - this implies that the minimum lateral clearance is equal the normal shoulder width.

Vertical Clearance at underpasses for full width of the urban roads should be 5 5 m minimum.

Vertical Clearance for Railway electric traction (Broad gauge) is 5.87 m and 4.88 m minimum for non-electric traction.

The minimum width and vertical clearance for *Pedestrian Subways* and *Cycle Subways* is 2.5 m. The width of pedestrian-cum-cycle subway should be 5 m minimum for one-way traffic and 6.5 m for two-way traffic. The minimum height should be 2.5 m.

Passing Places or Lay-byes for Hill Roads

There is no need of providing passing places on two-lane National and State Highways having roadway widths as recommended. But in the case of single lane sections on Nationat/State Highways which have a narrower roadway, provision of some passing places will be desirable.

On other catagories of roads, these should be provided in general at the rate of 2 or 3 per kilometre. Normally these should be 3.75 m wide, 30 m long on the inside edge and 20 m long on the outside edge.

Camber is provided on all the straight reaches of a road and super-elevation is given to the curved position. On superelevated sections, the shoulders should normally have the same cross-fall as the pavement.

The cross-fall for earth shoulders should be atleast 0.5 per cent more than the pavement camber subject to a minimum of 3 per cent. If the shoulders are paved, a cross-fall appropriate to the type of surface as given above should be selected.

Clear Width of Roadway Between Kerbs at Bridges

Single-lane bridge	4.25 m
Two-lane bridge	7 5 m
Multilane bridge	3.5 m per lane plus 0.5 m for each carriageway.

Full roadway width should be carried through on culverts, scuppers and causways, (measured from inside to inside of parapet walls or kerbs) A width of 5.5 m is normaly required to enable the largest vehicles to pass each other, but most of the vehicles, however, will be private cars

Design Speeds for Different Classes of Roads for Various Terrains in kilometres/hour

Classification of Road	Terrain			
	Plain	Rolling	Mountainous	Steep
National & State Highways	80 to 100	65 to 80	40 to 50	30 to 40
Major Distt. Roads	65 to 80	60 to 65	30 to 40	20 to 30
Other Distt. Roads	50 to 65	40 to 50	25 to 30	20 to 25
Village Roads	40 to 50	35 to 40	20 to 25	20 to 25

For divided highways, the higher value may be adopted.

Turning speed should be taken at 0.7 of the speed of through road.

Mini roundabouts may also be used to reduce vehicle speeds along unbroken stretches of roads.

Design Speeds for Urban Roads-km/hour

Arterial	80
Sub-arterial	60
Collector street	50
Local street	30

A lower or higher value may be adopted depending on the presence of physical controls, roadside development and other related factors.

A lower design speed may be adopted in the central business areas, or areas with heavy road side development. In suburban areas however, a higher value need be adopted.

A design speed of 48 km/hr. is unlikely to be justified for residential roads simply on grounds of convenience or journey time. Lower speeds would often enable visibility splays to be reduced thereby permitting more economic use of land. Changes in road surface and gradients and mini roundabouts may also be used to reduce vehicle speeds along unbroken stretches of road. Restricted visibility in the absence of other precautions cannot be considered a safe means of reducing vehicle speeds.

Fuel Consumption at Various Speeds (comparison)

Speed km/hr.	10	15	20	25	30	35	40
Fuel km/litre	4.22	5.54	6.57	7.40	8.07	8.64	9.11

IRC Highway Research Bulletin No. 1-1975

Accident Risks as Related to Road Design Standards

Accident risk to pedestrians on a 'zebra' crossing at a "high speed" intersection (average vehicle speed more than 35 km/hr.) is more than twice the risk to pedestrians at a low speed intersection with identical road standard.

The risk to cyclists seems to be much higher than to pedestrians in locations with the same road design standard.

Experiments conducted in France indicate that there are twice as many accidents at cross roads than "Y" junctions at a speed limit of 50 km/hr. and four times more at 80 km/hr. At cross roads there are twice as many accidents at 80 km/hr. as at 50 km/hr. The effects of channellising traffic flow results in a 50 per cent reduction in accidents. Some engineers, however, disagree about the worth of staggered inter-sections. They consider that besides providing a poorer service, they are more dangerous than cross roads. Some engineers think on the contrary that given adequate length of the weaving zone, this type of intersection (staggered) was safer.

The summit radii in the longitudinal alignment and the radii of bends in the horizontal alignment should be made as long as possible for the changing visibility conditions arising from the lower driving positions of the more modern cars that are coming up.

The overtaking visibility distance should be provided on as great a percentage of the length of the road and as uniformly as possible.

22. CAMBER OR CROSS-FALL OR CROSS SLOPE

Excessive camber induces drivers to keep near the crown of the road and tends to uneconomic use of the highway. A cross-fall which is excessive increases tendency of vehicles to side-slipping. Therefore, excessive camber should be avoided and surface made as flat as can be permitted.

Surface type	Camber/Cross-fall
1. High type bitumenous surfacing or cement concrete	1 in 60 to 1 in 50 (1.7 to 2.0 per cent)
2. Thin bitumenous surfacing	1 in 50 to 1 in 40 (2.0 to 2.5 per cent)
3. Water-bound macadam or graval	1 in 40 to 1 in 30 (2.5 to 3.0 per cent)
4. Moorum or kankar roads	1 in 24 to 1 in 30 (4.0 to 3.0 per cent)
5. Earth roads and foot-paths	1 in 30 to 1 in 25 (3.0 to 4.0 per cent)

(The camber is from crown to edge of pavement)

Berms :

Paved or treated ... ... 1 in 24 to 48

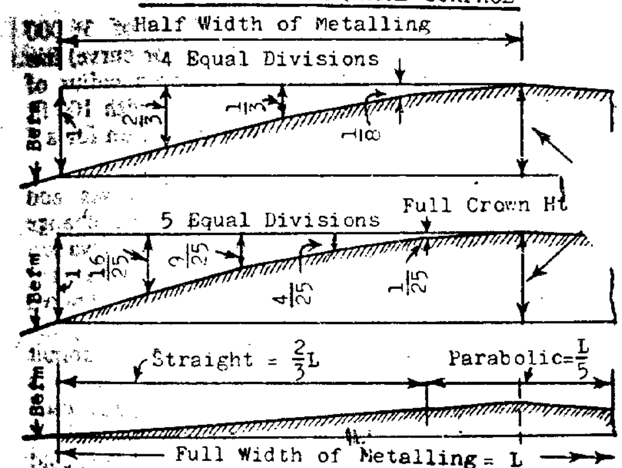
Kachha (earthen) ... ... 1 in 16 to 24

Camber is provided on all the straight reaches of a road and super-elevation is given to the curved portions.

Use steeper values in the Table for high rainfall areas and lower values where the intensity of rainfall is low.

The crowned surface is generally curved to the shape of a parabola or an ellipse which is flatter at the central half width and slightly steeper at the sides, but a combination of uniform slope with a parabolic curve, or two straight lines meeting at the crown may be used instead. For single-lane carriageways the camber should preferably be parabolic. While making for equal divisions for the half width, some engineers take 9/16, 1/4 and 1/16 instead of 2/3, 1/3 and 1/8 as shown in the illustration.

CAMBER CURVE FOR ROAD SURFACE



Since the object of cross-fall is to facilitate drainage of the surface water from the road the roughter the surface the steeper must be the camber. Road surfaces which are not expected to come up to the mark when completed due to lack of control or other reasons of foundations, etc., should also be given steeper cambers. Steeper cambers are also adopted for hill roads. Dry areas are made flatter than areas subject to heavy rainfalls. Shoulders are given greater cross slopes The necessary camber should be given in the sub-grade so that the hard crust is of uniform thickness throughout the section of the road. Wooden tamplates are made according to the proposed camber of the road, which are fixed at 15 to 30 metres apart across the centre line of the road.

When the road has longitudinal gradients greater than 1 in 20, flatter cambers may be provided.

23. ROAD CURVES

A *Simple curve* is a single circular arc connecting two tangents. The point at which the curve starts is called the "point of curvature" and the end of the curve the "point of tangency".

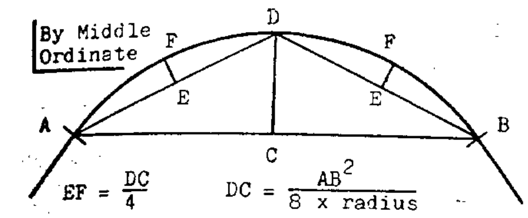
A *Compound curve* is formed by two adjoining simple curves (circular areas) of different radii which lie on the same side and have a common tangent and a common tangent point. Compound curves are not recommended for roads since the abrupt change of curvature provides an element of danger particularly at night. Much attention should be given to the flattening of the curves.

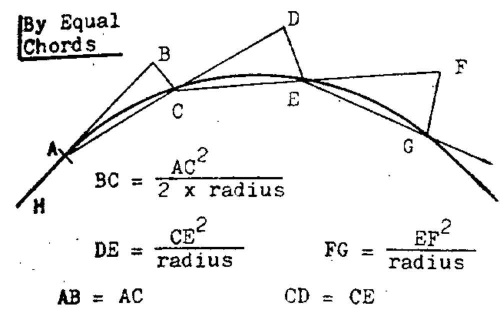
A curve is designated by its radius (in feet or metres), or by the number of degrees of the central angle covered (or subtended) by a 100 ft. (30.48 m) length of the chord of the curve. For pratical purposes the length of the chord and the arc is taken as the same. Thus a 1 deg. curve in which a chord of 100 ft. (30.48 m) is subtended by a central angle of 1 deg. It is called *Degree of Curvature.*

A circle with 5730 ft (1747 m) radius has a circumference of 36,000 ft. (1093 m) subtending 360 deg. A 100 ft. (30.48 m) chord (or curve) has 1 deg. Or in other words, a 1 deg. curve is considered to have a radius of 5730 ft. (1747 m), a 2 deg. curve 2865 ft. (874 m) and so on (with 100 ft. chords). This is not absolutely correct but a close approximation for a 5 deg. curve or any flatter curve.

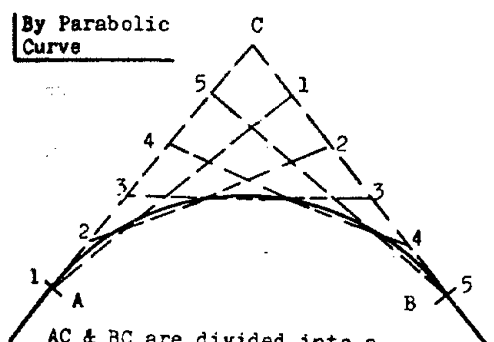
Curves on highways are mainly of two types– Horizontal curves and Vertical curves. A horizontal curve is a curve in plan provided to change the direction of an alignment. A vertical curve is a curve in elevation provided at change of gradient. There are two classes of vertical curves—(*a*) Convex or summit curves and, (*b*) Concave or valley curves, also known as 'sag' curves. The design of a summit curve is governed mainly by the 'sight distance' required to provide adequate visibility, unless the summit is so small (less than 3 ft. (0.91) above the lowest point of the curve) as not to interfere with visibility. In practice a simple parabola curve is used instead of a circular arc for a vertical curve, which joins the two gradients gradually. When two slopes meet, the change from one to the other should be gradual. This is achieved by using a vertical curve to unite the two gradients. Sharp points are particularly dangerous at the end of long straights or at the bottom of a hill.

SETTING OUT CURVES





Tangent HA is produced to B
AC is produced to D, CE is produced to F



AC & BC are divided into a number of equal parts; 1-1, 2-2, 3-3, etc. are joined. Inner intersections are on the curve.

Curves of small radii should be set out in chords not exceeding 1/20th of the radius. 100 ft. (30.48 m) chords to be used up to 5 deg., 50 ft. (15.24 m) chords for 5 deg. to 15 deg., 25 ft. (7.62 m) chords from 15 deg. to 30 deg. Curves of 1/2 mile (805 m) radius and upwards can be set out with 200 ft. (61 m) chords. Some engineers prefer to use 100 ft. (30.48 m) chords for radii 2000 ft. (610 m) or more. The difference in length between a chord of 100 ft. (30.48 m) and its arc is practically negligible. Refer to the first figure of the illustration. Curves can also be set out by making offsets from the long chord AB, by calculating a number of offsets

Setting out Curves : Setting out circular curves from a tangent point (or from the point of common tangency between a transition and a circular curve can be done by taping along, or more commonly by theodolite and tape.

A *Reversed curve* is formed by two simple curves in which a right-hand curve is followed immediatly by a left-hand curve, or *vice versa,* not necessarily of the same radii, which have a common tangent point. Reversed curves are ordinarily used to connect parallel lines. It is good practice to insert a length of straight 60 meters or an easement curve between the two curves, which introduce the change in radius gradually. Super-elevation is changed along this trasition section. Compounding circular curves of greatly different radii is considered poor practice.

Horizontal Road Curves

30.5 metres is the minimum turning radius for large lorries.

California practice is to have a minimum radius of 61 metres for "blind" curves and 30.5 metres for "open" curves. (Blind curves are outside curves round the hill or spur and open curves are inside curves heading a gully or a ravine.)

A road curve should be at least 152 metres long for $\phi = 5$ deg. which should be increased 30.5 metres in length for each decrease of 1 deg. in the ϕ. Where topography permits, use simple 20 minutes to 1 deg. curves without super-elevation or widening.

The straight portion in-between two curves in an S-curve should not be less than 152 metres.

British Ministry of Transport Recommendations

When speed is restricted to 48 km/hr. (30 miles/hr.) curves should have radius of not less than 305 metres (1000 ft.) for through roads and 152 metres (500 ft.) for roads passing through densely populated areas. On reverse curves a short length of straight should be introduced between the two curves. Road width should be increased by 30 cm per traffic lane for curves between 305 to 457 metres, and 46 cm for radius of less than 305 metres. Transition lengths on each side of the curve will be 31 to 46 metres minimum.

At **Railway Crossings** the minimum curve should be of 61 metres radius in the case of main roads and 46 metres in the case of other metalled roads, measured to the centre of the road. The angle of crossing should not be less than 45 deg.

The following alignment rules should be borne in mind :—

(*a*) A horizontal curve should not begin at a summit ; it is better to suggest the change of direction before the road goes out of sight.

(*b*) Combination of vertical summit curves and sharp horizontal curves is dangerous.

(*c*) Curvature at the bottom of steep grades should be avoided.

(*d*) In order to provide good visibility it is necessary to avoid as much as possible sharp changes from tangents to short radius curves and from long radius to short radius curves. Whenever possible long tangents should be joined to long radius curves.

(*e*) Where, for unavoidable reasons, there have to be rather sharp vertical and horizontal curves on summits, the horizontal curve should extend, if possible, beyond the ends of the vertical curve in order to call attention to the change in alignment.

Reverse curves are designed to have a minimum radius of 30 metres for the compound curves and a straight distance of 9 m between their transitional ends. In exceptional cases, the radius can be reduced to 22.5 m and the straight distance is dispensed with. A minimum radius of 18.3 m (60 ft.), with 12.2 m (40 ft.) in exceptional cases have been used on some of the hill roads in India.

Minimum Radii for Horizontal Curves

Design speed km/hr.	Desirable min: radii — Flat or rolling country m	Desirable min: radii — Urban areas m	Absolute min: radii m
25	—	50	30
30	100	60	50
40	130	100	60
50	170	125	100
60	250	—	155
80	300	—	250
100	500	—	370

Minimum Radii of Horizontal Curves for Various Classes of Hill Roads

Road Classification	Mountainous terrain				Steep terrain			
	Areas not affected by snow		Snow-bound areas		Areas not affected by snow		Snow-bound areas	
	Ruling min : (m)	Absolute min : (m)	Ruling min : (m)	Absolute min : (m)	Ruling min : (m)	Absolute min : (m)	Ruling min : (m)	Absolute min : (m)
1. National Highways & State Highways	80	50	90	60	50	30	60	33
2. Major District Roads	50	30	60	33	30	14	33	15
3. Other District Roads	30	20	33	23	20	14	23	15
4. Village Roads				15	20	14	23	15

Ruling minimum and absolute minimum radii are for ruling design speed and minimum design speed respectively. (IRC : 52-1981)

Transition (easement or spiral) Curves

A transition curve has a progressively increasing (or decreasing) radius used in joining a tangent with a simple circular curve, or in joining two circular curves of different radii.

With fast moving traffic on sharp curves it is essential that the effect of super-elevation provided on the curve be gradually felt to have a comfortable ride and for this transition curves are introduced for the purpose of connecting a tangent (straight line) with a circular curve in such a manner that the change of direction and elevation from one to the other takes place gradually. Transitions provide a convenient "running in" length for changing from natural camber to super-elevation. Natural camber is changed over to super-elevation in the transition length of the curve, (in entering a super-elevated curve camber should always be eliminated on the curve itself and never on the straight) and the full extent of the super-elevation necessary is attained before the vehicle begins to run along the curve. As a working rule, transition curves should be applied to all curves needing super-elevation. A simple method for achieving this transition is to provide a circular curve of long radius on the inner side of the road curve. If the transition curve is laid throughout and if the inner edge and the outer edge curves are laid out independently the road width will be widened at the curve.

The length of a transition curve depends on the radius and the design speed of the vehicle. The sharper the radius the longer the transition length.

Since the road width is also widened at the circular curve, both widening and super-elevation are provided at a uniformly increasing rate in the transition length.

The minimum length of transition provided for hill curves shall be as under (IRC) :

(*a*) 15 m for design speeds up to 40 km/hour.
(*b*) 20 m for design speeds 40 to 50 km/hour.

Minimum Transition Lengths for Various Speeds and Curve Radii (IRC)

Curve Radius in metres	Plain & Rolling Country					Hilly Tracts				
	Design Speed in km/hour									
	30	50	60	80	100	20	25	30	40	[illegible]
20	—					20	35			
30	80					15	25	30		
40	65						20	25		
50	50						15	20		
60	45							20		
70	—							15		
80	40							—	25	
90	30								25	
100	25	70							20	[illegible]
125	—								15	[illegible]
150	20	45	65							[illegible]
200	15	35	50							[illegible]
250	—	30	40	85						[illegible]
300		25	35	75						
400		20	25	55	100					
500		—	20	45	90					
600			—	35	80					
800				30	60					
1000				—	50					

Curves having radius greater than 1000 metres need not necessarily be provided with transitions.

Hill Curves

A road going on the outer side of a hill or spur has a convex curve which is known as a *salient curve or blind curve*. Inside curve at the valley of a hill and which is concave curve is known as *re-entrant curve or open curve*.—Surpentine curves, hairpin bends, and corner ends have also to be provided in hill allignments.

Hair-Pin Bends

Hair-pin bends are circular curves with transition curves at each end. Compound circular curves can be provided instead of hair-pin bends.

The following design criteria should be adopted normally for the design of hair-pin bends :

(a)	Min : design speed	20 km/hour
(b)	Min : roadway width at apex :	
	(i) National/state highways	11.5 m for double-lane 9.0 m for single-lane
	(ii) District roads	7.5 m
	(iii) Village roads	6.5 m
(c)	Min : radius for inner curve	14.0 m
(d)	Min : length of the transition	15 0 m
(e)	Gradient—max : 1 in 40; min : 1 in 200 Approach gradients should not be steeper than 1 in 20 for 40 metres. (See page 18/139)	
(f)	Superelevation in circular portion of the curve to be 1 in 10. This can be reduced to 1 in 15 for light traffic roads.	

Inner and outer edges shall be concentric with respect to centre line of the pavement.

Straight length between two successive hair-pin bends should be 60 m min; excluding the length of circular and transition curves.

Vertical Curves or Summit Curves

A curve with convexity upwards is called a summit curve or vertical curve. The design of a vertical curve (length) is principally governed by the sight distance required to provide adequate visibility, unless the summit is so small (less than 0.9 m above lowest point of the curve) as not to interfere with visibility. In practice a simple parabola curve is used instead of a circular arc for a vertical curve, which joins the two gradients tangentially. Vertical curves are necessary at the intersections of different grades to smoothen out the vertical profile.

The minimum radius of a vertical curve depends on the design speed of the road and the required safe stopping sight distance. Minimum radii and lengths for different speeds recommended by IRC are given in the following table.

Speed-Km/hour	25	30	35	40	50	60	80	100
Min : radius (m)	60	105	135	165	240	415	650	930
Min : length (m)	10	12	15	20	30	40	50	60

Curves designed for min: sight distance give radii greater than the values given above. A simple Russian engineers' formula is given below for the two opposing vehicles of the same height of eye level :

$$R=\frac{L^2}{9.5}$$

Where L is the sight distance for the particular design speed of the road.

To provide adequate visibility at night the radii of concave vertical curves should be from 0.35 to 0.5 of the convex curve radii. The lengths of vertical convex curves shall be at least 300 metres and those of vertical concave curves at least 100 metres.

Widening of Carriageway at Curves (metres)

Radius of Curve (m)	Single-lane	Two-lanes
up to 20	0.9	1.5
21—40	0.6	1.5
41—60	0.6	1.2
61—100	nil	0.9
101—300	nil	0.6
above 300	nil	nil

Extra widening should be equally distributed on the inner and outer sides and it may be on the inner side only in the case of sharp curves with radius of less than 60 metres. (Prefer inside widening for cart traffic.)

Where transition curves are not used, all widening is added to the inside edge of the pavement.

According to American practice no widening of curves need be done for radius flatter than 1000 ft. (305 m) for roads designed for a speed of 50 miles/hr. (80 km/hr.) and for radius flatter than 700 ft. (213 m) for speeds of less than 30 miles (48 km).

The widening will start at the beginning or tangent point of the transition curve and progressively increased at a uniform rate till the maximum designed widening is reached at a point in the transition curve where the full designed superelevation is reached. Thereafter the same widening will be continued till a similar point in the farther transition is reached where the designed superelvation starts reducing.

Above 150 m radius of curve no widening need be done.

For pavements wider than the standard width, *i.e.*, 3.75 m for single-lane and 7 m for two-lane roads, values in the above table should be reduced by excess over the standard width. Negative values are to be ignored. For pavements narrower than the standard width, values in the table should be correspondingly increased. Widening should be equally distributed on the inner and outer sides except in the case of sharp curves of radii less than 50 metres, where the widening should be effected on the inner side alone.

24. SUPER-ELEVATION OR BANKING

Transverse inclination to the pavement surface is known as super-elevation. A vehicle travelling a curved path on a flat surface has a tendency to slide outward or to overturn about the points of contact between the outer wheels and the pavement, depending on the sharpness of the curve and the weight and speed of the vehicle. To offset this the roadway surface is sloped upward towards the outside of the curve.

Superelevation to be provided on curves is calculated from the following formula :

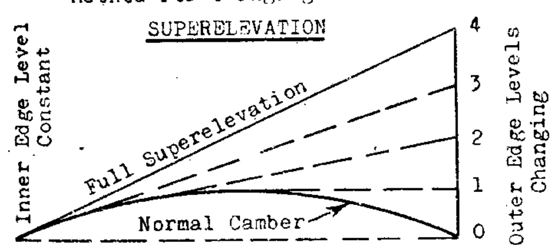
$$e=\frac{V^2}{225R} \quad \text{(IRC)}$$

Where :

e = superelevation in metres per metre width of carriageway;
V = design speed in km/hr;
R = radius of the curve in metres.

For Indian conditions (as regards loaded cart traffic) a superelevation of 1 in 15 for plain/rolling terrain and snow-bound hill roads; 1 in 25 for urban roads and 1 in 10 for hill roads not affected by snow, is considered appropriate. The greater the superelevation, more the inconvenience to the slow-moving traffic with danger of side slip. Superelevation where provided shall not be less than the camber of the road recommended for the particular road type to facilitate drainage—appropriate to the type of wearing surface. The change over from normal section to fully superelevated section shall be achieved uniformly over the length of the transition curve.

Method For Changing From Normal CAMBER to SUPERELEVATION



There are in general two methods of applying super-elevation: (*a*) Outer edge super-elevated and inner edge depressed, (*b*) Grade at inner edge retained and outer edge super-elevated.

Radius Beyond which no Super-elevation is Required

Classification of road	Radius in metres
(*a*) National & State Highways	2300
(*b*) Major District Roads	2000
(*c*) Other District Roads	1400
(*d*) Village Roads	600

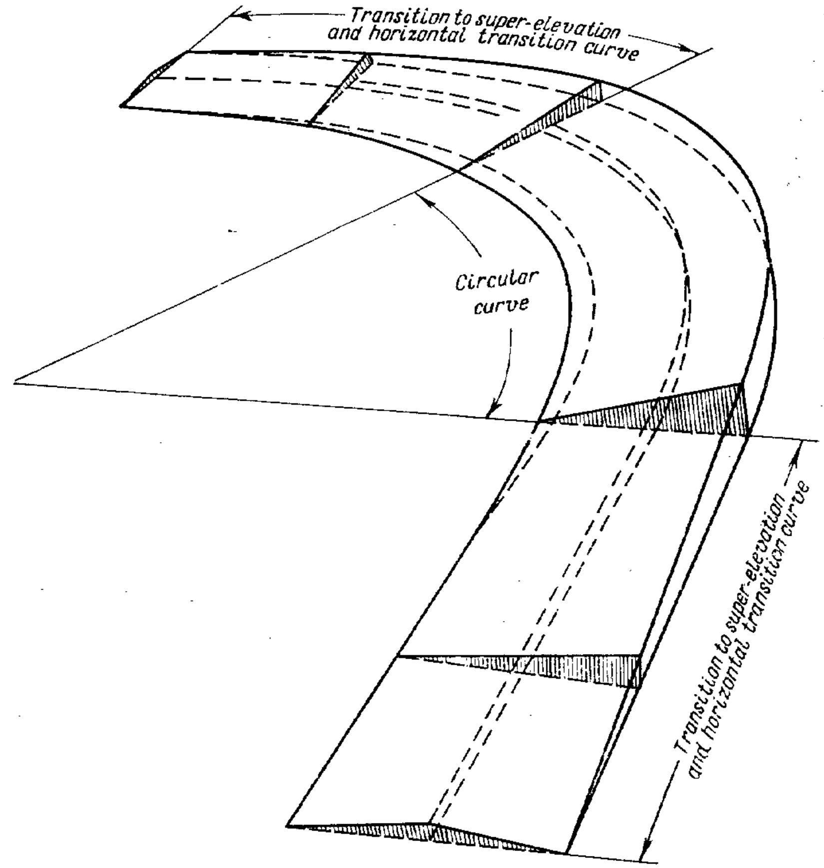


Diagram showing application of super-elevation to a cambered carriageway

Radii Beyond which no super-elevation is Required

Design speed km/hr.	Radius in metres for Camber of 1 in 30	1 in 40	1 in 50	1 in 60
30	130	160	200	240
50	370	450	550	650
60	540	640	800	940
80	950	1100	1400	1700

Minimum Radii in metres of Horizontal Curves when super-elevation is limited to:

Design speed km/hr.	Superelevation 1 in 15	1 in 25
30	30	40
50	90	105
60	130	150
80	230	265

25. TRAFFIC ENGINEERING

Road Crossings

Best crossing is when the side road joins the major road at a right angle. Junctions of lesser important roads with a more important one should be at right angles, and adequate provision should be made for visibility (line of sight) extending for a greater distance along the major road than along the minor road.

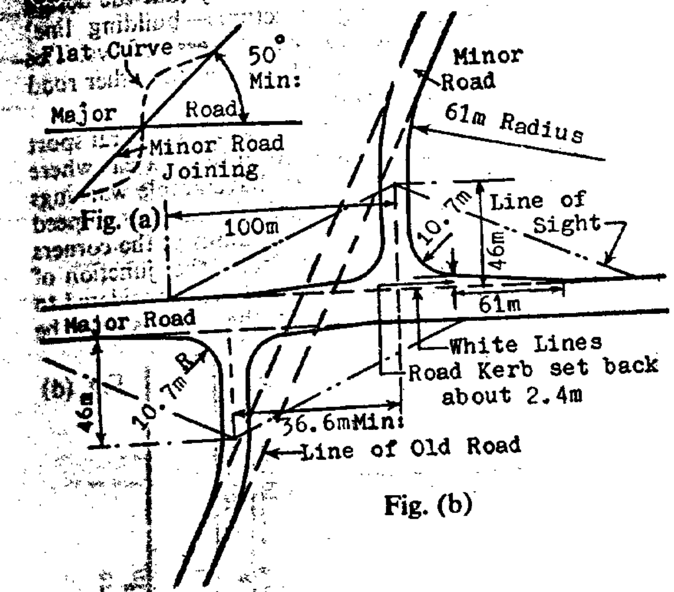


Fig. (b)

When the minor road is of little importance relatively to the main road, or when both roads are unimportant, the crossing of the minor road should be staggered as shown in the illustration. When the side road is important but not sufficiently so as to justify a " round-about ", the "baffle" junction may be used by making separate turning lanes (with an island in-between the two) for the side road.

The two illustrations show the methods of alignment of new roads or re-alignment of the old roads. (Based on British Ministry of Transport recommendations.) The radius of the kerb line at the junctions is 10.7 m which is sufficient to enable even the largest vehicles to keep close to the kerb when turning. The additional space for manoeuvering and particularly for decelerating and accelerating with minimum interference with the flow of traffic on the major road, is provided by tapered widenings of the carriageway to the extent of 2.4 m in a length of 61 m.

Intersection of highways on sharp curves should be avoided if possible. It is also desirable to avoid vertical curves at intersections ; no junction should occur at a change of grade. The whole of the junction area should be in one plane and as nearly as possible in that of the main road. Adequate sight distance must be provided along all roads and across their common corners.

Design of Road Juntions

Visibility at Corners, Bends and Junctions

Road intersections are broadly grouped under two headings :

(*i*) Uncontrolled intersections where the intersecting roads are more or less of equal importance and there is no established priority.

(*ii*) Priority intersections, like minor-major road intersections, where one road takes virtual precedence over the other.

It is of first importance in the interest of road safety that the design of the roads layout (and those of the adjoining structures—building line) must provide for a clear line of sight to enable drivers of vehicles approaching the junction to have a full view of the traffic on the other road at a sufficient distance, to reduce speed if necessary.

The following illustrations (Based on the British Ministry of Transport recommendations) show the sight distance required in town areas where appropriate traffic signs are provided and the drivers have ample warnings of the presence of the junction and they do not normally travel at a speed in excess of about 30 km/hr. This is formed by the buildings at the corners splayed to a line joining points 4.6 m back each way from the junction of the building lines. The foot-paths on both the roads are considered to have 4.6 m of width. Where footways are narrower the splay should be increased proportionately.

Fig. (c) Fig. (d)

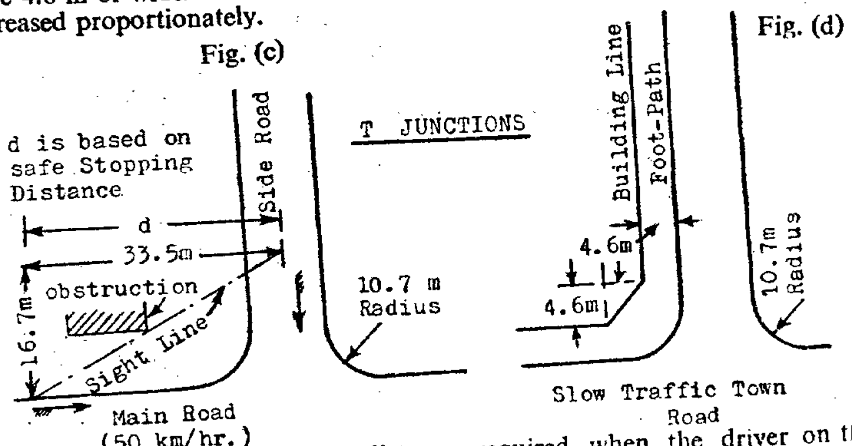


Fig. (c) shows the sight distance required when the driver on the through road of the T maintains a speed of 50 km/hr. and the drivers approaching from a minor road should have a view along the major road for a greater distance than at normal junctions.

In the case of a junction in open country the sight distance on the major road should be 100 m and on the minor road 46 m to enable the drivers approaching from a minor road to have view of the major road (Fig. (a)).

Min: Visibility Distance Along Major Roads at Priority Intersection for Rural Roads (IRC)

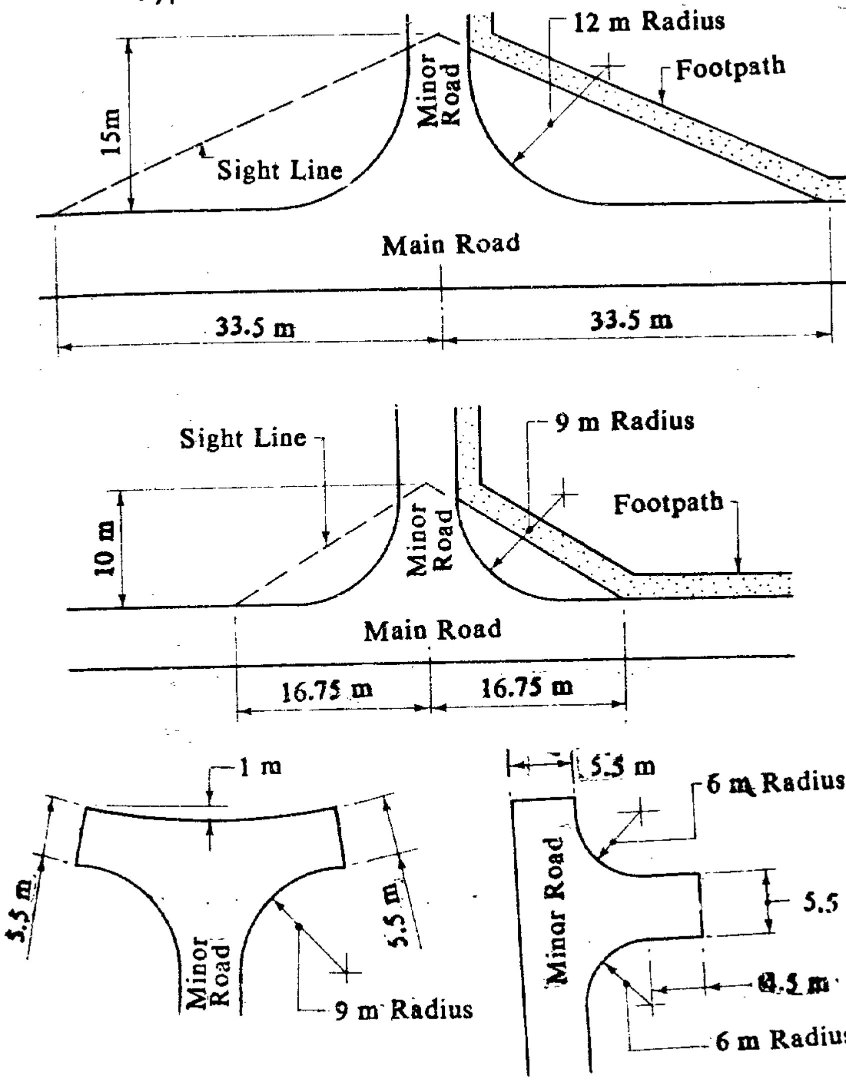
Design speed of major road	km/hr.	100	80	65	50
Min: visibility distance along major roads	metres	220	180	145	110

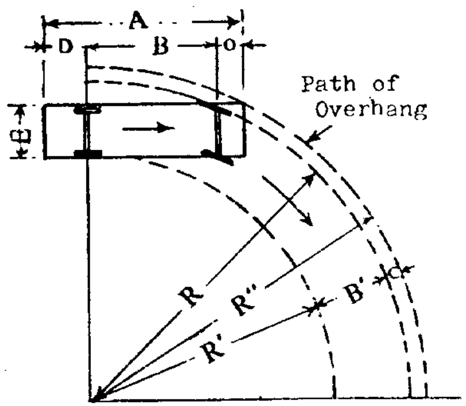
IRC recommend a min: visibility distance of 15 metres along the minor road. On this basis, the sight triangle at priority intersections should be formed by measuring 15 metres along the minor road and the distance given in the table along the major road.

No obstruction should exist between the sight line and the road kerb.

T-junctions with carriageway of more than 17 metres width with another of similar size should preferably be controlled by a rotary.

Typical Road Junctions & Terminations





SHARPEST RIGHT ANGLE TURN

The min: turning radius of a vehicle is the radius of the sharpest curve that could be traced by its outer front wheel. When a vehicle is turning, the rear wheels do not follow the tracks of the front wheels and the track followed by the inner rear wheel normally determines the radius and shape of curve to which the kerb line is set.

If road kerbs are designed with a turning circle of 10.7 m radius, most of the vehicles can manipulate easily; and which should be 17 m radius for roads used by buses and trucks, or tractor and trailer combinations. In congested town areas used mainly by light cars, a kerb radius of 4.6 m should be regarded as absolute minimum and it must have road width of at least 4.6 m. The vehicles can turn only with very slow speeds. Where the angle is more than 90-deg. greater radius will be required.

When the kerb radius is less than 4.6 m it is advisable to adopt a curve laid to two different radii rather than use a circular curve, a smaller curve at the commencement of the turn and the larger curve at the end of the turn.

Small curve radii are used in town areas to restrict speed for safety.

Some American engineers recommend for important 90-deg. intersections which have to accommodate large trucks, a three-centered compound curve of minimum radii successively of 36.6 m, 11.6 m and 36.6 m. With this, minimum lane width near the centre of the curve will approach 6 m for turning large trucks.

Certain design features of complicated intersections can be tested by actual use. Temporary channelizing island can be made of sand bags which can be easily shifted around. Sand sprinkled over the carriageways will indicate vehicle paths.

Min: Turning Radii for Different Classes of Vehicles

Type of vehicle	Length	Width	Path width	Over-hang	Min. desirable radius			Min. right angle turn radius		
	A	E	B'	C	R	R'	R''	R	R'	R''
Car-m	6.1	2.1	2.6	0.40	11.7	9.1	12.3	8.5	5.9	8.9
Bus or Truck-m	10.67	2.44	3.9	0.61	18.7	16.8	19.3	13.7	11.4	14.3

Maximum Dimensions of Road Vehicles

Width		2.44 m
Height	(a) Single-decked vehicle	3.81 m
	(b) Double-decked vehicle	4.72 m
Length	(a) Single unit with two axles	10.67 m
	(b) Single unit with more than two axles	12.19 m
	(c) Semi-trailer tractor combination	15.24 m
	(d) Tractor and trailer combination	18.29 m

Usual dimensions of vehicles in India are:

Bus 8 m long, 2.4 m wide;

Cycle riksha—2 m long, 1.25 m wide.

Acute Angles and Y Junctions

It is undesirable that roads shall join each other at an acute angle. Where an acute angle must be made the roads should be so designed or located as to intersect at an angle of 50 deg. min: with 30 deg. absolute min: ; prefer not less than 60 deg. It has been established that all things considered, the intersection of traffic streams at about right angles (75 deg. to 106 deg.) is most favourable.

Inset figure show the alternative method of treating an acute junction where it is not possible to re-align it at a right angle. Use easy flat curves at junctions where roads join at an angle less than 90 deg.

Min: turning radii at junctions for different intersecting angles are given in the following table :—

Minimum Turning Radii at Junctions
Turning Speed 30 km/hour

Angle of Junction	Radius for Cars up to 6.1 m length	Radius for Bus or Truck
90-deg.	9.1 m—10.7 m	15.2 m—16.8 m
105-deg.	10.7 m	18.3 m
120-deg.	13.7 m	21.3 m
135-deg.	18.3 m	27.4 m
150-deg.	36.6 m	45.7 m

Camber or Cross-fall at Junctions

The camber of the side road should not be carried into the major road; it should meet the channel of the main road, the camber of which should continue unchanged across the junction.

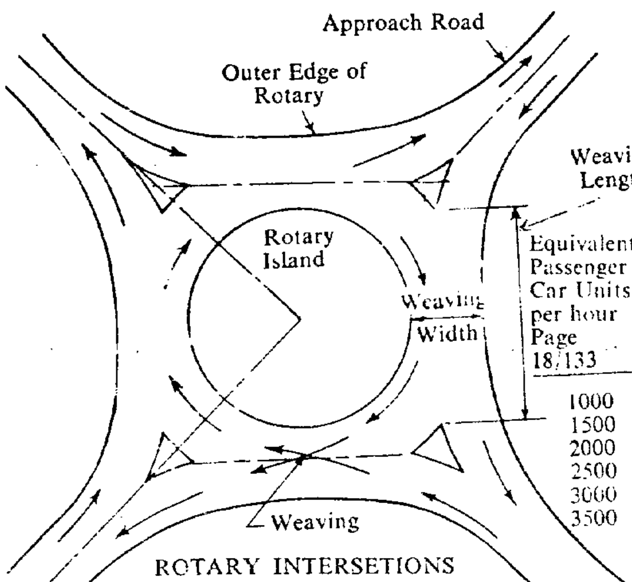
The crown of the main road should be a continuous line, and the necessary changes of cross-section for marrying in with the crown of the intersecting road should be as gradual as possible.

Roundabouts or Traffic Rotaries

Roundabouts are provided where two or more roads cross each other and where the traffic density exceeds 500 vehicles (passenger cars) per hour of all the intersecting roads. In UK they recommend the provision of a rotary at the crossing of a light trafficked and a heavily trafficked road in which the volume of traffic entering from the minor road is more than 25 per cent of all the traffic entering the intersection, or where many vehicles make a right hand turn. Some engineers recommend the adoption of rotaries where the right hand turn is 50 per cent or where the cross traffic is 30 per cent or more of the total and the traffic density of over 400 vehicles. As a rough guide, at a fourlegged junction, a rotary is more justified than a traffic signal control if the right-turning traffic exceeds about 30 per cent of all approaching traffic.

The maximum volume that a traffic rotary can handle efficiently can be taken about 3,000 vehicles per hour under Indian traffic condition, entering from all intersection legs. (Where most of the traffic is only passenger cars, 5000 vehicles per hour can manupulate).

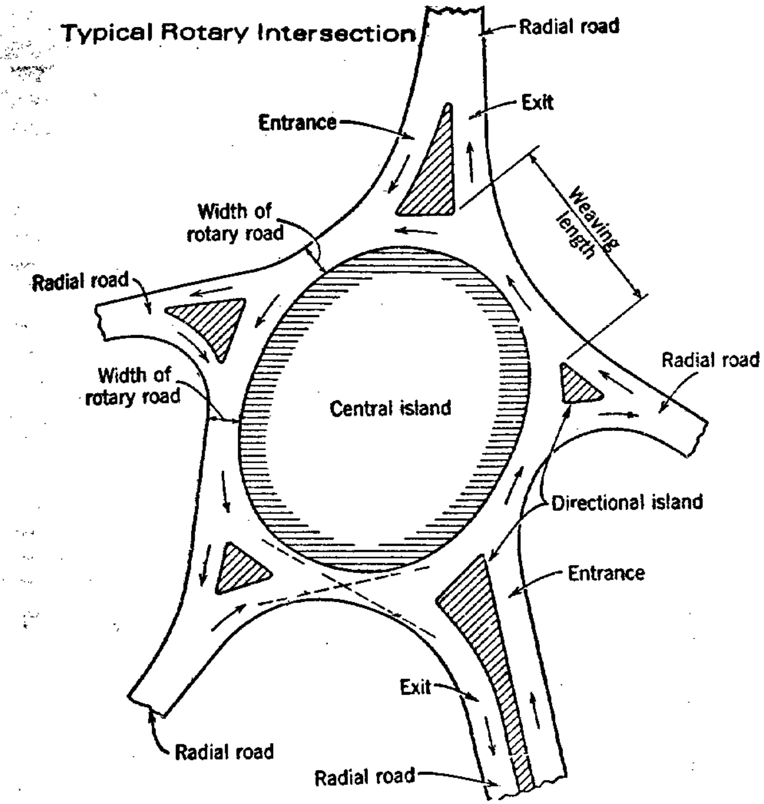
A roundabout imposes restraint on the speed of all streams and all classes of traffic, but provides for continuous movement. All traffic except that making a left-hand turn is forced to make a detour but even on a large roundabout this is not incommensurate with the advantage gained. Roundabouts are not suitable where the angle of intersection of two roads is very acute, where the distance between the intersections tends to be small, or where pedestrian traffic is large.



Equivalent Passenger Car Units per hour Page 18/133	Minimum Length of Weaving Section metres
1000	75
1500	120
2000	200
2500	300
3000	420
3500	560

ROTARY INTERSETIONS

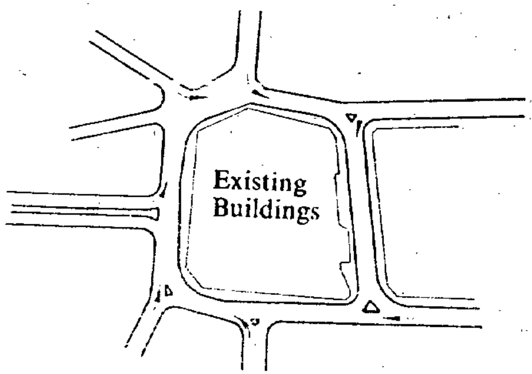
IRC : 65

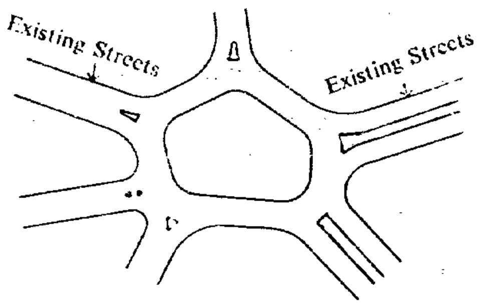


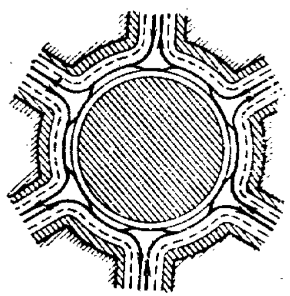
Shape of the Central Island

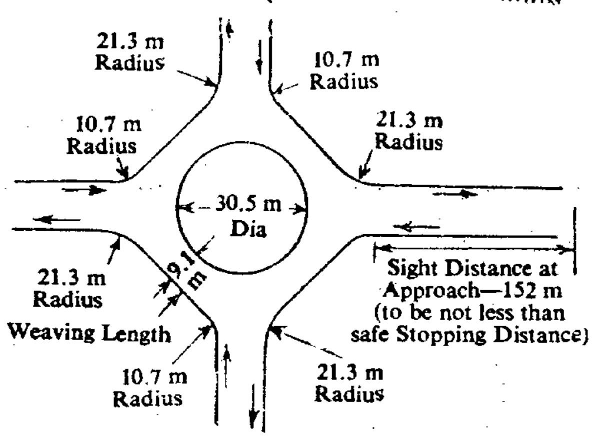
Simple geometric figures such as the circle or ellipse may not be the most suitable where the angles between the approach roads are irregular and the site is constricted. Asymmetric shapes, either wholly curved or with a combination of straights and curves as shown in Diagrams will often provide the only satisfactory solution.

Pavement widths should at least be equivalent to half the width of the widest radial road — the width of one lane. Maximum number of lanes should be four. Reflectorized or lighted signs should be provided well in advance of the rotary to warn traffic. Whenever feasible the entire rotary should be illuminated.









For counting equivalents of trucks or slow moving vehicles to passenger vehicles, see under "Traffic Counts. Equivalency factor for vehicles." Heavy vehicles are lesser at peak hours than for average conditions. According to IRC the passenger car unit equivalents may be taken as follows : See Page 18/133

Shape, Size and Design of Traffic Islands. The most usual shapes are circular or elliptical (elongated) and depend upon the site conditions, angles of intersections of the roads and the amount of traffic on the particular roads. Circular shape is the most economical and is suitable where all the roads carry nearly equal volumes of traffic and intersect at nearly equal angles. Elongated shape is adopted at intersections where the cross traffic is small ; the elongation should be in the direction of the greater flow and it should be so limited that the radius of curvature at the sharpest point is not less than 15.2 metres and the island is not elongated too much so that there is not large difference between the speeds of the traffic of the major and minor roads. The exact shape is determined by connecting the sections of the road round the central island to form a closed figure giving at least the minimum weaving length between adjacent radial roads.

At intersection of any two or more highways designed for different speeds it seems proper to base the design of a rotary on the highest design speed of any of the highways, regardless of whether or not it carries the greatest volume of traffic.

Vehicles can travel round at a speed of 25 km/hr. of a 30 m diameter island and at 40 km/hr. round a 46 m diameter island. Size of the central island should not be less than 30 m (UK recommendations) although islands have been built with 18.3 m or even 15.2 m diameter. An 18.3 m diameter island has been observed to take 2,500 vehicles (including cycles) per hour, a 23 m diameter island 3,000 vehicles per hour, and a 30 m diameter island 3,500 vehicles per hour. (Data collected from abroad).

The recommended *design speed for traffic rotaries in India* is 40 km/hr. in rural areas or where one or more of the converging roads is/are important, and a speed of 30 km/hr. for rotaries in urban areas and restricted locations.

Radii of Curves of Rotary Roadways at Entrance and Exit

IRC suggest a radius of 25 m to 35 m for design speed of 40 km/hr. and 15 m to 25 m for design speed of 30 km/hr. for radius at entry. The radius of the exit curve is recommended to be 1½ to 2 times the radius on entry. (Buses and trucks should be able to take a right turn at these curves at the design speed. See table for "Min: Turning Radii for Different Classes of Vehicles"). Where practicable three centred entry curves may be provided instead of simple circular curves.

In UK they recommed a maximum radius of kerb at entry of 35 ft. (10.7 m) and at exit 70 ft. (21.3 m) in built up areas, and 60 ft. (18.3 m) and 150 ft. (45.7 m) in areas outside towns.

The radius of *central island* is recommended to be kept slightly larger than that of the curve at entry. A value of 1.33 times is suggested by IRC. The radius should be as large as practicable.

Width of Carriageway at Entry and Exist

Carriageway width of the approach road	Radius at entry (m)	Width of carriageway at entry and exit (m)	
7 m (2 lanes)	25—35	6.5	Inclusive of widening needed on account of curvature
10.5 m (3 lanes)	,,	7.0	
7 m (2 lanes)	15—25	7.0	
10.5 m (3 lanes)	,,	7.5	

Based on IRC : 65

IRC recommend that the min: width of carriageway be at least 5 metres with necessary widening to account for the curvature of the road.

Width of the carriageway round the island should not be less than one-fourth the total width of all the radial roads and not less than one-half the width of the widest road plus width of one lane, or the widest single entry into rotary, whichever is more. A minimum width of 24 ft. (7.3 m) is recomended in USA and 30 ft. (9.1 m) in UK. Width should not be in excess of the traffic requirements. It is important that spacious areas which permit "open-field running" be eliminated by the provision of directional islands that leave little choice of route.

Weaving Length. Minimum weaving length should be 45 m for 40 km/hr. design speed and 30 m for 30 km/hr. design speed. The maximum weaving length should be restricted to twice the values given. Where the rotary diameter and the width of the carriageway according to the table given does not provide for adequate weaving length, the diameter of the rotary should be increased. (Weaving is merging and diverging operation.)

Width of Weaving Section of the rotary should be one traffic lane (3.5 m) wider than the mean entry width thereto.

Adequate and proper *super-elevation* can rarely be given on rotary roads (max: super-elevation is 12.5 per cent.), therefore, a large radius for the island should be aimed at. Inner portion of the rotary roadways is used by vehicles turning right and the cross-slope required by vehicles at entry and exit is opposite to it. The meeting of these cross slopes produces a "crown line" which will be at a distance of about 2 lanes from island in a 3- lane road. Entrance, exit and rotary roadways should be designed with as flat superelevation slopes as is practicable. A cross slope of about 1/20 is generally adequate with max: of 1/16. In all cases of changes in the cross slopes, the transition should be gradual.

Channelizing Islands, Directional Islands, or Refuges are small triangular shaped raised pavements or islands made at the entrance of the radial roads to the rotary carriageway to reduce the area of conflict between intersecting traffic streams by providing separate entries and exits of the rotary. Channelizing islands are also used as pedestrian refuges and for erecting signs and lighting the roadway. All channelizing islands should be provided with highly visible rounded kerbs, 10 to 15 mm high, which may permit a motor vehicle occasionally to mount the island in case of emergency. Where these islands are used as pedestrian refuges the kerbs should be straight and 18 to 23 cm high. By providing channelizing

islands, vehicles are confined to paths. When drivers or pedestrians have free choice of routes through large intersections, their actions cannot be predicted by others. This creates confusion and congestions and often leads to accidents.

Median Strips or traffic separators are long directional islands usually provided in four-lane pavements or wide avenues to demarcate the line of separation of the lanes for incoming and outgoing traffic. These take the form of a kerb 60 to 120 cm wide with concrete edging and grass turfing in the middle.

Pavement strips or lane markers should not be used on the pavement of the rotary roadway or on the entrances and exists.

Sufficient sighting distance should be provided where possible and cautionary sign boards (or the standard sign indicating the presence of the rotary) should be put up at proper places advising drivers to reduce speeds according to the design speed of the roundabout. Shrubs or bushes should be planted at the centre of the island so as to screen the glare of on-coming headlights, or some sort of masonry structure built at least 120 cm high.

It is preferable, though not essential, that a traffic rotary should be located on a level ground. It may be sited to lie on a plane which is inclined to be horizontal at not more than 1 in 50 and the maximum grade of any approaching road in the vicinity of the rotary should not exceed 1 in 30. Grades approaching or within a rotary should in no case exceed 1 in 20.

Straight road speed in km/hr.	30	40	50	65	80	95
Rotary turning speed in km/hr.	25	30	40	50	56	65
Min. radius of edge of pavement or kerb (for trucks and buses) in meters	16.8	16.8	30.5	39.6	54.9	76.2
Min. diameter of central island: in metres						
without super-elevation	30.5	42.7	64.0	85.3	146.3	195.1
with 1/16 super-elevation	30.5	36.6	54.9	76.6	115.8	164.6
Width of carriageway around island	6.7 to 9.1 meters					
Weaving length in metres						
Min:	27	30	45	54.9	64.0	73.2
Max:	—	60	90	121.9	152.4	182.9

Traffic Markings and Road Signs

White Lines—Traffic Lane Markings

The lines mark the division of the carriageway lanes and are made 100 to 125 mm wide (75 mm minimum, 100 mm common) longitudinal strips, and 150 mm wide transverse strips, set out in accordance with the following schedule :

(*a*) On straight lengths of highways : one metre lines with 4.6-metre gap ; 2.7-metre lines with 2.7-metre gap ; 1.5-metre lines with 3-metre gaps.

(*b*) In town areas with heavy traffic : one-metre lines with one-meter gap ; one-metre lines with 2.7-metre gap.

(*c*) At bends and near junctions : Central continuous line extending 30 metres in each direction beyond the tangent points. Continuous lines are used to prohibit crossing.

White and black strips have greater visibility than full white surface. A white line placed 46 cm from the edge of the pavement allows higher night speeds with greater safety. The lines should be rough textured. The common white paint is not quite suitable as it is not very durable, and is smooth. White asphalt and mastic can be used, or a mixture of the following :

Resin ; oil ; white sand : white filler (whiting) and white pigment. Pigment content must be 5 to 10 per cent for visibility. Paints should preferably be made with lacquer (varnish in alcohol) and glass beads of the size of fine sand to increase night visibility ; glass beads reflect light at night. For painting white lines machines are also available which apply the paint by brush or by a power driven sprayer. Hand painting is generally done with stencils cut of the size of the lines. Plastic material has now been introduced for marking white lines which is applied hot by machine. Powdered glass or crushed marble are generally used with the plastic binder .

Road Signs

Standard designs for road signs in India should be obtained from the IRC. Certain general principles are recognised internationally. Signs giving warning of danger are indicated by a red triangle of sides 46 x 46 x 46 cm made of cast iron plate, with 75 mm red border and the centre white or hollow. The triagle is fixed above a rectangular plate of size 46 x 38 cm to 30 x 25 cm. Symbols and words indicating the danger are given on the rectangular plate, preferably with a red border. The space between the red triangle and the symbol plate is about 15 cm. Prohibitory signs include a red disk of 61 cm diameter fixed above a rectangular plate on which the subject of the prohibition is indicated, or a red ring fixed above a rectangular plate. Informatory signs are rectangular plates with no red colour.

All road signs should be erected at a distance not greater than 3 metres and not less than 1.8 metres from the edge of the carriageway in outside town areas. In built-up areas and mountainous country, the distance between the edge of the sign nearest to the pavement and a

vertical line drawn from the pavement (face of kerb) shall be not less than 30 cm. The height of the lower edge of the signs is 2.1 metres from the level of the crown of the road. Some engineers, however, consider that the ideal height of the centre of a traffic sign is at the eye level of the person for whom it is intended—in open country it should be 107 cm and in towns 206 cm above road level. 80 x 80 x 8 mm T-iron may be used for the post fixed about 70 cm in the ground. Vertical posts should be painted in 23 cm black and white bands, commencing the black from bottom.

All warning and information signs should be fixed at 122 metres on level roads, 152 metres on steep downgrades, and 76 metres on steep up-grades from the point of danger or the site, and should be erected on the left hand side of the road facing the aproaching traffic. Warning signs should be placed, as far as possible, on a straight section of the road (23 metres visibility is essential) and should be clear of bushes, trees or other obstructions to visibility so that the full length of the supporting post is seen.

Reflecting Road Studs are very useful for the guidance of drivers at night. Spherical glass 'cats eye' reflectors are used which are embedded into notches made on the edges of the kerbs or fixed at 9-metre centres on straight roads midway between the dashes of the interrupted white lines, and at 3.7-metre centres on bends which have the continuous white line. On vertical planes the studs are fixed 107 cm above road level.

All reflectorized signs should face approximately one-tenth of the width of the sign toward the centre line of the roadway. All other signs should face slightly away from traffic. On grades reflectorized signs shall be tilted forward or backward and raised or lowered so that the face of the line is aproximately perpendicular to and in line with the headlines of approaching vehicles.

26. STREET AND HIGHWAY LIGHTING

Lighting from one side of the road only will be unsatisfactory except on bends and narrow roads. For reasons of economy sometimes single-side lighting is adopted even for two-lane roads. Maximum distance between two opposite rows of lights should not exceed 9 m if a dark centre to the road is to be avoided. Therefore, roads up to 9 m width can be effectively lighted by lights mounted vertically above the kerb-line at each end. For road width up to 12 m, brackets should be used to overhang over the road-way to reduce the distance between the rows of lights to 9 m. Roads more than 12 m width will require a line of lights vertically above each kerb-line, and additional lights suspended over the centre of the road at intervals not exceeding every third span.

The maximum overhang of brackets over the kerbs should not be more than 1.8 m in order to avoid unduly dark patches on the kerbs and footways.

IRC has specified the horizontal clearance required for lighting poles as follows :—

(*a*) For roads with raised kerbs (as in urban roads)—min: 0.3 m, desirable 0.6 m from the edge of the raised kerb.

(*b*) For roads without raised kerbs (as in rural roads)—min: 1.5 m from the edge of the carriageways, subject to a min: of 4.0 m from the centre line of the carriageway.

Side-mounted lamp posts may be arranged either opposite to each other or staggered. A staggered arrangement is better.

Usually suitable mounting height of lamp posts is 6.1 m to 7.6 m in streets (with 4.6 m min:), and 7.6 m to 9 m in highways, with 36.6 m to 45.7 m spacing. Ratio of spacing to mounting height should not exceed 8 : 1 in built-up areas and 10 : 1 in outside built-up areas. With high mountings, uniform illumination can be maintained even though individual units are widely spaced. High mounting also greatly reduces the blinding effect of direct glare. Large lamps with high mountings and wide spacings should be preferred.

Lights are installed at closer spacing on curves than on straight lengths and on outside of the curves. At vertical summit curves lights should be installed at closer intervals near the summit. Single-side mounting should be employed at long curves having a radius of curvature of less than 610 m.

The minimum vertical clearance required for electric power lines up to 650 volts has been specified as 6 m above the pavement surface by IRC.

Poles carrying overhead power and telecommunication lines should, excepting the urban areas, be erected at least 10 m awav from the nearest edge of the roadway, provided also that these are at a minimum distance of 5 m from the nearest line of avenue trees.

Where there is interference from trees bordering the road, lights should be centrally suspended ; kerbs and footways will not be well lighted with this system except in comparatively narrow streets fronted by light-coloured buildings which will reflect the light.

On *bends*, a lamp should always be dead ahead of a driver, or nearly so. On an S bend it will be necessary to change the lamps from one side of the road to the other at some point in the middle of the bend.

At road junctions and roundabouts it is particularly important to ensure that sources of light are so placed that a driver cannot only see from some distance away that he is approaching a junction but can also appreciate the route which he should follow. At small roundabouts with central islands of diameter of say 18 m, a single light of the cut-off type having symmetrical distribution, and mounted centrally at a height of 9 to 10.6 m above the carriageway will be found suitable. For bigger islands lights should be provided above the kerb of the central island in line with each approach traffic lane. Lights are required over the weaving portions of a rotary on the outer kerbs of the roundabout carriageway where the central island exceeds 30 m in diameter. Lights should also be provided at the kerbs near footpaths crossing rotaries in order to provide adequate visibility at the crossings for drivers leaving the roundabout.

A bright patch must be provided at the mouth of the junction of a crossing, and this entails siting a lamp on the far side of the junction from the observer (opposite the on-comming traffic lane). An intersection of two roads will require four lamps, one not more than 12 m along each road on the left-hand side, and the other not more than 36 m from the crossing.

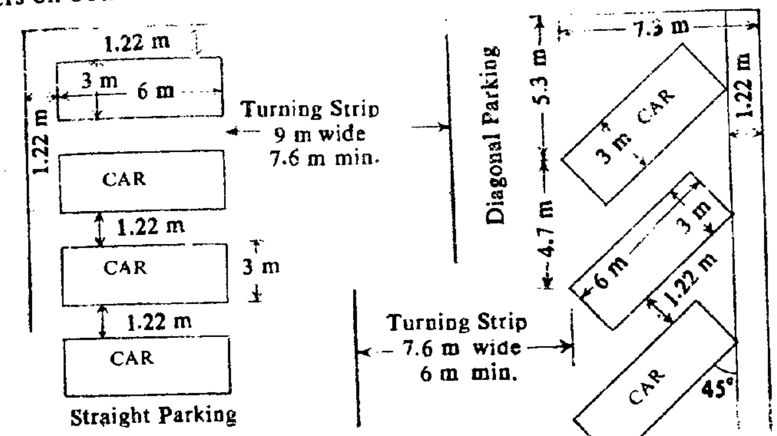
27. PARKING PLACES AND LAY-BYES

For parking at open places a gross area of about 23 sq. metres for each car may be taken. (This makes due allowance for access roads, irregularities in manoeuvring and opening of doors). Parking lane width (for parking parallel to the road kerbs) should be 3 m and which may be reduced to 2.5 m where available space is limited. Where sufficient carriageway width is available, angle parking may be adopted. For parking either diagonally or at right angle to the kerb a width of at least 6 m is needed. IRC recommend the following min: parking space :

Cars—3 m × 6 m when individual parking space is required.
2.5 m × 5 m when parking lots for community parking are required.

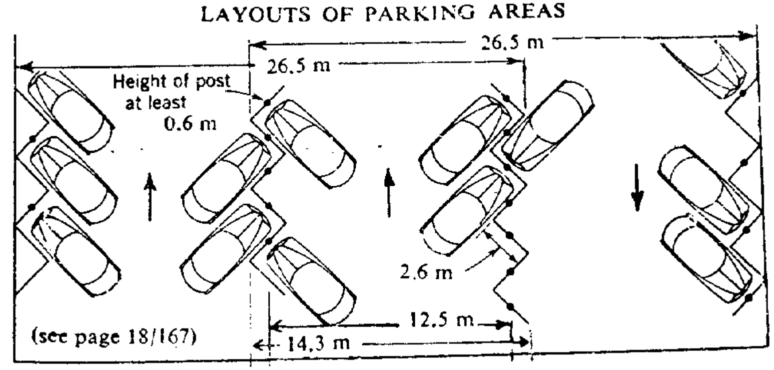
Trucks—3.75 m × 7.5 m.

Lay-byes. On important roads lay-byes should be provided at intervals to enable vehicles to draw off the road for temporary parking for break-downs and repairs. Bays should also be provided for bus stops. These should normally be 3 m wide and at least 30 m long with 15 m end tapers on both sides.



PARKING LAYOUTS

LAYOUTS OF PARKING AREAS



28. ROAD EMBANKMENTS

(Embankments have been described in detail in the Section on "Irrigation").

Borrowpits for Highway Embankments

As far as possible no borrowpits should be dug on the roadland. Borrowpits should be retangular in shape with their long sides parallel to the road alignment. If they must be dug on roadland, they should be dug as near the road boundary as possible.

No borrowpits should be dug within 5 metres of the toe of the final section of the road embankment, after making due allowance of future development.

Borrowpits should not be continuous unless intended to be used as side or catch-drains and should be of regular shape preferably of equal size in multiples of 3 metres to facilitate measurements, and not more than 60 cm deep. Ridges of not less than 8 metres width should be left at intervals not exceeding 300 meters. Small drains may be cut through the ridges, if necessary, to facilitate drainage.

Where the embackment acts as a *flood bank*, all earth for the embankment should be borrowed, as far as possible, from the river side. The inner edge of any borrowpit should not be less than 15 metres from the toe of the bank. Where the borrowpits must be dug on the landside, a berm at least 24 metres wide should be left between the borrowpit and the toe of the bank.

The finished road level should be at least 45 cm above the highest flood level. The rear side of the bank should have a cover of 0.75 metre to 1.25 metre over the "saturation line" drawn at a slope of 1 in 6 from the high flood level on the river side. (Also explained in "Irrigation"

Side Slopes of Road Embankments

Side slopes should be as gentle as possible. Slopes steeper than 2 : 1 (hor. : ver.) should never be used. Provide 4 : 1 if the embankment is up to 60 cm height to enable vehicles getting down in case of emergency, and natural slope of the soil if over 60 cm height. Some engineers recommend 4 : 1 if under 1.8 metres high and 2 : 1 if over 1.8 metres high. Any slope steeper than 3 : 1 is not desirable. Provide guard-rails where embankment is 1.8 metres high or higher. Top and bottom of the slopes of banks and cuttings should be rounded off. Desirable shoulder slope has been given under "Camber or Crossfall".

The soil used for embankments shall be free from stumps, trees, roots, rubbish, or any other material likely to deteriorate or affect the stability of the embankment. Highly expansive clays exhibiting marked swell and shrinkage properties, shall be deposited at the bottom of the embankment and no such material shall be placed nor permitted to remain in the top 50 cm portion of the embankment below the subgrade. Where any unsuitable materials occur in the embankment foundations, these shall be removed and replaced by approved material.

In all cases the original ground shall be consolidated as much as reasonably possible by rolling or tamping. Any empty pockets or depressions left in the soil as a result of clearing or grubbing operations shall be

Normally the height of the road embankment shall not be less than 60 cm. Where, however, this is not feasible, and the original ground is soft, it shall be fully compacted by loosening, watering and rolling in layers of 25 cm loose thickness up to a depth of at least 50 cm below the top of the subgrade and for a width equal to the proposed width of the pavement plus one metre on each side thereof. If the next 15 cm depth of the original ground below this excavation is also not hard enough, it shall also be compacted.

Where an embankment is to be placed on steep sloping ground, the surface of the ground shall be benched in steps or trenched, or broken up in such a manner that the new material will properly bond with the existing surface.

If the soil for embankment has less than the required moisture content for proper consolidation, necessary amount of water should be added to it, either in the borrowpits before excavation is made, or after the soil has been spread loosely on the embankment. If the soil as delivered is too wet, it should be dried by aeration and exposure to the sun till the moisture content is correct for compaction.

Allow a monsoon to pass over a road bank and compact it again before metalling.

Widening of Existing Embankments

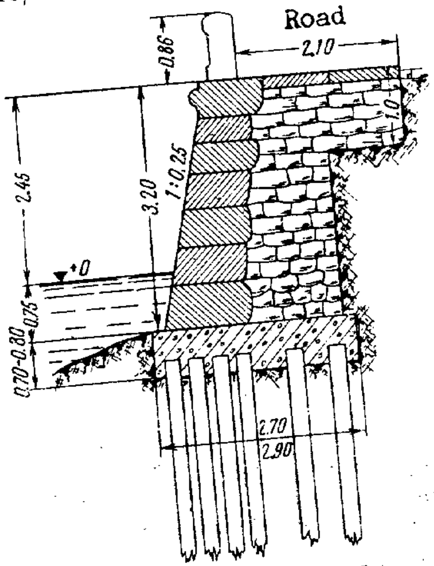
When an existing embankment is to be widened, and its slopes are steeper than 4 : 1, horizontal benches of up to 60 cm width should be cut into the old slope for ensuring adequate bond with the fresh embankment material to be added. When the existing slope against which the fresh material to be placed is flatter than 4 : 1, the slope surface may only be ploughed or scarified instead of restoring to benching. End dumping of earth from trucks for widening operations should be avoided as far as possible.

Treatment of Embankment Slopes for Erosion Control

Simple Vegetative Turfing. The method cosists of preparing slope area into seed beds by grading it and then broadcasting seeds or planting root slips of locally available plants. In poor soils, instead of treating the whole of the slope, plantation could be encouraged by puting in seedlings in isolated pockets of specially enriched soil.

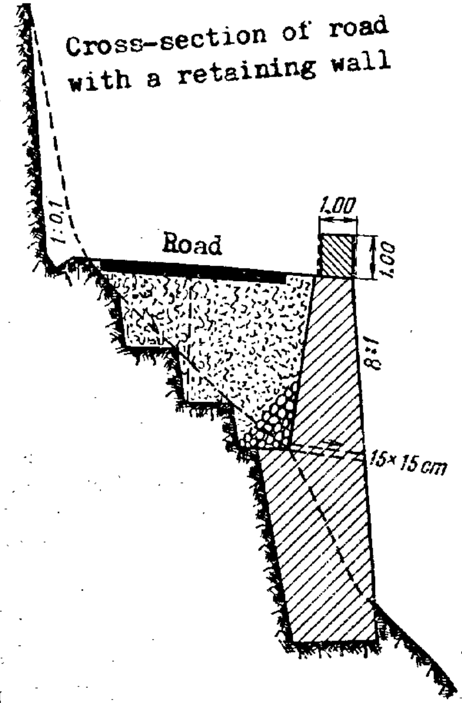
It is also possible to provide vegetative turfing by 'sodding' which involves the bodily transportation of blocks of turfs of grass (with some 5 to 8 cm or so of soil covering the grass roots) from the original site to the side slopes of the embankment to be treated. In localities where most of embankment material is not conducive to plant growth, the top soil suitable for plant growth existing over the embankment foundation area or borrowpit can be stripped and stored for covering embankment slopes. Slopes should be roughened and wetted slightly in order to affect proper bond.

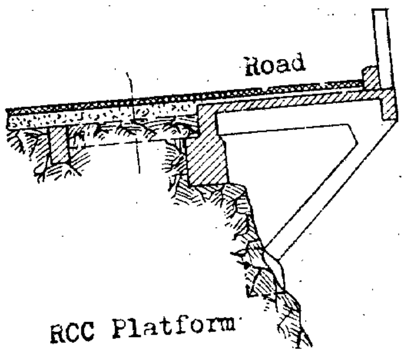
When an embankment is made up of sandy soil, one of the possible measures against erosion is to have a blanket cover of clayey soil. 25 to 30 cm of clay is tamped well on the slopes and subsequently provided with simple vegetative treatment described above.



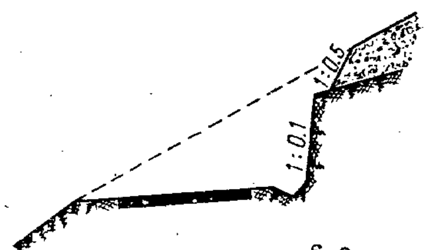
Solid retaining wall on pile foundations

Cross-section of road with a retaining wall



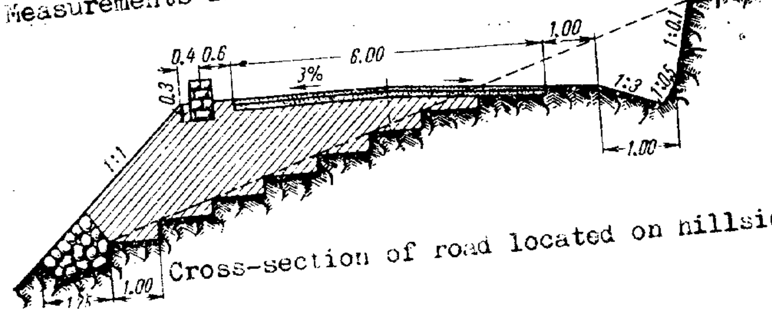


RCC Platform



Cross-section of a bench type cutting

Measurements in metres



Cross-section of road located on hillside

29. ROADSIDE DRAINAGE

This subject has been described in detail in the Section on "Drainage and Sewerage" and also under "Sub-soil Drainage and Moisture Control", in this section.

For traffic safety the roadside drainage channel should be wide and shallow. For an open ditch channel, a drain which is as wide as it is deep will be perhapes, the cheapest. A bottom width less than 1.22 m is difficult to construct and clean out. The depth should vary from a minimum of about 30 cm below the edge of shoulder in regions of low rainfall intensities to a minimum of 45 cm in districts of high rainfall. The bottom should be rounded since a rounded section is less conductive to erosion than the trapezoidal or V-section. In rock cuts, the back slopes will necessarily be very steep where the drainage channel may have a narrow V-shaped cross-section. The sustained flow level in a ditch channel in cutting should not be higher than bottom of the base course of the road.

Road-side Gullies or Inlets

The distance apart for placing road-side gullies depends upon the available longitudinal fall and cross fall, and generally varies between 45 to 90 m, and each gully is required to drain an area of about 250 sq. m. This spacing will be all right for a road about 4.6 m wide. For wider roads, the distance apart of gullies should be proportionately reduced. The inlet openings are either horizontal or vertical. The bars of the horizontal gratings must be closely spaced to be strong enough to withstand traffic. Gratings to be effective should have openings parallel to the flow. Road inlets are generally combined with catch-pits which are about 60 cm deep below the invert of the outlet. (See also under "Kerbs".)

In order to facilitate surface drainage, channels should be graded to a minimum fall (longitudinal slope) of 1 in 250. (Prefer 1 in 200).

A concrete channel is formed in front of the road kerb about 2.4 m wide and 40 mm deep towards the kerb to give a cross slope. If the road slab is of concrete a longitudial joint should be made between the pavement slab and the channel slab.

Underground Ducts. All ducts should be laid with a slope of 1 in 400 to drain out any water.

Telegraph, Telephone or Electric Poles etc. should be fixed normally at least 1.5 m outside the existing road edge and if possible at the road boundaries.

30. PREPARATION OF HIGHWAY PROJECTS

Surveying and Planning for a Road Project

Survey and alignment of a new road do not generally present many problems except in a hilly and mountainous country which call for special skill. The aim should be to establish the easiest, shortest and the most economical line of communication between the obligatory points which will have the minimum cost of construction and maintenance. In trying to reduce the cost of construction it is false economy to lower the standards of alignment and grade of the road. Sharp curves and steep gradients should be avoided and balancing cuttings and fillings obtained as far as possible. As far as possible, the road alignment should be on a high ground or ridge to ensure good drainage. Sometimes the length of the route will necessitate increase (which should be done only within economical limits of the motive power and the extra construction and maintenance cost) to obtain easy curves and gradients. Of recent years greater attention has been paid to fitting the road into the landscape, and doing all possible to preserve the beauty of the countryside. It is usually agreed that this is best done by the use of long sweeping curves, both vertical and horizontal, and by avoidance of hard lines. In laying down the minimum vertical and horizontal curves the basic idea is that a reasonably long section of any road should permit of the same speed being maintained throughout. A series of large radius vertical curves are much more economical.

Field Reconnaissance and Survey

Reconnaissance and preliminary investigation is of utmost importance and should be undertaken by an experienced and responsible officer. General examination can be done by walking or riding along the probable routes when full notes should be taken of all the conditions existing around the general direction of the route, a wide belt on either side should be examined. This general examination enables the engineer to select one or more promising routes which have to be investigated in detail. The field reconnaissance and preliminary survey can be done by plane table and prismatic compass, aneroid barometer and the clinometer (for hill works). Distances can be estimated by pacing or measuring wheel or rotometer. 16 to 24 kilometres can be covered in a day in the plains and 3 to 6 kilometres in the hills, depending on the nature of the country. Proposed alignments are marked on the ground plans prepared from the prismatic compass survey.

Reconnaissance in mountainous country is better run from the summit downward. Particular notice should be taken of suitable places for turns. Long stretches in one direction before reversal are preferable. In snow areas, location should, if possible, be confined to slopes exposed to the sun in order to avoid icing of the roadway. Aerial photographs are very useful for locating positions in such situations.

Cross-section of the country are taken at right angles to the centre-line of the alignment with greater or less frequency according as the original surface of the ground is more or less transversely sloping. For a preliminary survey in the hills cross-sections are taken at 150 to 300 metres apart with a clinometer, Abney level or ghat tracer. The trial line can be run by clinometer and ranging rods and marked by stakes or by making white marks on trees. The distance can be chained and typical cross slopes noted by clinometer readings. The trial line should be traced in the field at a slightly flatter and easier grade than specified as ruling to allow for shortening when curves are put in.

For final survey cross-sections are taken extending to between 50 to 100 metres according to circumstances, but always beyond the water-line of the flooded land on either side of the line. When the side slope of the

country is steeper than 1 in 15, these cross-section should be taken at about 30-metre intervals or closer, and sufficiently far on each side of the centre line to allow of adjustment of alignment in the office.

Alignment

Alignment is the course or route along which a road is located. In locating the line during the initial layout or when carrying out the detailed survey, no attempt should be made to run in curves but the alignment should be a series of tangents. The road boundaries are fixed by a theodolite traverse. Centre line is pegged every 30 metres on straights and every 15 metres or less on curves. All tangent points are pegged and reference pegs placed opposite all tangent points and not more than 100 metres on straights.

The alignment of a hill road should be on the side of the rock which is sound and solid. The dip of strata in the side of hill should not be towards the road otherwise landslide will occur. There should also be no inclined fissures.

The following surveying instruments are generally required :—

Plane table, prismatic compass, dumpy level, clinometer, ghat tracer or Abney's level, Aneroid barometer, theodolite, 30-metre tape, foot rule, spirit level, poles, chain, string, pegs, hammer, white lime powder for marking lines, white paint for marking on trees, hatchet for cutting bushes, spades, jumpers, note books.

The progress for laying out the alignments depends upon the nature of the country. On an average a party can cover about 6 kilometres per day in the plains (adding extra time for laying out curves) and only about 16 kilometres in a month in the hills.

Project Estimates for Highway Schemes

A road project estimate should contain the following information :—

Report

The report should be brief but full of all the important information and which should be written as the survey (or reconnaissance) progresses and should outline :

Previous history of the road if any existing, or history of the proposal of the road : its present condition ; topographical and geological features ; possible future developments of the areas surrounding the road ; rainfall and floods, etc.

Give details of the proposed alignment, its special features and justification for the selection of the particular alignment. Necessity for any heavy banks or cuttings proposed and why these could not be avoided. Details of gradients and curves with radii, and where they are in any way unusual. Sight distances and the corresponding provisions made at horizontal and vertical curves.

Formation width with relation to the existing and future traffic. Land widths to be acquired ; permanent and temporary acquisitions.

Masonry works to be built—Bridges, culverts, causeways, drainage, retaining walls, rest houses, boundary stones, etc. Prepare separate sub-estimates for all important works.

Materials and Labour

Materials available locally with reference to the various sections of the road and their suitability as road metal. How labour will be provided for the work. Agency for execution of the project. Any machinery recommended.

The Estimate of Quantities should contain the following items :—(separate estimates)

(*a*) Main road work—earthwork, base course, metalling ; small masonry works, pipe culverts, retaining walls, side drains, parapet walls, fencings, boundary stones, sign posts, siding and parking places.

(*b*) Bridges, culverts, causeways, rest houses.

(*c*) Arboriculture, nurseries, wells.

(*d*) Acquisition of land—estimates of cost for different categories of lands.

(*e*) Temporary structures, and also for water-supply.

(*f*) Estimate for preliminary survey.

For hill roads the increase in length over air distances may be from 45 to 80 per cent. As a rough guide a 65 per cent increase may be assumed in the absence of exact details, for the preparation of estimates of cost of survey and rough cost of constructions.

Drawings

Key map : Can be drawn to one of the following scales depending on the extent of area to be covered. It should show the location of the road with respect to important towns and industrial centres or grain markets within 5 kilometres and the existing means of communications (rail, road, waterways) in the neighbourhood.

Scales : 150 mm to 10 km
75 mm to 10 km
40 mm to 10 km
10 mm to 10 km

Index map : It should show the general topography of the road and important towns and industrial centres served by the road and also the existing and proposed means of communications. One or more alignments of the proposed route should be marked. For hill roads contours are marked at 15 or 30 metre intervals. Geological survey maps may be obtained.

Preliminary survey and location or alignment plans : Are drawn to a scale of either 125 mm or 250 mm to 1 km, showing the proposed alignment, diversions, curves, width of right-of-way, building and control lines, village boundaries, etc.

Detailed Drawings—Recommended Scales :

Plain and open country :

Horizontal for plan and L-section 70 m=25 mm

Vertical for L or cross-sections 3 m=25 mm

Close or hilly country :

Horizontal for plan and L-section 30 m=25 mm

Vertical for L or cross-section 6 m=25 mm

(The vertical scale usually adopted is 1/10th of the horizontal scale.)

(*i*) The detail plan should show : Centre line of the proposed road, boundaries of the right-of-way, existing structures, contours of levels, description of soils, drainage courses, ponds, tanks, curve data, drains crossings, reduced distances of cross-sections, bench marks, etc.

(*ii*) The longitudinal section should show : The datum line, ground levels, formation levels, height of bank, depth of cuttings, soil classifications, gradients, vertical curve data, drainage crossings, and any bench marks of which levels may have been taken.

(*iii*) The cross-sections should show : The existing and formation levels, areas of cuttings and fillings, side drains, catch drains, land widths, building lines and control lines, etc.

Roughly initial costs of different types of roads taking for water-bound macadam per unit of surface area :

	Water-bound macadam	40 mm asphalt macadam	120 mm cement concrete
Cost	1	3	6
Life	3 to 4 years	10 to 12 years	40 years

Repairs and Maintenance

The following figures may be taken for approximate estimating of different road surfaces under medium conditions of traffic :

per unit of surface area

(*a*) *Water-bound macadam*

Taking initial cost	100
Routine annual maintenance ...	5
Renewal coat every 3 or 4 years (or earlier)	50

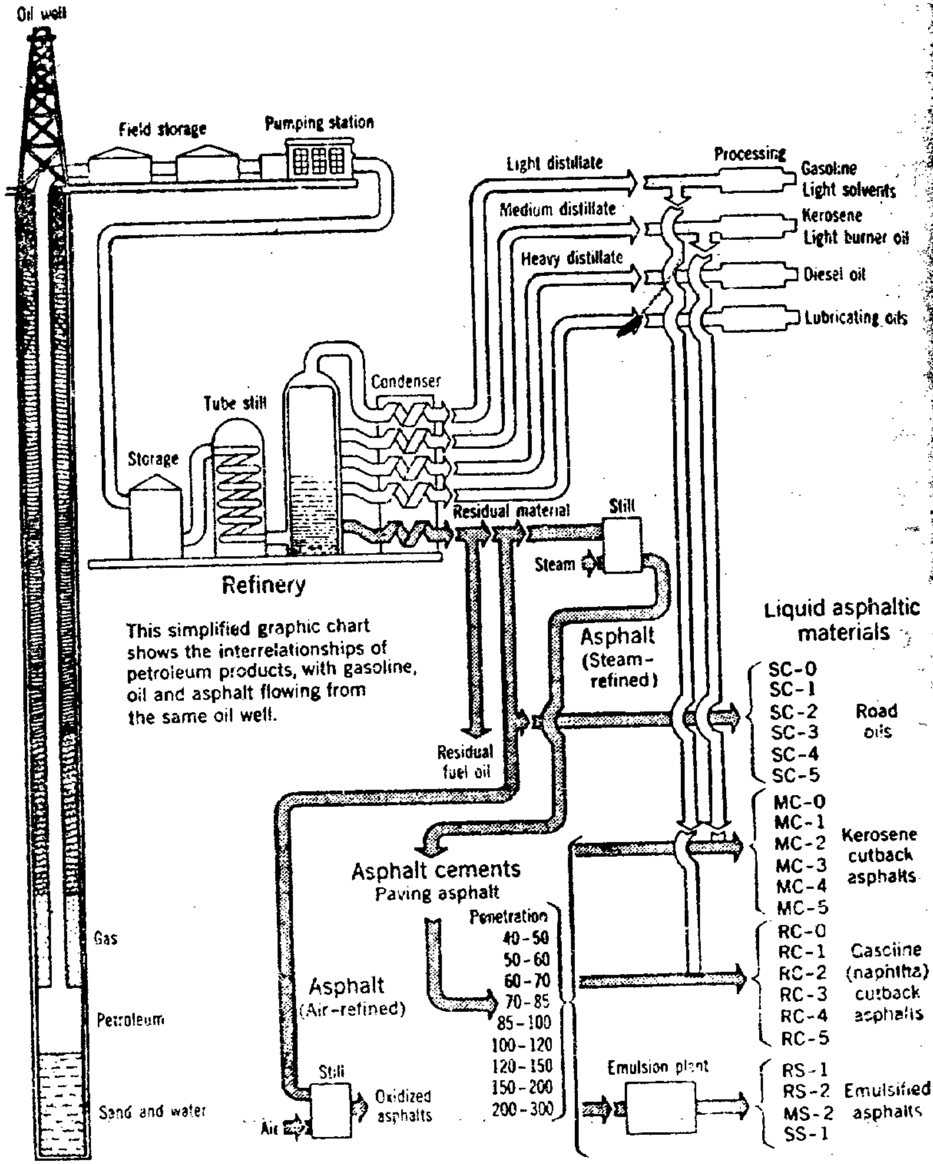
(*b*) 40 mm *Asphalt macadam* :

Taking initial cost	100.0
Routine annual maintenance ...	2.3
Seal coat every 4th year	24.0
Renewal coat every 10th year ...	85.0

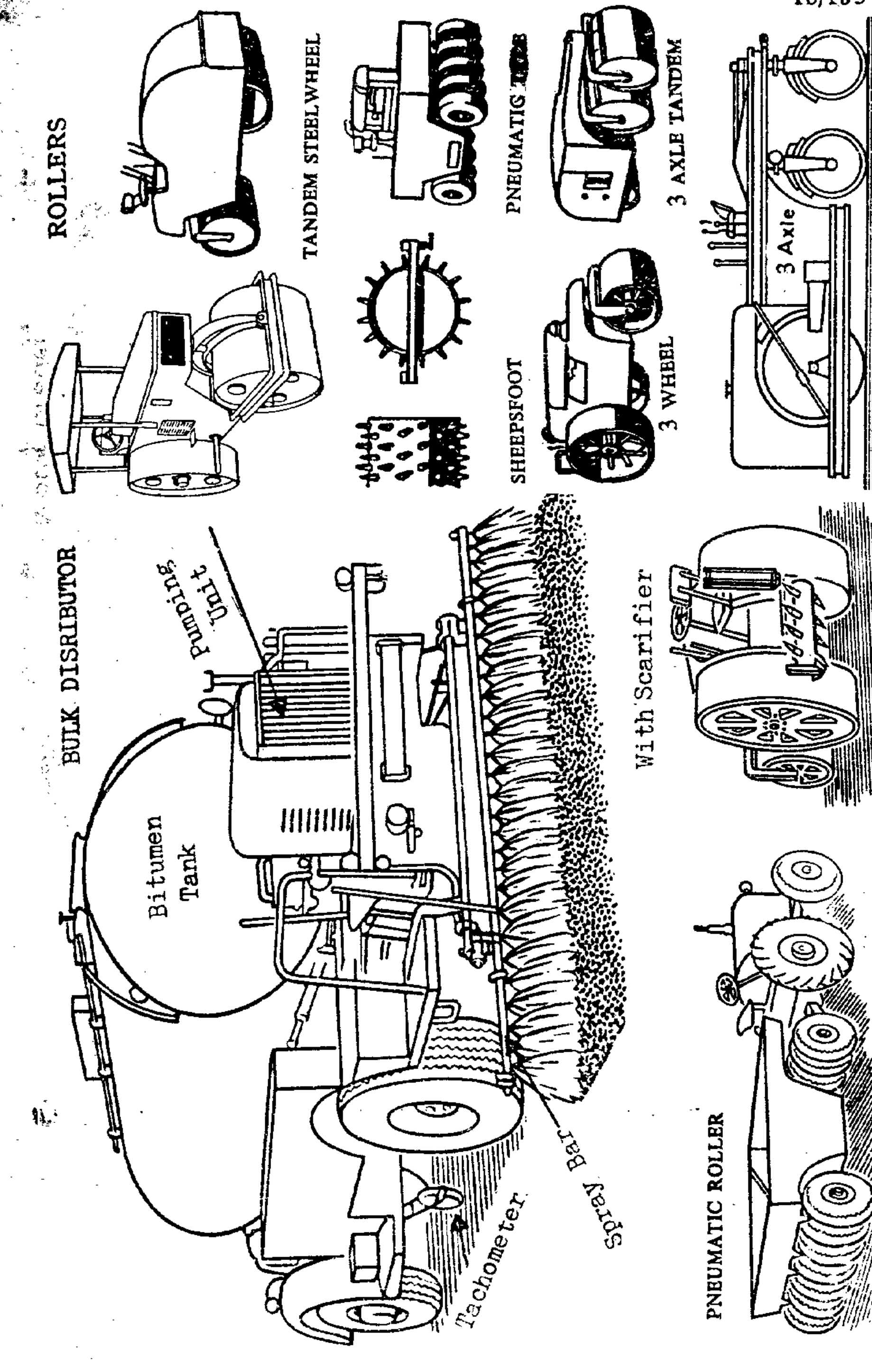
(*c*) 120 mm *Cement concrete :*

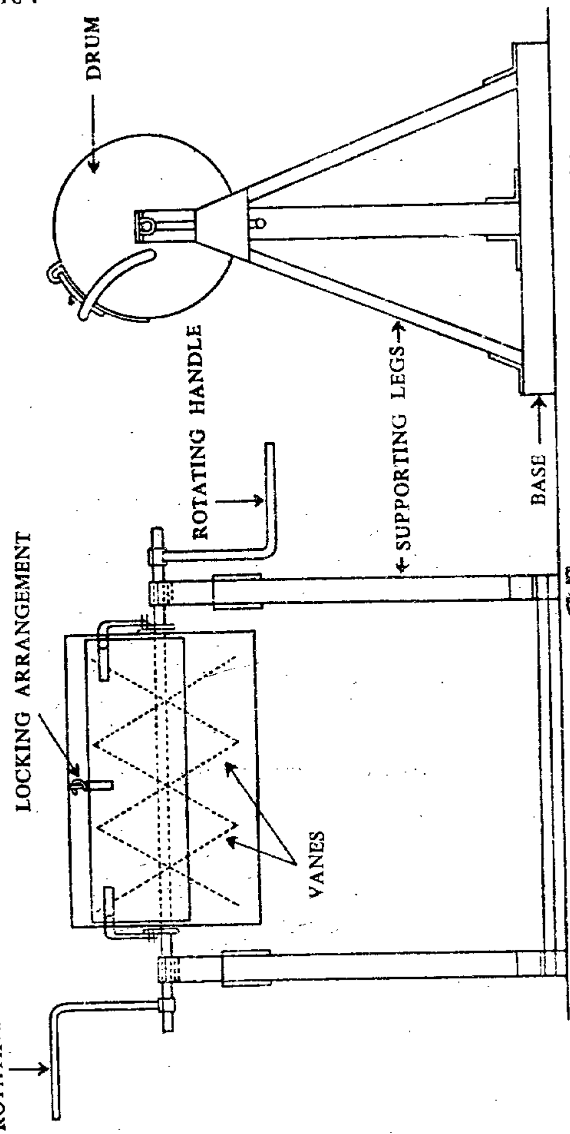
Taking initial cost	100.0
Annual maintenance	00.5

It is considered that about 60 to 70 per cent of the repair grant should be available for re-surfacing.

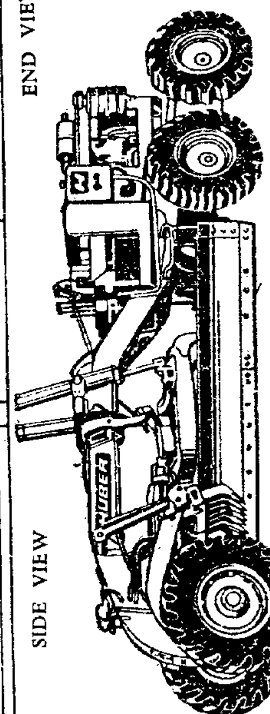


Simplified flow chart showing recovery and refining of petroleum asphaltic materials (Courtesy The Asphalt Institute)

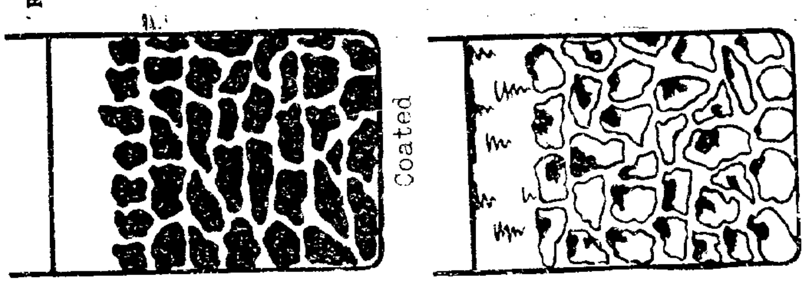




HAND DRUM MIXER



Motor Grader



STRIPPING TEST

SECTION 19

BRIDGES & CULVERTS

1. SURFACE DRAINAGE OR FLOOD DRAINAGE

Run-off is that part of the total rainfall which reaches a stream, drain or sewer or any given point from a certain catchment area within a given time. It is the total rain-fall minus evaporation and percolation losses.

Intensity of Rainfall. The maximum rainfall during a short period (usually taken at one hour) is called the "intensity of rainfall". The intensity of rainfall generally varies both over the catchment and within the period of storm. Storms of short duration are more intense than storms of longer duration, and larger the area the smaller the over-all intensity.

Catchment area or catchment basin means the area from which rainfall flows into a drainage line, outfall or reservoir, etc. The boundary line of this basin is called the ***water-shed***.

Drains and culverts are designed to take the run-off from the drainage area resulting from a storm which is considered to produce the greatest momentary run-off that can occur once a year. Exceptionally heavy storms of rare occurrence are permitted to cause flooding for the reasoning that the damage done by such flooding does not justify the provision of large drains or culverts at heavy extra costs capable of taking the heaviest flow likely to occur at any time.

Since the intensity of rainfall is not uniform throughout the storm period, the following formula is used to arrive at the maximum intensity that is likely to occur during an interval of any one hour within the duration of the storm :—

$$i=\frac{F}{2}\left(1+\frac{1}{t}\right)$$

Where : i= the maximum intensity of rainfall (one hour rainfall); F= total rainfall of the storm; t= duration of the storm in hours.

For calculating flood discharge for the design of bridges, the severest storm ever experienced in the region is considered.

The correct procedure for finding i is to take a number of really heavy and prolonged storms and work out i, from the F and t of each of them.

The critical (or design) intensity for a catchment is that maximum intensity which can occur in a time interval equal to the concentration time T of the catchment and can be found from the equation:

$$I=i\left(\frac{2}{T+1}\right) \qquad \text{I is critical intensity}$$

For the design of minor bridges and storm water drains, it will be sufficient to take "one hour rainfall." For greater accuracy this has to be modified further with the time of concentration to arrive at the critical (or design) intensity.

HEAVIEST RAINFALL IN ONE HOUR (in mm) FOR IMPORTANT PLACES IN INDIA

Agartala	66	Dibrugarh		Mawsynram	127
Ahmedabad	80	(Mohanbari)	93	Minicoy	70
Aligarh	51	Dum Dum	68	Mukhim	57
Allahabad	65	Dumri	66	Nagpur	78
Amini Devi	53	Durgapur	90	Nandurbar	73
Amritsar	74	Gangtok	81	New Delhi	79
Anantapur	38	Gannavaram	61	North Lakhimpur	65
Asansol	86	Gauhati	61	Okha	76
Aurangabad	61	Gaya	70	Okhaldunga	52
Bagdogra	70	Gorkha	62	Palgauj	56
Bagra Tawa	63	Gwalior	63	Panaji	54
Bangalore		Hazaribagh	78	Panambur	37
(Aerodrome)	58	Hirakud	82	Panchat Hill	71
(Central)	61	Hyderabad	102	Pathankot	68
Barakachar	51	Imphal	48	Patna	59
Barahkshetra	89	Indore	60	Pokhara	62
Barhi	56	Jabalpur	77	Puno	47
Barmul	75	Jagdalpur	73	Port Blair	61
Baroda	71	Jaipur	55	Punasa	75
Barrackpore	58	Jamshedpur	62	Pupanki	77
Bhimkund	60	Jamui	60	Putki	49
Bhopal	72	Jawai Dam	98	Raipur	49
Bhubaneshwar	46	Jharsuguda	77	Ramgarh	56
Bhuj	49	Jadhpur	60	Sagar Island	94
Bishungarh	60	Tonk Dam	46	Shillong	43
Bokaro	58	Kathmandu	44	Sindri	50
Bombay		Khalari	63	Sonepur	78
(Colaba)	129	Khijrawan	66	Srinagar	22
(Santa Cruz)	91	Kodaikanal	83	Shanti Niketan	49
Calcutta		Konar	59	Taplejung	59
(Alipore)	62	Luchipur	63	Tehri	41
Chambal	84	Lucknow	70	Tezpur	63
Chandwa	46	Madras		Thikri	61
Cherrapunji	127	(Meenambakkam)	62	Tiliava Dam	80
Coimbatore	80	(Nungambakkam)	75	Tiruchirappalli	78
Dadeldhura	36	Mahabaleshwar	51	Trivandrum	96
Dhanbad	74	Maithon	54	Vengurla	66
Dhanwar	20	Mangalore	72	Veraval	64
Dholpur	47	Marmagao	60	Visakhapatnam	63

Meteorological Department

The maximum rainfall of a place is usually taken to be 1.51 times the average annual rainfall (of 35 years). (See under "Storage of Rainwater for Irrigation" in Section 17.)

Time of Concentration. This is the time required for the storm water to run from the most remote point of the area under consideration to reach the site of the culvert at which the maximum run-off is being estimated. The time of concentration depends upon the slope of the ground, distance to be travelled and the nature of the soil. An engineer is concerned with the maximum rate of run-off and this occurs when the duration of the storm is equal to the time of concentration, because if the periods are short, the whole of the watershed will not be contributing the water, and for longer durations the intensity of rainfall will be smaller.

Run-off from Catchments. Run-off or storm water flows from catchments depend upon a number of factors such as intensity and duration of rainfall, area and shape of the land and its contours (slopes), initial stage of wetness; losses from evaporation (depending upon the climate) percolation (depending upon the nature of the soil and its absorbing qualities), transpiration by vegetation, etc. Other factors being equal, a catchment with a higher average rainfall will show a higher average loss. Floods from a larger area will take longer to rise and will be of less intensity relative to that area than floods from a smaller catchment.

There are a number of complicated empirical formulae for calculating run-offs of catchments which are not strictly accurate and give varying results; their use should be avoided. It will be appreciated that there are such a large number of factors in the estimation of run-off which are impossible to assess with any great degree of accuracy. Elaborate methods should be deprecated because they give a false impression of accuracy, which is, in fact, unobtainable. The calculations should be as simple as possible.

A rational method has been evolved in the form of the following general equation for calculating the flood discharge of an area:

$$Q = \frac{R.A.P.}{36}$$

Where Q is total run-off in cu. metres/sec.; R is intensity of max. rainfall in cm/hour (based on concenration time); A is drainage area in hectares contributing to run-off; P is factor of imperviousness which may be taken as follows :

Impermeability Factor of Surfaces

(Percentage coefficient of run-off for the catchment characteristics) :

Steep, bare rock	0.90
Rock, steep but wooded	0.80
Plateaus lightly covered, ordinary ground, bare	0.70
Densely built up areas of cities with metalled roads and paths ...	...0.70—0.90
Residential areas not densely built up, with metalled roads ...	...0.50—0.70
ditto., with unmetalled roads ...	...0.20—0.50
Clayey soils, stiff and bare	0.60
ditto., lightly covered	0.50
Loam, lightly cultivated or covered	0.40
ditto., largely cultivated	0.30
Suburbs with gardens, lawns and macadamised roads ...	0.30
Sandy soil, light growth	0.20

ditto., covered, heavy bush	0.10
Jungle areas	0.10—0.20
Parks, lawns, meadows, gardens, cultivated areas ...	0.5—0.25

The maximum values should be used for small districts having steep slopes, and the minimum values for large and comparatively flat districts.

Run-off from fan-shaped (a shape more or less like a sector) catchments are greater than from fern-shaped (elongated) catchments. Rugged surfaces in the catchment area reduce the run-off whereas smooth surfaces increase the run-off. The following percentages give approximate flood run-off available from the total precipitation :—

65 to 55 per cent in coastal zones; 55 to 30 per cent in intermediate or transit zones; and 30 to 15 per cent in dry zones.

Rainfall lost in :—

Evaporation and absorption by vegetation ...	30 to 50 %
Percolation ...	15 to 25 %
Available as surface run-off ...	25 to 55 %

The catchment area should be divided into small sections with reference to the rainfall and topography, the discharge or run-off from each section worked out separately and added.

2. WATERWAYS FOR BRIDGES

It is not economical or advisable to reduce the regime width of a stream (or restrict the waterway) when constructing a bridge or a culvert. A restricted waterway increases the velocity and hence scour, thus necessitating deeper foundations and also training works. The max. flood discharge is computed for which the waterway is to be designed. The linear waterway of the bridge must be ample to handle the whole discharge without detrimental afflux. Keep overall waterway equal to bed width plus full supply depth of the channel.

For working out the discharge three cross-sections should be taken at the stream site where it is proposed to build a bridge, one at the selected site, one upstream and another downstream. The cross-sections may be taken at the following distances :—

Catchment area	Distance upstream and downstream of the site
2.5 sq. km or less	150 metres
2.5 to 10 sq km	300 metres
over 10 sq. km	400 to 1600 metres

For non-meandering natural streams not wider than 30 m in alluvial beds but with well-defined banks, and for all natural channels in beds with rigid inerodible boundaries the linear waterway shall be the distance between the banks at that water surface elevation at which the designed maximum discharge determined can be passed without creating harmful afflux. (IRC Code)

For large natural streams in alluvial beds and hàving undefined banks, the linear waterway should be determined from the designed discharge, using any rational formula.

Determination of Effective Width of Linear Waterway : Lacey's formula.

For natural streams in alluvial beds and having undefined banks, the linear waterway may be determined from the following formula for regime conditions :

$$L = C\sqrt{Q}$$

where : L = the effective linear waterway in metres, in straight reach of the stream,
Q = the design max : discharge in cu.m/sec.,
C = a constant usually taken as 4.8 for regime channels, but it may vary from 4.5 to 6.3 according to local conditions

The formula gives the min : width of the waterway for undisturbed cross-section prior to the construction of piers. For calculating the effective linear waterway, the width of obstruction due to each pier shall be taken as the mean submerged width of the pier and its foundation up to the max: scour level. The obstructions at the ends due to the abutments or pitched slopes shall be ignored.

It may occasionally be found uneconomical to design a bridge for the absolute maximum discharge which may occur once after many years.

Afflux or Backwater

Afflux is the rise (or heading up) of water level (above the normal) on the upstream side of a bridge, caused by an obstruction across the channel (abutments and piers) and is the difference in levels of the water surfaces upstream and downstream of the bridge. Afflux is also caused when the effective lineal waterway of a bridge is less than the natural width of the stream immediately on the upstream side of the bridge. The waterway of any bridge is generally made less than the natural waterway of the stream, for which guide banks or training banks are provided. The amount of afflux will determine the top levels and lengths of the guide banks and also the "free board"

The greater the afflux, the greater is the velocity produced, which will form a greater scour entailing a greater depth of foundation. It also effects the discharge under the bridge.

Afflux may be taken 60 cm in alluvial and deltaic regions, 90 to 120 cm in trough regions and higher in steep reaches of rivers with boulders and rocky beds (for rough calculations).

Free-board or Vertical Clearance

Is the difference between the designed highest flood level, allowing for afflux, and the lowest part of the bridge structure. The free board for high level bridges shall not be less than 60 cm.

For arched openings of high level bridges, the clearance below the crown of the intrados of the arch should not be less than 1/10th of the max: depth of water plus 1/3rd of the rise of the intrados. Springing line should be above the high flood level in small bridges.

The following min: vertical clearance is recommended—IRC: 5-1970

Discharge in cu.m/sec.	*Min: vertical clearance*
Up to 0.3	15 cm
Above 0.3 and up to 3.0	45 ,,
Above 3.0 and up to 30.0	60 ,,
Above 30.0 and up to 300	90 ,,
Above 300 and up to 3000	120 ,,
Above 3000	150 ,,

In structures provided with metallic bearings, no part of the bearing shall be at a height less than 50 cm above the design highest flood level taking into account the afflux.

When the waterway is restricted to such an extent that the resultant afflux will cause the stream to discharge at erosive velocities, protection against damage by scour shall be afforded by deep foundations, curtain or cut-off walls, rip-rap, bed pavement, bearing piles, sheet piles, or some other suitable means. Likewise, embankment slopes adjacent to all structures subject to erosion shall be adequately protected by pitching, revetment walls or other suitable construction.

Considerations of economy require that small culverts in contrast with relatively larger structures across defined channels, need not be designed normally to function with full clearance for passing floating matter.

3. BRIDGE FOUNDATIONS

Determination of the Maximum Depth of Scour

Scour occurs when the bed velocity of the stream exceeds the velocity which can move the particles of the bed material. Velocity varies with the gradient, the hydraulic mean depth and the character of the bed and banks and depends more on the depth than on the gradient. A river has to adjust its velocity to what its bed and banks can stand by changing its section. The prevailing velocities of winter supplies in rivers may be as low as 3 cm to 150 cm/sec. and in floods 4.5 m to 9 m/sec. As greater scour follows increased velocity, the tendency is for the deeper parts of the section to become deeper still. The velocity of a falling river is greater than that of a rising river. When the velocity is retarded silt is dropped and when the velocity is increased, silt is picked up. Scour is worst when the river is falling.

Depth of scour should be ascertained as far as possible by actual soundings at the proposed site of the bridge during or immediately after a flood before the scour holes have had time to silt up. Due allowance should be made for increase in scour resulting from obstructions (contraction) at the bridge which will increase the velocity and hence the scour. The scouring action of the current is not uniform all along the bed width, it is not so even in straight reaches; there is deeper scour than normal at the piers or other obstructions and also at bends. Therefore, the maximum scour depth has to be determined.

Normal Depth of Scour in alluvial streams in regime condition, on a straight and unobstructed part (where a bridge structure does not obstruct the flow), and where the effective linear watrway provided is not less than the regime width, based on Lacey's theory :— (Also see under "Irrigation.")

$$d = 0.473 \left(\frac{Q}{f}\right)^{1/3}$$

where:

d = normal depth of scour in metres below the highest flood level corresponding to the value of Q adopted (it is the depth of non-scouring flow below flood level for regime conditions in a stable channel). On an average the actual depth of scour at a pier is twice the normal depth of scour according to Lacey;

Q = the discharge adopted for the design in cu.m/sec.;

f = Lacey's silt factor for the bed material (values given in Section 17)

$= 1.76\sqrt{m}$, where m is the mean dia. in mm.

Depth of scour under abutments is generally taken = 1.5 d and under piers 2 d. Some engineers consider that there is additional local scour due to the presence of the pier and which may be taken equal to the width of the pier.

Where the waterway of a regime channel is obstructed by the construction of a bridge and where the effective linear waterway provided is less than the regime width of the channel, the depth of scour will be greater and can be found from the equation:

$$d' = d\left(\frac{W}{L}\right)^{2/3}$$

where:

d = normal scour depth when L = W;

d′ = depth of scour under the bridge, in metres, below the flood level; (when L is less than W);

W = regime width of the stream;

L = obstructed width (effective linear) of waterway under the bridge.

In calculating the velocity through the bridge, 10 per cent should be added to allow for the effect on the velocity of the contraction of the current past the piers and abutments. The bottom of scour which is counted from the surface of water will be higher at upstream by the amount of afflux.

Due allowance should be made in the calculated depth of scour for increase in scour resulting from possible concentration of flow through a portion of the waterway.

The normal scour depth should be multiplied by the following factors for obtaining the max: scour depth:—In a straight reach of the stream and when the

bridge has no piers obstructing the flow	1.27 d
at a moderate bend	1.50 d
at a severe bend	1.75 d
at bad sites on curves or where diagonal currents exist, or the bridge is a multi-spans structure	2.00 d
at right angled bend	2.00 d
at noses of piers	2.00 d
at upstream noses of guide bunds	2.75 d

The above rules apply when no bed floor is provided and the stream is free to scour as it may. Floors of bridges and culverts are sometimes paved and bounded by deep curtain walls. For large spans it is generally cheaper to carry the foundations below the scour depth but for small spans pavement may be cheaper. (See under "Culverts"). A covering of dumped stones or concrete blocks, old masonry structure or matted vegetation will prevent the bed from scour.

A useful rough rule, applicable to torrential nala beds of boulder and shingle is, that the depth of scour below bed level in straight runs, where no restrictions to waterway are caused by the structure, is equal to half the maximum depth of water at high flood level, and on curves it is equal to the maximum high flood depth. This will be one-third the maximum depth of water in alluvial soils in straight runs.

Depth of Bridge Foundations

The depth of foundations is determined by considerations of the safe bearing capacity of the soil after taking into account the effect of scour. The foundations should be taken below the scour level and to the level at which there is minimum variation in the moisture content of the sub-soil.

In *Erodible beds* if 'D' is the anticipated maximum depth of scour below the designed highest flood level including that on account of possible concentration of flow, the minimum depth of foundation below the highest flood level should be taken 1.33 D (at site), subject to a minimum depth below the scour line of 2 metres for arched bridges (for both abutments and piers) and 1.2 metres for other types of structures unless directly resting on rock where rock is met with at a lesser depth. In such cases the foundations shall be securely anchored for about 0.3 metre into rock and about 0.6 metre where there is other hard material. If the rock is sloping, the masonry should be anchored to the rock by means of dowel bars for about 30 cm.

For small bridges of moderate height on dry land sufficiently solid, a depth of 1.5 to 2.0 metres below the bed level in non-erodible streams will be found sufficient. (Also see under "Size of Abutments"). In the case of small culverts in non-erodible beds, foundations should be taken down to at least 45 cm to 60 cm from the bed level. Depth of foundations on lands which are not exposed to the erosive action of stream currents may be taken down to a depth on a firm bed as calculated from the Rankine's formula given in Section 6 under "Depth of Foundations".

Foundations may be located at a comparatively shallow depth below the surface of the bed if the bed material is protected against scour by a bed pavement (bed floor) in conjunction with curtain walls (drop walls), or sheet piling, etc.

Where the foundations of the abutments are carried below the scour depth and the water-way of the stream is not restricted and the velocity is not expected to increase during the floods beyond the critical velocity (for critical velocity see under "Irrigation") for the bed material, there should be no necessity of providing a pavement.

Where the water-way has been restricted but the velocity is not expected to exceed about 2 metres/sec. loose stone or block pitching may be constructed extending up the end of the wing walls. Where the water-way has been considerably restricted so that the resulting velocity exceeds 2 metres/sec. and also for the culverts founded on erodible soils, floors should be paved and a paved apron given downstream extending up to the end of the wing walls.

Width of foundation in the direction of the span levfth need not exceed a metres.

The *pavement* should be designed to withstand upward pressure resulting from the head of water in the stream. The top of the floor is kept about 30 cm below the bed level of the stream. Drop walls are provided at the upstream and downstream ends carried to depths depending on the velocity of the flow and erodibility of the bed material or scour depths but not less than the depth of the footings.

In the absence of any precise scour data (in erodible soils) the abutment foundations should be taken to at least 1.5 metres below the natural bed level, upstream drop walls 0.9 to 1.5 metres deep, and downstream drop walls 1.5 to 2.5 metres deep from the top of the floor. (Illustrations in the pages following show the minimum depths for drop walls in good soils.) This will prevent erosion or undercutting.

Pavements and drop walls are not provided in hard soils or in canals where flow is controlled unless so designed, and in some cases only a single drop wall on the downstream side will suffice. Where the outfall of the stream is a free overfall a suitable cistern with the drop wall (or baffle wall) must be added for the dissipation of energy and stilling of the ensuing current. (See under "Water Cushion" in Section 14 — Hydraulics).

In calculating the pressure at the base of a substructure, reduction on account of skin friction on the sides of the substructure shall normally be ignored. The bearing capacity of soil generally increases with depth, as such the safe load on a deep foundation is generally greater than that on a shallow foundation. If the depth of a stratum suitable for foundation is very great, or when a very large footing area is required in order to reduce the pressure to a proper amount, the use of piles is often indicated.

Well Foundations

The common well foundations are thick hollow cylinders of masonry which are placed on well curbs and sunk to the required depth, or are placed on hard beds. The hollow cylinders are plugged at the bottom with cement concrete to have a watertight foundations, the interior is filled with sand or concrete and the top again sealed with a thick layer of concrete on which piers and abutments are built.

If a well is to be made on a dry piece of ground, an open excavation a little larger (about 1.2 to 1.5 metres) than the well is necessary. Excavation is done up to the sub-soil water level before a curb is placed. In shallow waters the area is surrounded by an ordinary earthen "coffer-dam" and water pumped out of the enclosure. (Pumping water out of foundations has been described in the Section on "Design of Foundation".) Bore holes are sunk to determine the nature of the sub-soil strata before well sinking.

Recommended Safe Loads on Different Soils for Bridge Foundations:

Soil Material	Tonnes per sq. metre	
	Deep Foundations	Shallow Foundations
Alluvium and silt	24	15
Confined quicksand	34	—
Fine sand	44	29
Coarse sand and gravel	54	39
Mixed sand and clay	34	24
Hard clay	58	44
Hard pan	88	68
Rock	195	195

Soil material is considered below maximum depth of anticipated scour.

Well Curbs and Steening have been described under "Water Supply".

The usual shape adopted for small wells up to 6 metres diameter is a single circular or octagonal and for large wells a combination of two or more cylinders can be made. Twin octagon are usually considered best. They are connected at the top by a cap to give the required base area.

Much reliance cannot be placed on the load taken due to the effects of skin friction for the reasons that: It is not possible to measure accurately the depth which can contribute towards skin friction. The co-efficient of friction may by very substantially reduced by the vibrations of the super-structure in floods.

The thickness of the masonry shell or steening is about 45 to 90 cm according to the diameter of the well, span and load. For large wells thicker steening is used up to one-quarter of the external diameter of the well. The shaft of a well is from 1.8 to 2 4 metres diameter (prefer 2.4 metres or more) to give room for operating the excavating gear. When the well has been sunk fully, the bottom of the well is plugged with 90 to 120 cm of cement concrete under water and when the concrete has set the water is pumped out and the well shaft filled with sand within about 90 cm from top. The top portion is again filled with cement concrete and RCC capping built to form a platform for the pier.

Where wells are less than 4.6 metres apart, alternate curbs should only be pitched, and their walls built and fully sunk before the intermediate wells are begun. No well should be plugged until those immediately

adjacent are fully down. Before plugging, all loose sand or mud which may have entered since undersinking was stopped, should be dredged out, and the deepest wells should be plugged first.

In excavating, the depth of excavation below the cutting edge of the well curb should never exceed 1.2 metres in a sandy bed or a "blow" is certain to take place tending to throw the well out of plumb. When a well sticks in sinking owing to increasing hardness of or friction with the river bed material, a small charge of dynamite about 50 grams, placed in the centre should usually suffice, to get the well on the move again.

A Coffer-dam is a temporary enclosure built to exclude water from the working area and to permit free access to the area within, during the construction of a foundation or other structure that must be undertaken below water level. Coffer-dams are usually made of earth, timber or sheet piling. It is a sort of "bund".

The coffer-dam has a diameter of at least 3 metres more than the outside diameter of the well to be sunk. It may be built of sand, clay and boulder mixed together. A mixture of clay and gravels in equal proportions known as "puddle" is quite suitable. The finished bund should be 60 cm above water level with 90 to 150 cm width at the top for small enclosures and shallow waters say, up to 2.4 metres deep, and 2.4 to 3 metres above water level and 7.6 to 9 metres width at top for big dams and deep waters, with the material assuming its natural slopes on both sides. These bunds require protection on the outer sides, especially the toes, against being washed away and the same can be provided by stones, rubble or sand bags. Such enclosures can also be made by first placing the sand bags and filling inside with clay or sand. The over-all stability of a coffer-dam should be investigated as a dam or as an earth retaining structure.

Such type of earthen coffer-dams will suit situations where either the water is standing or has a velocity of less than 60 cm per sec. and depth not exceeding 2.4 to 3 metres. A sheet coffer-dam is generally adopted if the depth of the water is more than 3 metres. A single row of planks or sheets is driven vertically and earth dumped on both sides of it. A sheet pile will also be necessary on porous soils to stop infiltration of water. The size of wooden sheet piles can be 23×8 mm. Sheet pile walls of moderate height may be designed as vertical cantilevers embedded in the ground. If the face to be retained is more than 3 meters high, the piles may be tied back to anchorage. The piles have to be driven to sufficient depth, to stop infiltration of water and to be considered as "fixed" for bending moment calculations and well below the level to which excavation will be carried out. (Also see under "Piles and Pile Driving" in Section 6.)

In deep waters and where the velocity is high, a double row of sheet piles is driven and the space filled in with puddle. This space between the rows of piles is generally kept about 1.5 to 2.4 metres. The piles are closely fitted with longitudinal interlocking joints, or with guide piles, and securely strutted. For such type of works, steel sheet piling should be preferred. Timber sheet piles should be provided with cast iron shoes.

If the underground water happens to issue as a spring at any spot. it is localized by covering it with a vertical pipe or a cylinder and when the foundation work is completed, the pipe is plugged with cement grout and influx stopped permanently. A good method for dewatering the seepage water of the area in sandy soils or where the water-table level is high, is by driving G.I. tubes of about 50 mm dia. to a depth of about 3 to 3.7 metres. The bottom 0.9 to 1.2 metres of the tubes is perforated to admit water inside. Water is pumped out by suction from these tubes and collected in a common sump or pipe wherefrom it is pumped out. A rapid influx of water into the foundation trench which brings sand and silt with it is dangerous. Such sites should be surrounded by sheet piles driven down to firm strata. If the depth of the water is great and water leakage cannot be stopped by sheet piles, a caisson foundation will have to be adopted.

A **Caisson** is a water-tight box like structure or a chamber, made of wood, steel or concrete, usually sunk by excavating within it, for the purpose of gaining access to the bed of a stream and placing the foundatian at a prescribed depth and which subsequently forms part of the foundation itself. Caissons are adopted when the depth of water is great and the foundations are to be laid under water. Caissons are generally built on the shore and launched into the river floated to the site and sunk at the proper position. There are three types of caissons.

Open Caissons: Top is open and sides are detachable from the bottom. The masonry is built on the platform and the caisson sinks gradually, the sides are detached when the masonry with the bottom rests on the prepared foundation. The bottom is generally made of RCC. The caisson has sharp edge at the bottom and when the masonry is raised over it, it gradually sinks due to its weight. Cylinder caissons with dredging wells are adopted for average depths with large areas.

Box Caissons: They are closed at the bottom and open at the top. Big boxes of iron or RCC are built on the shore, launched and floated to the site where they are sunk by filling with stone or concrete and top finished off with a concrete cap. Box caissons are for small depths, and are sunk on prepared foundation. This foundation may consist of piles or it may simply have been formed by levelling an area of the bottom.

Pneumatic Caissons: Are closed at the top and open at the bottom. They are used where the depth is great and it is not easy to pump out the water. Compressed air is used to exclude the water and the process is more complicated than with the other two types of caissons. It has a working chamber in which the air is maintained above atmospheric pressure to prevent the entry of water into the excavation.

Steel Cylinders are usually made in lengths of from 1 to 3 metres without vertical joints and of 1.2 to 3 metres diameter, with plates 10 to 25 mm thickness with 40 to 50 mm internal flanges. The first section is provided with a cutting edge. Water is pumped out to assist its movement. The central core is excavated and filled with concrete or masonry.

The thickness in millimetres, to withstand the pressure of the water, of small thin cylinders may be found approximately from the formula: $t = dh/3.5$, where d=dia. in metres, h=depth of water in metres. This will stand a stress of 850 kg/sq.cm. The thickness should not be less than 6 mm.

Steel caissons or cylinders should be braced internally to prevent collapsing while sinking. Cylinder piers are used for river beds requiring well foundations.

4. DESIGN OF ABUTMENTS

(Design of Abutments and Wing Walls is also given in Section 7—"Masonry Structures", and Abutments for Arches under "Bridge Arches" in this Section.)

Abutments

The forces acting on an abutment are the weight of the end of the bridge span with any load that it may carry, the traction force of the train or trucks, the pressure from the earth fill and the surcharge due to live load, the weight of the abutment, centrifugal and wind forces acting on the bridge span and transmitted to the abutment and the reaction of the foundations. An abutment must be stable against sliding and overturning and against failure within the structure itself. It must be stable with and without the bridge in place. Abutments should be designed to resist saturated earth pressure.

The length of abutments at the top small normally be equal to the formation width.

It is not customary to consider an impact effect from moving loads since vibration is supposed to be entirely dissipated through the embankment. In computing the pressure on an abutment, equivalent live load surcharge in addition to the usual earth pressure is to be considered. If framed approaches are provided for at least 3.0 metres (10 ft.) lengths on either side of a bridge no live load surcharge need be considered (see page 19/21). The effect of the live load is reduced with the height of the abutment and the earth cushion over it. It is recommended in the Railway Code that dispersion of the surcharge load below the formation level or road surface may be taken at a slope of 1 horizontal to 2 vertical. (Also see Section 7.) The following figures show the amount of earth cushion if given over a culvert, no live load surcharge need be considered.

Height of abutment	Earth cushion	
up to 3 m	1.2 m	By "height of earth cushion" is meant the height between the top of abutment and the road level.
4.5 m	1.1 m	(When earth pressure is considered for design the pressure due to the weight of the "earth cushion" mentioned above will be taken as level surcharge as explained under "Retaining Walls.")
6 m	0.9 m	

Culverts under high overfills should be either of round pipe, reinforced concrete box type or the arch type. Abutment section of small heights suitable for roadway embankments will be heavy.

No filling should be placed behind the abutments during construction unless these are temporarily strutted apart or alternatively, until the deck is constructed and is capable of taking the load.

In ramming the "backing" of the abutments of bridges, it must be done in layers of not more than 30 cm in depth and each layer to have a slope of 45 deg. towards the ground level, the slope to commence from immediately behind the abutment. A fair amount of water must be used whilst ramming. Only sound material such as sand and sandy earth should be used and under no circumstances black soil is to be used for back filling. Behind abutments and wing walls full compaction must be done.

In filling in the approaches of a bridge, or the spandrels between small arches, the earth should be raised simultaneously with the wing walls in the former case and with the face wall in the latter, in order that the filling may be well trodden down under the feet of the labourers, and in filling in foundations and backing to revetments the earth-work should similarly be brought up level as the masonry proceeds.

Weep-holes or drain pipes not exceeding one metre spacing in both directions should be provided to prevent any accumulation of water and building up of hydrostatic pressure behind the abutment and wing walls. The weep-holes should be provided above low water level. 15 cm drain pipes can serve the purpose. (See Section 7)

Size of Abutments

The top width of abutments may be taken 72 cm for spans up to 3 metres and 80 cm for spans above 3 metres for road slabs. This provides 46 cm width for the backfill wall and 30 cm or 40 cm for the slabs eating according to the span. (IRC Special publication No. 13 recommends slightly less widths as tabulated hereunder.)

Width of abutments at ground level may be taken according to the following table :-

Height of abutment from ground level to top of bed stone	Widh of abutment for about 22 t/sq. m max: presure on soil	Width of abutment for about 44 t/sq m max. pressure on soil
1.5 m	158 cm	128 cm
1 8 m	173 cm	143 cm
2.1 m	190 cm	152 cm
2.4 m	207 cm	160 cm
2.7 m	233 cm	170 cm
3.0 m	265 cm	185 m
3.6 m	330 cm	218 cm

The above dimensions are for spans up to 3 metres, for spans above 3 metres, increase width by 8 cm. (IRC Special publication No. 13 gives slightly different dimensions as tabulated hereunder.)

The front of the abutments may be made vertical or with a batter of 1:12. The ratio of the height of abutment or pier to its base width shall not be more than 6. The bottom width will be 138 cm more which provides for a projection of 24 cm of concrete all round. Minimum depth below ground

level in ordinary soils should be 1.83 m (1.38 m for masonry and 45 cm for the concrete). This is for IRC heavy loading where no "approach slab" has been provided; surcharge due to live load is based on Bridge Code. It is generally safe to increase the permissible bearing pressure at the rate of about ½ tonne/sq. metre for each additional 30 cm of depth below one metre depth.

Equivalent Heights of Surcharge of Earth for Live Loads (IRC: 6)

In the case of bridge abutments, the concentrated surface live loads due to the wheel or track load on the back-fill shall be considered to have the same effect as equivalent heights of surcharge of earth shown below in metres: —

Depth of abutment below road level in metres	IRC Class AA & Class 70 R loadings		IRC Class A loading		IRC Class B loading	
	Single lane bridges	Multi-lane bridges	Single lane bridges	Multi-lane bridges	Single lane bridges	Multi-lane bridges
0.2	26.0	15.4	14.3	17.2	8.3	10.0
1.0	15.0	9.1	8.5	10.0	5.1	5.8
2.0	8.0	5.5	5.1	6.1	3.0	3.7
3.0	6.8	4.1	3.8	4.6	2.3	2.7
4.0	5.5	3.3	3.0	3.5	1.8	2.1
6.0	3.8	2.3	2.2	2.6	1.3	1.5
8.0	3.0	1.8	1.7	2.0	1.0	1.2
10.0	2.6	1.5	1.4	1.7	0.9	1.0

The above figures are based on the following values for the constants for the abutments and the backfill :

(i) Length of abutment=4.5 metres for single lane bridges and 7.6 metres for multi-lane bridges.
(ii) Angle of internal friction of the backfill=30 deg.
(iii) Weight of backfill...1600 kg/cu.m.
(iv) The resultant earth pressure acts in a horizontal direction.

If the abutments are designed to retain earth but are not protected (against scour) in front, the foundation shall be designed to withstand the earth pressure and other horizontal forces for the condition of a max: scour depth of 0.63 D in front of it, where 'D' represents the max: scour depth below highest flood level. In case the abutments are protected reliably in the front, relief due to earth pressure in front may be considered assuming the design scour at the toe of the protection work.

Soil material is considered below maximum depth of anticipated scour.

For portions of the foundation lying below the water level, full buoyancy effect should be considered, whatever be the type of soil in which the foundation is laid, for calculating the pressure on the soil.

Under the worst combinations of all loads and forces, no tension shall be permited under the foundations.

SIZE OF ABUTMENTS FOR CULVERTS

Span	1 metre to 4 metres						5 metres and 6 metres				
Ht.	1.50	2.00	2.50	3.00	3.50	4.00	2.00	2.50	3.00	3.50	4.00
D	0.15	0.20	0.25	0.30	0.35	0.40	0.20	0.25	0.30	0.35	0.40
J	0.50	0.70	0.95	1.10	1.25	1.40	0.60	0.85	1.00	1.20	1.40
L	0.30	0.30	0.30	0.30	0.30	0.30	0.40	0.40	0.40	0.40	0.40
R	—	—	0.10	0.20	0.30	0.50	—	0.10	0.20	0.40	0.60
B	1.25	1.50	1.80	2.00	2.20	2.40	1.50	1.80	2.00	2.25	2.50
C	2.45	2.70	3.20	3.60	4.00	4.60	2.70	3.20	3.60	4.25	4.90

SIZE OF WING WALLS

Span	up to 2 meters						about 3 metres					
Ht.	1.50	2.00	2.50	3.00	3.50	4.00	1.50	2.00	2.50	3.00	3.50	4.00
D	0.18	0.23	0.28	0.33	0.36	0.43	0.19	0.24	0.29	0.34	0.39	0.44
J	0.45	0.57	0.70	0.82	0.95	1.07	0.46	0.59	0.71	0.84	0.96	1.09
B	1.13	1.30	1.48	1.65	1.83	2.00	1.15	1.33	1.50	1.68	1.85	2.03
C	1.93	2.10	2.28	2.45	2.63	2.80	1.95	2.13	2.30	2.48	2.65	2.83

Span	about 4 metres						about 6 metres				
Ht.	1.50	2.00	2.50	3.00	3.50	4.00	2.00	2.50	3.00	3.50	4.00
D	0.19	0.24	0.29	0.34	0.39	0.44	0.25	0.30	0.35	0.40	0.45
J	0.48	0.60	0.73	0.85	0.98	1.10	0.63	0.75	0.88	1.00	1.13
B	1.17	1.34	1.52	1.69	1.87	2.04	1.38	1.56	1.73	1.90	2.08
C	1.97	2.14	2.32	2.49	2.67	2.84	2.18	2.35	2.53	2.70	2.88

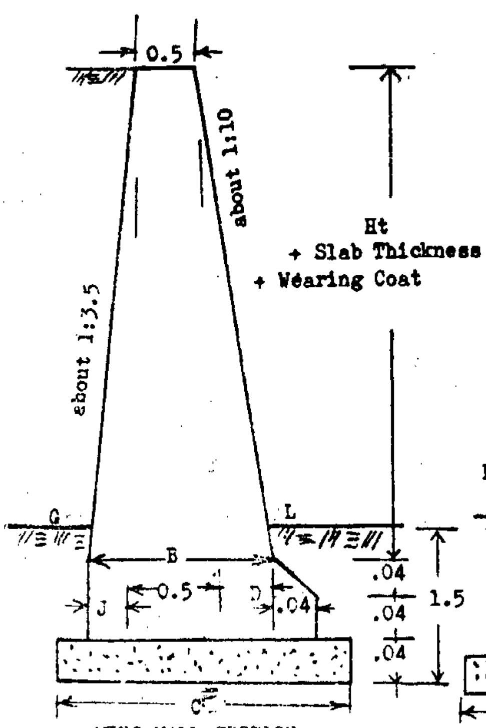
Based on IRC Special Publication No 13.

Notes :— 1. Abutment and Wing Wall sections are applicable for a minimum bearing capacity of the soil 16.5 t/sq. metre. For soils with higher or lower bearing capacity, the sections should be adjusted accordingly.

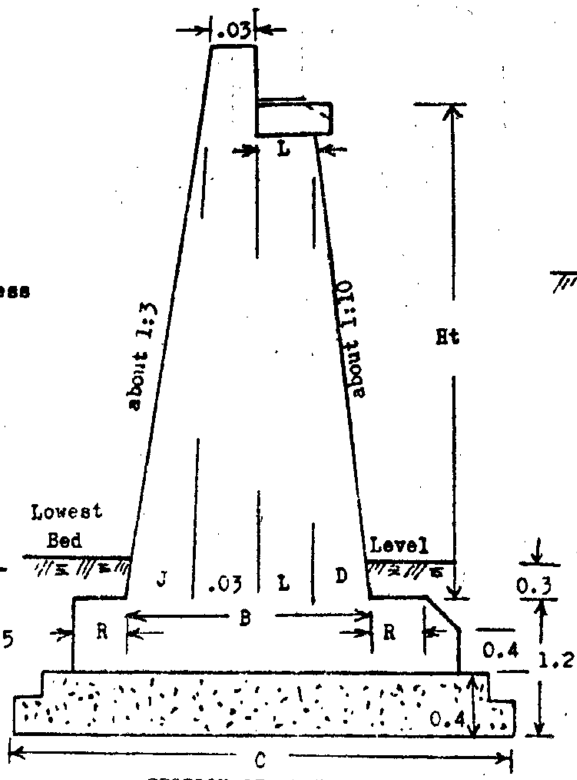
2. The various dimensions to be suitably adjusted to suit the size of bricks where necessary.

3. The sections are applicable for culverts designed for IRC class 70 R or two lanes of class A loading without provision of *approach slabs.*

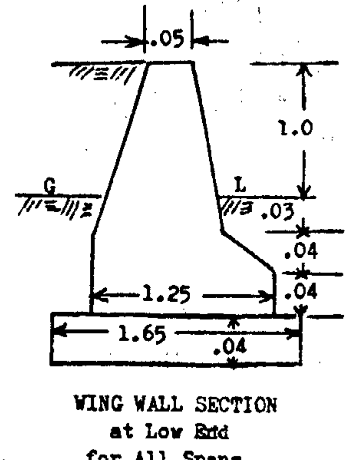
4. The sections shall be in brick masonry or coursed rubble masonry in cement mortar 1:3. Foundation concrete shall be 1:3:6 cement concrete.



WING WALL SECTION at High End



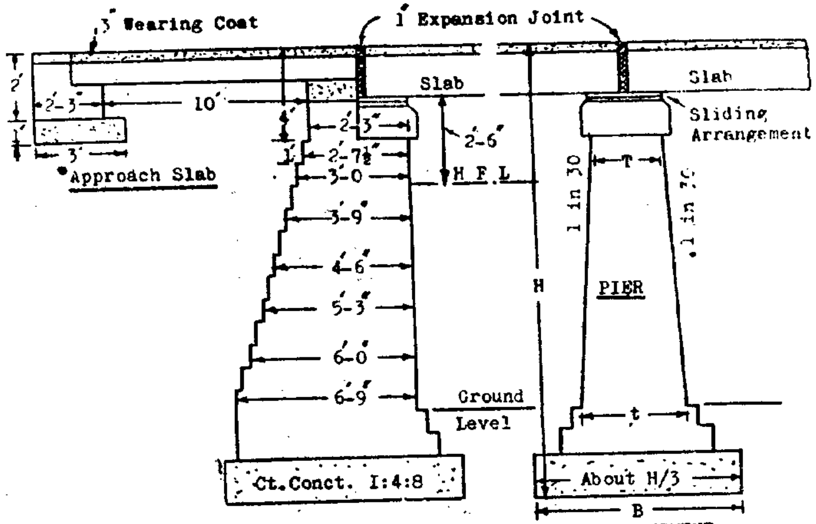
SECTION OF ABUTMENT



WING WALL SECTION at Low End for All Spans

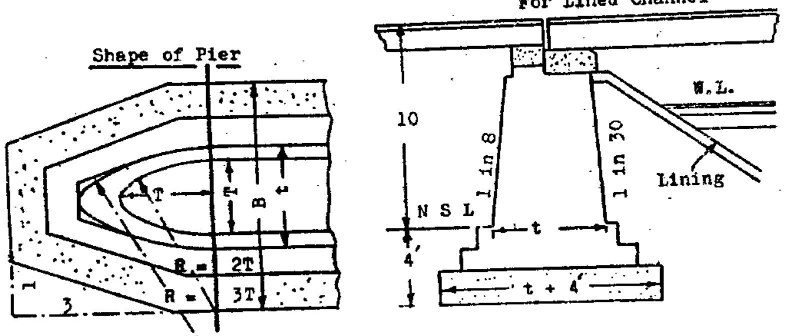
Based on IRC Specifications

All dimensions in metres



ABUTMENT

ABUTMENT
For Lined Channel



Sections Suitable For I.R.C. Class "A" Loadings

The illustration shows work already built

Width at top of Piers, Abutments and Wing Walls according to IRC: 40-1970

Cement Masonry

The min: width at the top of piers and abutments of slab and *girder bridges* just below the caps shall be as given in the table :—

Span in metres	*3m*	*6m*	*12m*	*24m*	*40m*	*60m*
Top width of piers carrying simply supported span, in cm.	50	100	120	160	200	220
Top width of abutments and of piers carrying continuous spans, in cm.	40	75	100	130	170	190

The top width of wing walls and returns walls shall be not less than 50 cm

Roadway Approach Slab

If a reinforced concrete slab is provided, covering the entire width of the roadway, as an approach on either side, extending for a length of not less than 3.5 metres, no live load surcharge in the design of the abutments need be considered. This slab is considered to distribute the live load over a wider area and decrease the load effect. The earth under the slabs must be well rammed before slabs are laid. When the span of a bridge is equal to or less than one metre, provide 23 cm thick metalling in a length of 1.5 metres normal to the abutments instead of RCC approach slab. If the embankment is over 3 metres high, roadway approach slabs should invariably be used, or width of abutments increased taking into account equivalent height of surcharge of earth due to the live load as tabulated.

The approach slabs may be 3.5 metres wide, 30 mm thick, reinforced with main bars 20 mm dia. at 12 cm centres with distribution bars 16 mm dia. at 28 cm centres, for arterial roade. For light load village roads, the slabs may be 1.5 metres wide 21 cm thick, reinforced with 12 mm. dia. main bars at 21 cm centres. Every fourth main bar to be bent up. Main bars are parallel to the road length.

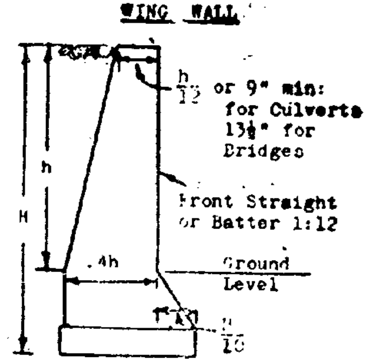
5. WING WALLS & RETURN WALLS

A wing wall is a splayed extension of an abutment of a bridge or a culvert and its function is to retain the side slope of the embankment, and to guide the water through the openings where required. The top of the wing wall should extend level past the shoulder of the embankment and then it should slope downward at natural angle of repose of the earth in the embankment to a height of from 0.6 to 1.2 metres above the ground level or above water level to prevent water outflanking the culvert, where necessary.

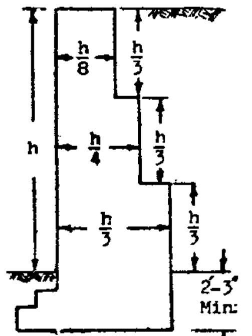
For embankment having a height less than 2.4 metres the height of the low end of the wing should be made equal to half the height of embankment, and usually not less than 60 cm.

Wing walls for culverts, say up to 3 metres span, generally have vertical faces with batter at the back of 1:4 and 23 cm min: broad at the top on the square. In the case of bridges, say 4.5 to 9 metres span, the walls have a face batter of 1:12, a back batter of 1:6 built in steps and a top breadth of 46 cm (35 cm min:) Where the height of the wing wall is reduced towards its outer edge, the thickness is also reduced accordingly.

WING WALL



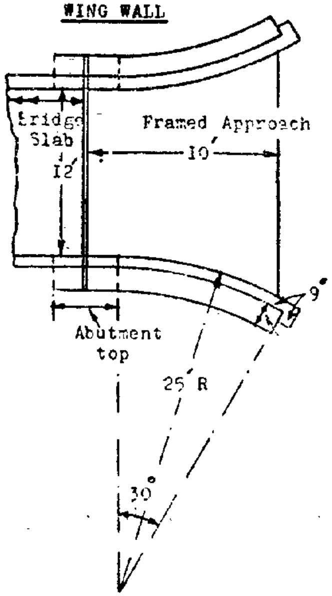
If the soil in the backfill is poor the wings should be extended to ground level. The lengths of all wing walls or return walls should be such as to prevent the earth from falling into the water-way. The cross-section is designed essentially as a retaining wall which should be for a saturated back-fill and follows similar lines to those of abutments, the chief difference being the absence of the heavy vertical load due to the bridge deck. This is an important difference in that there is likely to be a certain amount of unequal settlement between abutment and wing walls, so that in all but very small bridges there should be a free joint (a small gap) between the abutment and the wing wall right from the bottom of the foundations to the bottom of the parapets. Such a joint is also essential where the two structures are founded on different levels or on soils of different bearing capacity. The joint should be filled with bitumen and sand and should also, preferably, be provided with water stops as these joints are likely to open.



RETAINING WALL
(Economical Section up to 10 height - used on U.K. Rlys.)

Wing walls may be made at an angle with the abutment, called *splay* of the wings, generally 30° (min.) or 45°; (45° is most common), or at right angles with the abutment called **return walls,** making a U-shape. The angle is selected to meet the peculiar conditions of the site. Angle wings are made where waterway is restricted, to prevent scour, the large angle corresponding to the wider spread of the stream in high water and the smaller to a more confined channel. Splayed wings are sometimes made curved where the thickness can be slightly reduced due to the arch action. Wings are generally terminated with right angle returns. Wing walls should be so located that their front faces are flush with the edge of the bridge opening. The return wing walls (U-abutment) are economical where the banks of the river or stream are steep and of hard soil not subject to erosion and where the waterway is pretty wide and there is no necessity of guiding the current. This type is not safe for a river subject to heavy floods as there is a tendency for the flood water to damage the embankment. Where the soil is neither hard nor soft, the wing walls are cut short and returns are carried parallel to the embankment. The wing walls are stopped at the toe of the embankment but return walls are taken sufficiently inside so that the earth slope along them terminates outside the waterway. The

WING WALL



return wings confine the formation of the approaches and add to their strength.

Wing walls should always be founded on natural ground and they should not be made longer than is necessary, to allow the earth of the approach filling to be trimmed to its natural slope. Where streams are bridged, the wing walls should be of such length and so placed as to ensure that water cannot get behind them, more especially at flood times, and foundations should always be carried down far enough to avoid scour. The minimum depth recommended for bridges should be followed. Where long wing walls are to be constructed it will be necessary to provide joints at not more than 9 metres apart. A simple keyed joint extending through the full thickness of the wall is all that is necessary.

It has been reported that wing walls flared at 30 deg. used with pipe culverts increase the capacity of the culvert over that obtained with straight end walls, and especially so when set flush with the end of the pipe.

Wing walls at 45 deg. to the direction of the slope of a surcharge fill can be designed as for a level fill without appreciable error.

6. DESIGN OF PIERS

The function of a bridge pier is to support the intermediate ends of bridge spans with minimum obstruction to the stream.

Shape of Piers

Cut-water: The upstream nose of a bridge pier shaped for ease and smooth flow of water past it. Cut-waters need not be very long. Best shape is a half-round or semi-elliptical.

Ease-water: The downstream nose of a bridge pier shaped to promote merging of the water flowing out of the adjacent openings of a bridge cut-water Best form is either half-round or pointed.

Although the force of the current is reduced with certain shapes they are not all economical. The cut-water shape shown in the figure at page 19/20 is considered to give the minimum resistance to the stream combined with economy of material and simplicity in construction. The cut-water shape should be carried down to the base.

Extract from Journal of "American Society of Civil Engineers," Vol. 82, p. 334 :—

"Best practical form of nose is either the half-round or the semi-elliptical, these being better than the pointed nose. Back water may be appreciably reduced by using an efficient tail. Best practical form of tail is either half-round or pointed. A 90-deg. nose is not satisfactory; best angle for a pointed nose is 45-deg. or less. For piers of same design upstream and downstream, the half-round shape gives least backwater."

Experiments carried out at Poona Hydraulic Research Station show that there is very little difference between a highly tapering ease-water and an equilateral one bounded with circular arcs. Therefore, the downstream noses could be equilateral. Pillars on dry lands may be rectangular.

Minimum top widths for simply supported spans should be such as to accommodate the bridge seat with a clearance of about 15 cm between the bays (two bearings). See also under "Bearings". Length of a pier is usually made to extend $1\frac{1}{2}$ times the top width beyond the centre-line of the outer trusses or girders; the depths of ease-water and cut-water are not counted in the length.

Piers are generally given a batter of 1 in 30 for brick in cement mortar 1:4, and 1 in 24 for brick in good lime mortar. Max. batter given is 1 in 12, less for higher piers. Short piers may have vertical sides. The width of the piers at foundation level is not less than 1/3 rd of their total height. Full size of the pier foundations should be checked with the bearing capacity of the soil.

Compressive Stress Reduction Factors for Slenderness Ratio :—

l/d	Factor	l/d	Factor
4	0.88	14	0.40
6	0.80	16	0.35
8	0.70	18	0.30
10	0.60	21	0.25
12	0.50	24	0.20

(BS Code of Practice CP III)

Slenderness ratio $= l/d$.

l is effective height, d is least lateral dimension, which may be taken at the top of the foundation if the pier is uniformly battered.

Other general principles of stability described in Section 7 should invariably be followed.

Pier Caps: The function of a pier cap is to distribute the load from the bearing evenly over the area of the top of the pier. Pier caps generally are of hard stone, plain or reinforced concrete.

When the load of the decking is transferred to the pier cap, in the form of concentrated strip loads through bearings, pier cap shall be of thickness of at least 225 mm up to a span of 25 metres and 300 mm for longer spans.

The cap shall be reinforced with 2 per cent mild steel or 1.5 per cent high yield strength deformed bars, distributed equally at top and bottom and provided in two directions both at top and bottom. The reinforcement in the direction of the length of the pier shall extend from end to end of pier, while the reinforcement at right angles to this shall extend for the full width of the pier cap. In addition to this, reinforcement provision shall be made for two layers of mesh reinforcement each consisting of 6 mm bars at 75 mm centres in both directions placed directly under the bearings.

Top Width of Piers for IRC heavy loadings for spans other than arches

Span in metres	up to 3 m	4 m to 5 m	6 m to 7 m	8 m to 10 m
Top width T	60 cm	70 cm	80 cm	95 cm

To lighten foundation loads piers may be made hollow. In navigated waters and also in streams likely to bring down large pieces of debris in flood time, protective fendering may be needed, and in some

cases it may be justifiable to enclose the whole pier in a protective shell of masonry or concrete.

The possible vertical and horizontal forces acting on a bridge pier are:—

(*a*) End loads of two adjacent bridge spans including all dead and live loads, impact on either one or both spans. Eccentricity of pier loading must not be overlooked where the two adjacent spans differ widely in length.

(*b*) The weight of the pier itself: If the foundation is pervious, the weight of that portion of the pier below water level should be reduced by 1000 kg/cu. m. (See (*d*) below).

(*c*) Reactions of the foundation: The pier should be stable against:— Sliding downstream; sliding parallel to the axis of the bridge; overturning about the downstream toe; overturning in the direction of the exis of the bridge. The maximum unit pressure at the downstream toe and at the side should not exceed the allowable masonry and foundation pressure. (Also see under "Skew Bridges").

(*d*) Where water can enter under a pier or abutment an additional force, *viz.*, the *buoyant force or uplift*, of intrusive water should be considered especially those of submersible bridges, assuming that the fill behind the abutment has been removed by scour. No buoyancy is to be considered where bridge is founded on an impermeable strata while full buoyancy should be allowed for bridges founded on coarse sand or shingle. To allow for full buoyancy a reducton is made in the gross weight of the pier or abutment affected equal to the weight of a volume of water of the submerged portion.

(*e*) Impact due to Live Load

Impact effect is produced by sudden application of load, blows or shocks due to unevenness of the road surface or obstacles, causing momentary increase in the stresses. It is greater on a short span than on a long span and within certain limits increases with the speed of the vehicle.

Impact factor fraction for IRC Class A and Class B loadings for spans 3 to 45 metres:—

$$\frac{4.5}{6+L} \text{ for concrete bridges; } \frac{9}{13.5+L} \text{ for steel bridges.}$$

This will be 50.0 per cent for concrete bridges and 54.5 per cent for steel bridges for spans up to 3 metres, and 8.8 per cent for concrete bridges and 15.4 per cent for steel bridges for spans of 45 metres or more.

Where L is the length of the span is metres. This gives percentage of the live load which should be added to the live load. The maximum impact is limited to 50 per cent and which may be taken for spans less than 3 metres.

For IRC Class AA and Class 70 R loading, impact may be taken 25 per cent for spans up to 12 metres in the case of RC bridges and up to

23 metres in the case of steel bridges, beyond which in accordance with the above equations. This does not apply to suspension bridges and footpaths.

Impact effect on a railway bridge is much more, and which depends upon the weight of the engine to be used on the particular track.

The impact effect is considerably reduced if there is an earth cushion over the road slab or arch crown. It is considered that where there is a filling of not less than 60 cm including the road crust, the impact percentage to be allowed in the design may be assumed to be only one-half of what has been specified above, and this may be practically nil for a cushion of about one metre. But weight of the earth fill will be added to the dead load.

(*f*) **Traction Forces** producting longitudinal thrust due to breaking and acceleration act parallel to the centre line of the bridge and tend to overturn the pier and abutments in the plane of the force. It is about 20 per cent of the total live load on the span. This is allowed for in design according to the following equations, on any span of a bridge whether single-lane or multi-lane:—

(i) For IRC Class AA loading = 14 tonnes.

(ii) For IRC Class A loading = 5 + 0.4 L in tonnes, where L is the span in metres under consideration.

(iii) For IRC Class B loading = 60 per cent of Class AA loading.

All the longitudinal forces (consisting of traction forces and resistance offered by the bearings to movements due to temperature changes) are generally taken equal to 0.03 for roller bearing and 0.25 for sliding bearings, of the total dead load and live load reactions on the bearings. Full force is taken to act on a fixed bearing. The longitudinal force is rssumed to act 1.2 metres above the crown of the road. No increase in impact effect is made on the stresses due to longitudinal forces. The lateral bending and shear effects of the longitudinal forces should be taken into consideration in designing cross girders and floor beams.

(*g*) Centrifugal Force

Centrifugal force is produced where a bridge is situated on a curve:—

$$C = \frac{WV^2}{127R}$$

C = horizontal load due to centrifugal force normal to the traffic in tonnes per metre length of the span under consideration, W = total live load in tonnes per linear metre, V = max: design speed of the vehicles using the bridge, in kilometres per hour, R = radius of curvature in metres.

The centrifugal force is considered to act at a height of 1.2 m above the level of the carriage-way. No increase for impact is made on the stresses due to centrifugal action. The overturning effect of the resultant forces due to centrifugal movement is considered for the design of all piers and abutments.

Bridges should not be made on curves unless absolutely unavoidable.

(*h*) **Temperature Effects** — Forces resulting from the expansion and contraction of the superstructure due to temperature changes: See Section 5.

The stress or reaction produced in a member (or the support) restrained from expansion or contraction is equal to the stress in the restrained member × its cross-sectional area. The following range of temperature are generally assumed in design—

Steel Structures

Moderate climates : from minus 18 deg. C. to plus 50 deg. C.

Extreme climates : from minus 35 deg. C. to plus 50 deg. C.

Concrete Structures:

Provision should be made for expansion of 8 mm per 10 metres length of exposed structures.

(*j*) **Horizontal Forces Due to Water Currents** on any part of a road bridge which may be submerged in running water shall be designed to sustain safely the horizontal pressure due to the force of the current. On piers parallel to the direction of the water current, the intensity of pressure can be calculated from the following equation:

P = 52 KV (Generally taken for depths of water 6 metres or more).

P is the intensity of pressure due to the water current, in kg/sq. m (To arrive at the total pressure this has to be multiplied by the area in sq. m of the face of the pier exposed to the water current).

V is the velocity of the current at the point where the pressure intensity is being calculated, in metres per second.

K is the co-efficient according to the shape of the nose of the pier, having the following values:

1.50 for square ended piers and for the superstructure,

0.66 for circular piers or with semi-circular ends,

0.70 for piers with triangular cut ends, with angle between the faces of 60°,

0.90 ditto. with 90°,

0.50 for piers 5 to 6 times longer than their breadth with triangular cut and ease waters substending an angle of 30° and less,

0.43 for piers with cut and ease waters of equilateral of circle,

0.47 for piers with arcs of cut waters intersecting at an angle of 90°,

Trestle columns are treated as solid with value of K as 1.25.

Some engineers take:

(*i*) Horizontal pressure due to static head:

$$= \frac{WH^2}{2} \times \text{area of pier face (or } 2 \times \text{area of cut-water face).}$$

W is weight of water and H is depth of water (height of pier) plus afflux plus head due to velocity of approach. In case of submersible bridges, additional pressure WH will be taken (horizontal pressure overturning the bridge).

H is afflux + depth of road slab.

(*ii*) Pressure due to impact of water and debris:

$$= \frac{WV^2}{2g}\left(\frac{a}{A-a}\right)^2$$

where:

a = area of obstruction,

A = area of flow just upstream of site,

g = acceleration due to gravity.

(*iii*) Pressure due to eddies:

$$= W\frac{(V_1-V)^2}{2g}$$

V_1 = velocity through the opening,

V = velocity of approach, *i.e.*, velocity of the stream.

Where the pier is at an angle to the current, the velocity should be resolved into two components, one parallel and the other normal to the pier. As the velocity of the stream is maximum near the surface and much less near the bottom, the point of application of the total pressure (centre of pressure) is taken at 1/3 of distance measured from the top between the upper and the lower wetted limits of the surface under consideration. A force of 20 per cent of the pressure of water parallel to the pier should be taken as acting at right angles to the current and normal to the pier.

In case of a bridge having a pucca floor or having an inerodible bed, the effect of cross-currents shall in no case be taken as less than that of a static force due to a difference of head of 250 mm between the opposite faces of a pier.

The effects of the longitudinal forces and all other horizontal forces should be calculated up to a level where the resultant passive earth resistance of the soil below the deepest scour level (floor level in case of a bridge having pucca floor) balance these forces.

The overturning horizontal forces mentioned above need not be calculated except in the case of high bridges, submersible bridges or where the velocity of the stream is very high.

Piers should be located approximately parallel to the direction of the current so as not to cause a shift in the river channel, erosion of the foundation bed or unnessary obstruction to the flow of the stream resulting in increased backwater upstream in times of flood. A skew span shall be avoided as far as possible.

Permissible Working Stresses should not be exceeded for any combinations of above stated stresses that can co-exist except when temperature stresses are added when the permissible stresses may be increased by 15 per cent.

Trestle Bent Piers: (The term "bent" is used for a framed pier consisting of a group of two or more vertical piles or posts braced horizontally and diagonally which support a deck of a bridge or false work.) Trestle bents may be built of either RC, steel or wooden posts. Such type of piers are sometimes used in viaducts (over a dry dip) for economy and ease of construction. They are otherwise suitable only where the channel bed is fairly firm and not suited to rapid streams on stony beds. Trestles have the disadvantage of inducing scour which can be reduced by providing pitching in the bed. They are less stable than solid piers. High trestle bents are inclined (footings are spread out) about 1:8 to give additional stability. They are very suitable for low crossings of narrow paths. See under "Temporary and Wooden Bridges".

Pile-Bent Piers: Are now commonly used for piers and abutments. Piles of RC, steel or wood are used to support the main girders, which are capped with a beam. Piles are driven deep into the bed and also connected laterally above, and are about 1.2 metres apart. Pile bents are strong and stable and are suitable for deep streams with muddy bottoms. RC trestle bridges are suitable for spans of 3.7 to 6 metres.

Column Bent Piers: Two or more columns are built on a solid foundation to support the main girders, and which are connected laterally by means of beam and braced together. They are lighter than masonry piers and are used for continuous spans.

Cribs (Piers): are generally made of wooden sleepers which are placed transversly in layers, which may be spiked together where required more or less permanently. The bottom is usually filled with rubble to make it heavy for stability in running water. Cribs are quite stable and suitable for temporary jobs and in swift and shallow streams.

Cylinder Piers: See "Steel Cylinders" under "Well Foundations" given earlier. Cylinder piers consist of cast iron or mild steel cylinders sunk into the bed up to a solid foundation and projecting above the ground up to the bottom of the bridge girders or less, according to the requirements of the design. The cylinders are filled with concrete after being sunk and masonry built at top where required. The cylinders may be connected or braced together laterally.

Bearings

The type of bearing to be used for bridges will depend upon the amount of movement of the bridge ends at the bearings due to temperature changes, and upon the load carried by the individual beams or slabs.

A bearing stone may be of hard stone or RC. It may be 30 cm for spans up to 4 metres and 40 cm for spans 5 to 6 metres over abutments for RC road slabs or girders. Depth of bearing stone is half the width and it need project about 25 mm from the face of the abutment. The forward edges of the bearing stones may be chamfered off for about 50 mm to avoid the possibility of spalling (not so shown in the sketch). Tarred paper should be used under RC slabs over the bearing cap for small spans, and two layers of 3 mm thick (each) lead sheets under heavy beams or slabs.

Sliding Plate Bearing permits slight sliding of a girder end under expansion or contraction over the bearing. ***Bearing plates*** are satisfactory where the pressure is not more than about 30 kg/sq. cm in the bearing surface, and for spans not exceeding 14 metres. The bottom bearing plate should rest on sheet lead plates (as explained above). The lead plates also serve to equalize bearing pressure on the base when the span deflects under live loads and thus prevent high edge pressure.

In the case of girder bridges, a sole plate is rigidly fixed underneath the girder ends which rest on another plate rigidly connected to the bed block or the cap, in which slotted holes are made for the free movement of the bolts connecting the girders with the abutment cap. One end is made fixed. The plates should be made of metal highly resistant to corrosion such as, phosphor-bronze. The underside of the top plate should be well coated with graphite. Where bearing pressures are high and bearing plates are used at fixed supports the lower support should be ground to a large radius.

For simply supported RC and pre-stressed concrete structures, the span up to which plate bearings can be used shall be limited to 15 metres. For spans larger than about 15 metres, deep cast bases are used for steel girders. Slotted holes are made in the cast base for the free movement of the sole plate which rests on the cast base.

For large spans, say over 18 metres, and for expansion bearings subject to heavy loads rocker and roller bearings are used. *Rocker Bearings permit* slight angular movement in the supported ends of a bridge superstructure. *Roller bearing* is a bearing assembly consisting mainly of rollers with suitably designed top and bottom plates, which permits of slight longitudinal movements in the supported ends of bridge superstructure. Solid rollers of 75 to 250 mm diameter are used.

7. SKEW BRIDGES

A bridge whose centre line is not at right angle to the stream flow, is called a skew bridge. As far as practicable bridge should be made at right angles to the axis of the channel; where absolutely essential the angle of crossing with the road should not be sharper than 60 deg. A skew bridge is difficult to construct, especially arches, and the structure has to resist additional forces due to water pressure and traction. The stresses in a skew slab are different from those in a straight slab as the planes of stresses are not parallel to the centre line of roadways and the difference increases with the angle of skew. The reactions at the supports change with the skew angle and the exact reactions are difficult to known. It has been shown, however, that the reaction of an abutment of a 60 deg. skew arch uniformly loaded varies from zero to twice the average pressure, which should be kept in mind while designing footings. (Also see under "Masonry Structures" — Section 7.) The depth of the foundations has also to be increased as they are likely to be scoured more. The following methods have been recommended for the design of bridge slabs and reaction at foundations:

(*i*) For skews up to 20 deg. use span along centre line of roadway; design slab as straight, and assume footing reactions at abtuse angle corners free ends to increase from 0 to 50 per cent above the average pressure according to amount of skew. (This differential in support reaction exists only at the ends of freely supported slabs; in continuous slabs the loads coming from adjacent spans to a large extent equalize the pier reactions).

(*ii*) For skews from 20 deg. to 50 deg. use span perpendicular to support; obtain thickness of slab and amount of steel as though the slab were straight, then multiply steel required by secant squared of the skew angle, if steel is placed parallel to the centre line of the roadway. Assume footing reaction at abtuse angle corner to increase from 50 per cent to 90 per cent above the average pressure at the freely supported ends.

(*iii*) For skews larger than 50 deg., a T—girder bridge should be used even though the spans are short.

When T—girders are used, the footing reactions at the abtuse angle corners are somewhat greater than for a straight bridge, but the increase is small compared with that of a slab, and is usually ignored.

The approach parapet towards the abtuse should make an angle of 60 deg. with the exis of the channel and towards the acute angle should make an angle = two-third of the acute angle of skew.

The centre line of the pier of a skew bridge should be parallel to the line of flow of water.

Skew arches shall be so constructed that the courses are everywhere at right angles to the lines of thrust.

8. APPROACHES TO BRIDGES

The approach on either side of a bridge should have a minimum straight length of 15 metres and for a culvert 6 metres, increased where necessary to provide for the minimum sight distance. (See under "Roads" for minimum Sight Distance and Stopping Distance.) The length of the straight reach may be reduced to 9 metres for bridges in difficult country taking due precautions for "speed limit." If approach alignment is a curve, a curve under 4-deg. is desirable, with 6-deg. max.

Minimum horizontal distance of the bridge approach at road level on either side (measured from the face of abutment) should be 7.6 metres (prefer 9 metres for arterial and district roads, which may be reduced to 4.6 metres for village roads. Slopes of approach should be 1 in 30 max. to 1 in 16 absolute max. (Prefer 1 in 50 for arterial and district roads and 1 in 30 for village roads.) Humps should be avoided

No borrow pits should be made or spoil heaps deposited within 9 metres of the toe of an embankment or the edge of the cutting of a bridge approach.

The top level of the approach roads must be high enough not to be overtopped by floods. If the velocity of the stream (during floods) is V/sec., the water surface level, where it strikes the road embankment, will be $\frac{V^2}{2g}$ higher than the H.F.L.

9. PARAPETS AND RAILINGS

Parapets or railings must be protected by wheel-guards or kerbs. Hand-rails may be of L-irons 1.5 to 2.5 metres apart with galvanized iron piping of diameter 32 to 50 mm. A 100 x 75 x 8 mm L-iron will suit for up to 1 m height post 2.5 m apart. A masonry parapet can be 30 cm (or $13\frac{1}{2}''$) thick.

Railings can be made of the following section:

Verticals:

L-65 x 65 x 10 mm, 1.5 m apart, curved to 1.4 m radius for the top 0.65 m where desired.

Horizontal top:

L-75 x 75 x 6 mm with two flat irons 40 x 12 mm.

Roadway and footway railings or parapets of high level bridges should have a minimum height above the adjacent safety kerb surface of 1.1 m less one-half of the horizontal width of the top rail, or the top of the parapet. Where a cycle track is provided on the bridge and is located immediately next to bridge railing or parapet, the height of the railing or parapet should be kept 15 cm higher.

The clear distance between the lower rail and the top of the kerb should not exceed 15 cm unless the space is filled by vertical or inclined members for safety of the users.

For culverts, the parapets should have a minimum height above the road surface of 30 cm and be at least 20 cm thick.

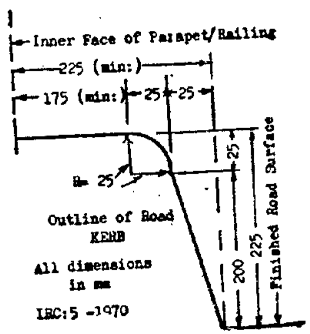
Parapets or other fencing should be provided on each approach of a bridge and on both edges of all banks over 2.5 metres high. *Guard-rails* should be provided where embankment is 1.8 metres high or more. *Fenders* consisting of stout posts (of stone or cement concrete) or rails well embeded in the ground should be provided at both ends of parapets; Size—0.65 m long, about 40 cm above ground.

On hill roads parapets may be made of dry stone with top course in mortar, about 60 cm high in small lengths, with gaps in-between.

Hand-rails, parapets or posts should be designed to resist a lateral horizontal force and a vertical force each of 150 kg/lineal metre applied simultaneously at the top of the railing or parapet.

10. KERBS OR WHEEL-GUARDS

Roadway kerbs should have a solid section not less than 225 mm wide at the bottom tapering towards the top with rounded edge, and not less than 225 mm high above the finished road surface. A safety kerb will have the same outline as that of a roadway kerb except that the top width shall not be less than 60 cm. Safety kerbs are designed as footpaths.



11. WIDTH OF ROADWAYS ON BRIDGES

For small bridges the width of roadway shall not be less than 4.5 metres for a single lane bridge and 7.5 metres for a two-lane bridge, between inner faces of the kerbs (clear carriageway). The width of a central verge, where provided shall not be less than 1.2 metres.

Formation width for village road bridges is 6 metres, and for arterial and district roads 9.75 metres.

12. FOOTWAYS ON BRIDGES & PEDESTRIAN OR FOOT-BRIDGES

Minimum width for footways or footpaths is 1.22 metres (IRC recommend 1.52 metres), but prefer 1 83 metres (6 ft.) where practicable.

Live loads for design of horizontal members may be taken at 400 kg/sq. m up to 7.62 metres span, and 300 kg/sq. m for spans of 30 metres, intensity of load reducing uniformly, for all footways on bridges, pedestrian bridges and bridges used by animals. Where crowd loads are likely to occur on the bridge, the live loads shall be taken at 600 kg/sq. m. Dead load on timber planks and decking is about 500 kg/sq. m on girder bridges.

Minimum Vertical Clearance on a bridge should be 5 metres above crown of the road surface. For footways and cycle tracks the vertical clearance is 2.25 metres.

13. I.R.C. STANDARD LOADINGS

Class "AA": and Class 70 R: Heavy loading — Adopted only in certain industrial areas and certain specified highways.

Class "A" Standard loading to be adopted in general for all permanent structures.

Class "B": Light loading to be adopted for temporary structures or timber structures. Class "B" loading is about 60 per cent of Class "A" loading. This loading conforms with the design of a road bridge to take a moving load of a 10-tonne road roller passing one at a time over the bridge.

14. RCC SLABS FOR ROAD BRIDGES

British empirical formula for Weight of Concrete Bridges

(*i*) Solid slabs: $W = 100 + 9L$ } in lbs./sq. ft. of floor
(*ii*) T-beams $W = 100 + 5L$ } area of slab.

L is clear span in feet.

(Includes weight of wearing surface at 40 lbs./sq. ft.)

Joint in RCC Road Slabs:

(Joints have been dealt with in detail in Section 8.)

Where an expansion joint is provided there should be a free joint through every part of the bridge deck at that point. Where wearing surfaces have to be carried continuously over joints in bridge

(carried over to page 19/36)

REINFORCEMENT DETAILS FOR R.C.C. SLABS FOR CULVERTS

Freely Supported with 75 mm thick Asphaltic Concrete Wearing Coat

Clear span	Bearing length at each end	Overall length of slab	Overall slab thickness	Reinforcement steel								Total quantity of steel per span	Total quantity of concrete per span
				Longitudinal bars				Transverse bars					
				Bottom bars		Top bars		Bottom bars		Top bars			
				Dia.	Spacing	Dia.	Spacing	Dia.	Spacing	Dia.	Spacing		
m	mm	m	mm	mm	mm	mm	mm	mm	mm	mm	mm	tonnes	cu. metres
1.0	300	1.6	190—150	16	150—115	10	300	16	150—125	10	300	0.582 0.662	3.65 2.88
1.5	300	2.1	220—180	16	105—90	10	300	16	150—100	10	300	0.848 1.054	5.55 4.54
2.0	300	2.6	240—215	16	100—80	10	300	16	125—100	10	300	1.117 1.268	7.48 6.71
2.5	300	3.1	270—245	16	90—75	10	300	16	125—150	10	300	1.500 1.634	10.40 9.30
3.0	300	3.6	305—275	16	75—70	10	300	16	125—150	10	300	1.861 2.000	13.20 11.88
3.5	300	4.1	340—315	16	60—65	10	300	16	125—150	10	300	2.184 2.278	16.80 15.47
4.0	300	4.6	370—345	20	100—90	10	300	16	125—150	10	300	2.506 2.506	20.41 19.05
5.0	400	5.8	420—395	20	90—75	10	300	16	125—150	10	300	3.402 3.703	29.26 27.50
6.0	400	6.8	475—445	20	75—68	10	300	16	125—150	10	300	4.513 4.581	38.80 36.30
			*		*				*			*	*

*Where M 200 (1 : 1½ : 3) Controlled Concrete is used.

Based on IRC "Guidelines for Design of Small Bridges & Culverts".

Notes for RCC Slabs for Culverts

(i) Design Loading : IRC Class 'A' (two-lanes) or Class 70 R (one-lane)

(ii) Roadway is 11.20 metres, 400 mm parapet wall provided on top of slab at each end, thus making total width of slab 12.00 metres. Height of parapet wall is 500 mm above the slab.

(iii) Ordinary cement concrete 1 : 2 : 4 may be used. Where M 200 (1 : 1½ : 3) controlled concrete is proposed to be used, figures are marked with* for (a) Overall slab thickness. (b) Spacings of bars, (c) Total quantity of steel, and (d) Total quantity of concrete, where shown, should be taken.

(iv) Reinforcement—Mild steel grade I tested steel.

(v) Every alternate bar of longitudinal bottom reinforcement should be bent up (cranked) at 45 deg. at a distance of 0.15 span from the edge of each abutment as shown in the illustrations given in Section 8 "Reinforced Concrete". (Some engineers recommend that one longitudinal reinforcement bar should be bent up in every four up to 3 m span and in every two for spans above 3 m.) All bars are hooked as usual.

(vi) If full length bars are not available and joints have to be provided, lap joints for 45 diameters length provided no joint comes at mid-span and all joints are well staggered.

(vii) Min : clear cover to reinforcement bars shall be 25 mm or dia. of bar whichever is greater.

(viii) Total quantity of steel given is inclusive of 5 per cent for laps and wastage.

(ix) Transverse reinforcement given in the table may be reduced towards the ends.

(x) Rules given in Section 8—"Reinforced Concrete" under "Design of Beams and Slabs" should be followed in general.

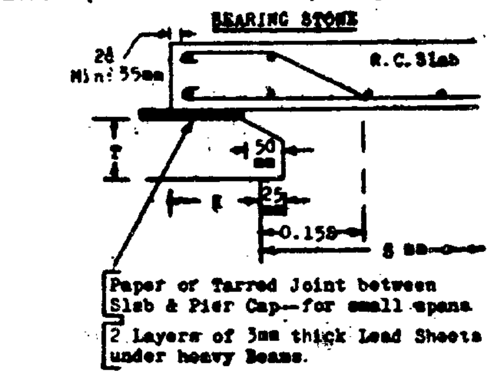
(xi) For spans over 3 metres a longitudinal camber of span/240 may be given in the slab.

(xii) It is generally economical to use single slabs instead of T-beams for spans up to 10 metres.

(xiii) 75 mm thick asphaltic concrete (see under Roads) is provided over the concrete slab.

(xiv) Abutment or pier caps are made of R C C (or stone slabs), 150 mm thick and of length 300 or 400 mm as given in the table, reinforced with 2 per cent steel by volume, distributed equally at top and bottom in both directions. Tar paper is placed on top of the bearing stone (under the slab).

(xv) 6 mm space is left between the end of slab and abutment for



K = Length of Bearing = 150mm + 0.04S mm or 360mm whichever is less

T = Depth of Bearing Slab or Pier Cap = 75mm + 0.02S mm expansion of the slab, and this space is filled with bitumen impregnated felt.

(xvi) The above table may be used for culverts on all highways. The heaviest loading may come on the bridge only once in a while and which can be taken care of by the factor of safety.

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decks, some form of continuous support over the joint is required. This is best provided by a steel plate secured to the deck at one side of the joint and free on the other or, if the movement is likely to be small, by a standard T section which is dropped into the expansion gap, without attachment to the structure. (This has been shown in the illustration in Section 8).

The width of a joint should be four times that actually required by theoretical calculations and the joints should be filled with bitumen and sand and should have a copper waterstop. The road slabs should have free longitudinal movements over abutments or piers and over sliding plates as explained before. Adequate joints should be provided between deck and parapets to take care of transverse expansions. Kerbs of slab bridges should be cast integral with the deck slab, but kerb and slab should have a joint on continuous T girder bridge. Joints in handrails are necessary to secure discontinuity of rails so that they will not act with the deck slab. Contraction joints should be placed in members with little reinforcement in the direction of restraint; for example, a longitudinal joint should be on the centre line of construction. Joint between the approach slab and the deck slab is essential, as shown in the illustration.

A gap of 25 to 50 mm, according to the length of the slab and temperature ranges, will be found adequate between two slab ends resting on one pier. Expansion joints should be placed at free ends of all continuous units to provide for longitudinal expansion.

Drainage outlet holes (spouts) 50 to 75 mm dia. should be provided on both sides of the road through the parapets at intervals of say, 2.5 to 3 metres, the road surface having been given adequate camber and longitudinal falls.

Road Surfacing over Concrete Slabs

(*a*) Paint the concrete surface with a coat of asphalt and cover it with 40 mm of earth as cushion over which 120 to 150 mm of metalling can be given duly surfaced.

(*b*) Asphaltic concrete about 50 mm thick as per specifications given under "Roads". A small amount of cement if added to the mix as filler will improve it considerably. Ramming is done with wooden rammers or light roller.

(*c*) Wearing surface of 75 mm cement concrete can be given as a second coat on the bridge slab as explained under "Cement Concrete Roads."

Strengthening Existing RC Slab Bridges

The strength of a 75 mm slab can be increased by adding more con-

crete on its top and bonding it with the old slab so that the two slabs act in conjunction.

Clear Span metres	Size of RS Beam mm x mm x kg/m
4 6	400 x 165 x 56.9
4.9—5.2	450 x 170 x 65.3
5.5—5.8	500 x 180 x 75.0
6.1—7.0	500 x 250 x 95.0
7 3—7.9	600 x 210 x 99.5
8.2—9.1	600 x 250 x 133.7

Rolled Steel Beams for Bridges with RCC Decking for IRC Class "A" Loading

(a) RS beams are 1.7 metres centre to centre.

(b) Details of RCC Slab Decking:

Slab thickness 19 cm with 7.5 cm wearing course. Slab reinforced with 12 mm dia. bars at 10 cm c/c at right angles to the direction of the beams.

Adequate reinforcement shall be provided at the top of the slab over the beams for positive bending moment.

Alternate bars bent up. Longitudinal bars (temp.) at bottom of slab 12 mm dia. at 20 cm c/c and at top 12 mm dia. at 30 cm c/c shall be provided.

(c) The decking slab will project 15 cm plus width of parapet on both sides of the end beams to make up for a 3.66 metres traffic lane. The slab may project up to 75 cm on the end beams to make up for greater road widths.

(d) Mild steel (rustless) bed plates 10 mm thick, or two layers of 3 mm thick lead sheets, shall be provided under the beams.

(e) Provision must be made for expansion of girders at the rate of 25 mm per 30 metres of span.

(f) 20 mm dia. anchor bolts in slotted holes shall be provided on each girder end—two bolts on each end.

(g) Bottom of the RCC slab either rests on the top of the beams or is made level with the bottom side of the top flange and the negative bending moment bars rest on the beams.

(h) 65 x 65 x 6 mm L spacer is fixed at the centre at right angles to the girders, in spans above 6 metres.

(i) Drainage holes shall be provided on both sides of the road at about 2.5 metre intervals.

Road slabs which have not been designed for live loads directly on to their tops should have an earth cushion of at least 1 metre under class "A" loading and 0.6 metre under class "B" loading.

15. MASONRY ARCH BRIDGES

(Also see "Masonry Arch Design" in Section 7)

Masonry and brick arches, with a good rise and ring thickness, good foundations, and proper maintenance, are capable of carrying very heavy loads. (Exact strength calculations of an arch are very intricate, therefore use of arches are being replaced with cement concrete slabs.) A good deal of strengh is derived from the filling where this has become well consolidated after many years. There is every reason therefore, to recommend the construction of arches for spans up to 9 metres, especially where brick or stone is readily available. The joints in brick and masonry arches permit some adjustments to changes of temperature without causing undue internal stresses, and shrinkage stresses are not set up; such types can even withstand minor abutment movements without severe distress. In all arch bridges rigidity of abutments, piers and foundations is essential and foundations must be absolutely unyielding and there should be no possibility of any settlement.

Masonry arch bridges are not recommended in seismic zones for spans exceeding 6 metres.

Rise/Span Ratio

Every endeavour should be made to keep the rise/span ratio as large as possible. The rise of the arch should be between 1/2 and 1/4 of the span as semi-circular arches exert no thrust on the abutments or piers and elliptical arches exert very little thrust. Site conditions may limit the choice of the shape of the arch. Skew arches should be so constructed that the courses are everywhere at right angles to the line of thrust.

Thickness of Arch Rings: (General Formula)

For segmental arches t should not be less than $0.45\sqrt{R}$.

For culverts under high fills, increase t by 50%.

Thickness of Abutments:

$$E = \frac{R}{5} + \frac{r}{10} + 0.6 \text{ m for h less than } 1\tfrac{1}{2}L$$

For greater *h*. add *h*/5, to the E obtained from the equation:

$$b = E + \frac{E.Z}{24r} \qquad R = \frac{a^2 + r^2}{2r}$$

Abutment back batter = 1 in $\frac{24 \times r}{\text{span}}$; front may be vertical or 1 in 12 to 1 in 24.

L should not be less than 2/3 h; F may be G/2.

(See illustration on the following page.)

R = radius of the arch intrados = $(a^2 + r^2)/2r$; r = rise of the arch; E = thickness of abutment at springing, b = thickness of abutment at any depth "Z" below springing, t = thickness of arch ring at crown, S = span.

The abutments should be back filled up to the springing before striking the centerings.

Thickness of Piers

The thickness of a pier should be sufficient to resist the thrust resulting from one of the two arches it supports when it is covered with the design live load while the other remains unloaded. The thickness at

the the top must be adequate to accommadate the skew backs on both sides.

Short piers may have vertical sides or better of 1 in 24 to 1 in 30 and long piers, a batter of 1 in 12.

K should not be less than h/3+23 cm and at least 30 cm more than P J may be 30 cm to 75 cm.

Every 4th or 5th pier should be an *abutment pier* of the same top thickness as the abutments with batters of 1 in 12 to 1 in 24.

Spandrel Walls are the walls built on the top of the arch rings up to the level of the roadway.

Sprandrel is the space extending from the top of a masonry arch to the top of the roadway.

$D = \frac{r + t}{2}$, D is depth of haunch filling at pier and abutment. C is earth cushion.

Working stresses for Masonry Structures have been given in Section 7. "Design of Masonry Structures".

For stone masonry work each stone should be carefully chisel-dressed to the required wedge-shape so that all joints are truly radial, also bed stones or skew backs. Joints should be not thicker than 3 mm.

The base widths of the abutments and piers depend on the bearing capacity of the soil. The pressure at the toe of the abutment should be worked out to ensure that the soil is not overstressed.

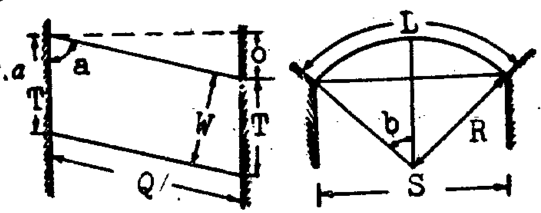
In scourable beds, the length of the pier should be kept minimum, and preferably made circular in the case of skew bridges.

Conditions of Stability of Abutments

An arch abutment should be investigated for the following three conditions of loading — (i) Dead load plus live load on 5/8 span adjacent to the abutment. (ii) Dead load plus live load on the other 5/8 span. (iii) Dead load plus live load on the entire span. For each of the above conditions of loading, the line of pressure in the course enumerated above shall lie within the middle-half of every section of the abutment. The line of pressure shall lie within the middle-third of the foundation and every effort should be made to keep it as near the centre of the base as possible.

Skew Arches

o = obliquity of arch-S cot.*a*
a = angle of skew,
b = half angle of arch,
R = radius of arch,
S = span on the square,
W = width on the square,
Q = span on the skew
$= S \text{ cosec.} a = \frac{T}{W} \times S$



T = length of impost = W cosec. *a*
L = length of arc = .0349Rb.
For semi-circular arch :
L = 1.571S = 90 × R × .0349.

DATA FOR MASONRY ARCHES FOR

I.R.C. Class "A" Loading two-lanes or

Clear span in metres		1.00	1.25	1.50	2.00	2.50
Arch thickners	t—cm	25	30	30	35	35
	t—ins.	9"	1'-1½"	1'-1½"	1'-1½"	1'-1½"
Earth cushion	C—cm	45	45	45	50	60
Haunch filling	D - cm	25	30	35	45	50
Abutment top	E_1—cm	65	75	85	90	100
	E_2—cm	70	80	90	95	110
Pier top	P_1—cm	40	50	55	60	70
	P_2—cm	50	60	70	70	80
Foundation depth	G_1—cm	30	30	40	40	45
	G_2—cm	45	45	55	55	60

E_1—abutment top—Ist class stone masonry in cement 1:3

E_2— „ — „ brick masonry „

P_1—pier top—Ist class stone masonry in cement mortar 1:3

P_2— „ „ brick masonry „

G_1—depth of foundation—cement concrete 1:3:6

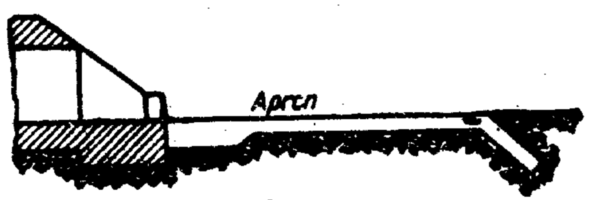
G_2— „ — „ 1:4:8 or good lime concrete

The tops of all abutments and piers should be in cement mortar 1:4 for a depth of at least 600 mm.

The masonry of the arch ring may consist of either concrete blocks (1:2½:5 or 1:3:6) or dressed stones or bricks in 1:3 cement mortar.

The crushing strength of concrete, stone or brick units shall not be less than 106 kg/sq. cm. Where stone masonry is adopted for the arch ring, it shall be either coursed rubble masonry or ashlar masonry (IRC-13).

The ratio of rise to span of the central line of the arch ring shall be 1/4.

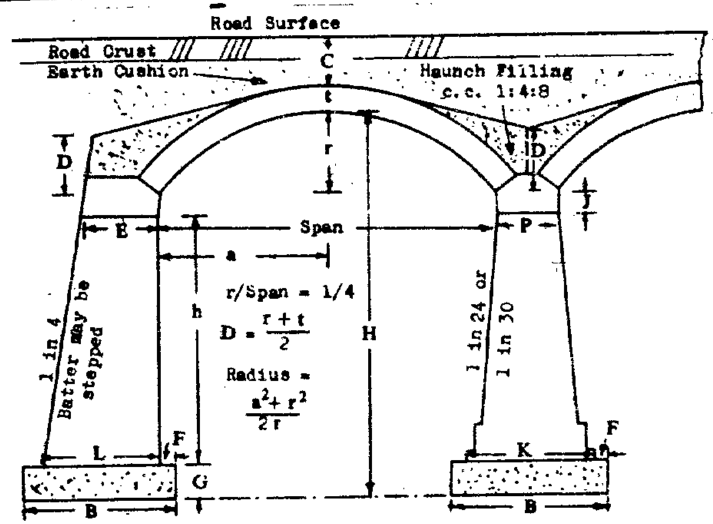


Arrangement of apron downstream of structure

CULVERTS & SMALL BRIDGES

class 70-R Loading one-lane

3.00	4.50	5.00	6.00	7.50	9.00	10.5	12.0
35	40	45	55	60	65	70	80
1'-1½"	1'-6"	1'-6"	1'-10½"	1'-10½"	2'-3"	2'-3"	2'-7½"
60	60	60	60	70	75	75	75
60	70	90	100	120	145	160	190
110	120	130	155	175	200	220	245
115	140	140	160	190	210	230	250
75	80	85	100	110	130	150	175
80	90	90	110	130	140	160	180
45	55	55	60	75	75	75	75
60	75	80	85	90	100	110	115



Fillings over Arches and behind Abutments

Filling of a porous nature such as brickbats or ballast, should be used to cover the whole extrados of the arch to a depth of at least 30 cm over the crown. Similar porous backfilling should be done at the back of the abutments.

Haunches (D) may be filled with cement concrete 1 : 4 : 8.

To safeguard against water penetrating and leaching. the entire extrados of the arch ring and the lower 15 cm of the spandrel wall should be covered with bitumen. The material used for filling over arches should be drained by providing drain pipes set near the springings to lead away all water running down the water-proofing of the arch extrados. Drain pipes or weepholes should also be provided in all abutments to drain out water from the porous backfills and which should be 15 cm above normal water level.

Striking Centres of Arches—The centres should not be struck before one week after the completion of the arch.

Strengthening Existing Masonry Arches—Existing arches of small spans can be strengthened by additional rings at the bottom. Additional ring at the top of the existing rings do not add to the strength of the existing ring.

16. SUBMERSIBLE BRIDGES AND CAUSEWAYS

Submersible Bridges or High Level Causeways

Selection of site for a submersible bridge requires the following considerations: (*i*) it should have a small width; (*ii*) well defined and high banks; (*iii*) and should have a straight reach for a considerable length.

Submersible bridges or causeways shall provide for at least two lanes of traffic (6.7 metres clear) between kerbing arrangements of posts fixed to body or walls.

Design:

(*i*) A submersible bridge should have deep foundations much below scour depth. Openings may be of arches, slabs or pipes.

(*ii*) The structure of the bridges should be heavy and massive, and should be safe against over-turning or uplift under the critical conditions *i.e.*, when the flood water is just about to overtop.

(*iii*) Section should be such as to have least area of obstruction to the flow of water; should have minimum number of piers and small thickness of the decking, with no parapets.

(*iv*) All filling should be such that would stand submergence.

(*v*) Headroom should be so fixed that the bridge would not be closed to traffic for a longer period than the traffic can afford. Spans will also be fixed according to the requirements of the traffic, height of bridge and the flow and duration of the storm water.

(*vi*) Railings shall be either collapsible or removable.

In addition to the forces acting on a bridge structure as detailed under "Design of Piers" a submersible bridge will have:

(*i*) Uplift pressure $= W \times h$—(horizontal and vertical force in kg/sq. m), where W is weight of water per cu. m and h is the uplift head under the decking, which is equal to the thickness of road slab including wearing coat and afflux (assumed) less the head lost due to increase in velocity through the bridge openings. The head lost due to increase in velocity $= (V_1^2 - V_2^2)/2g$.

V_1 = velocity under the openings,

V_2 = original velocity before approaching the bridge,

g = acceleration due to gravity = 9.8 m/sec./sec.

(*ii*) Pressure due to eddies:

$= W(Vv - V)^2/2g$ (horizontal force in kg/sq. m)

Vv = velocity through the openings,

V = velocity of approach.

(*iii*) Friction of water on surface in contact with water. All the horizontal forces are added together to act on the pier face (at $^1/_3$rd height). These forces are generally small compared to the bridge structure.

A detailed design for a submersible bridge has been worked out in IRC Paper No. 173.

Foundations

On beds of sand, loam or clay, a monolithic base construction or a cement concrete raft which runs continuously over the entire length, with sloped aprons and *cut-off walls or dwarf walls* on both the upstream and downstream sides are provided to guard against scour and undermining. Boulder pitching encased in wire netting is provided in the bed on both sides away from cut-off walls.

The abutments are made solid and of massive construction; the approaches which should generally follow the bank slope are also paved solid extending 45 cm above highest flood level and made in the form of scuppers with upstream and downstream small dwarf walls and pitching where required. Wheel guards are usually provided on the bridge instead of railings.

Causeways or Irish Bridges

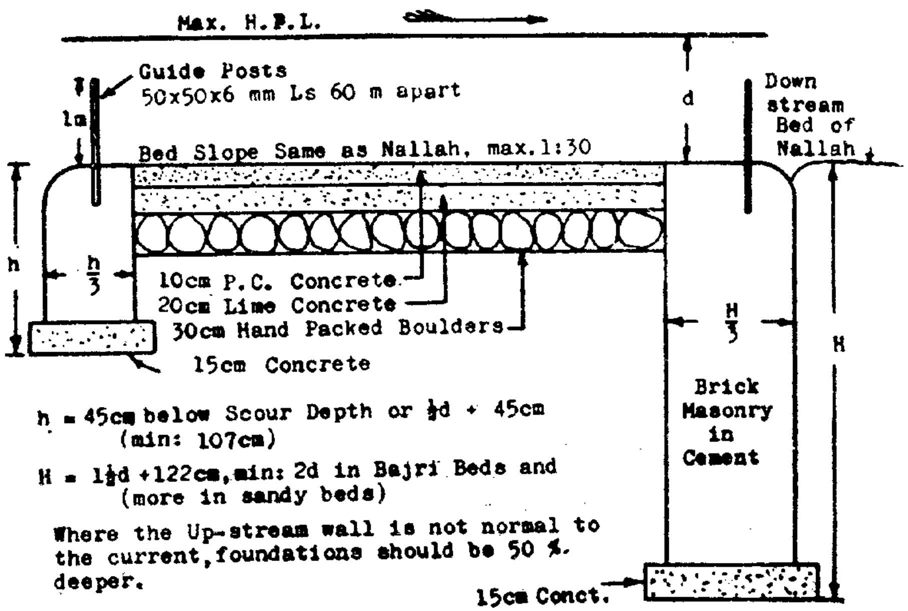
A causeway is a small submersible bridge without openings, or a paved dip in a road across a shallow drainage course (inundation stream) at or about the bed level, which will allow floods to pass over it. A *flush causeway* is a causeway at bed level of a stream.

A causeway must not contract the stream, and maintain a firm roadway against damage by floods, erosion or movement of the stream bed and it should be made at right angles to the water flow to avoid scouring. The embankment if liable to erosion by floods must be protected by dry stone walling or pitching carried least 90 cm below the stream bed level and 45 cm above highest flood level.

Scuppers

A scupper is a miniature form of causeway and extends across the entire formation width. A scupper should be laid down in three curves in the direction of the road alignment, convex at the ends and concave in the middle with the requisite cross slope of 1 in 12 in hill sections to 1 in 30 in flats. They are often preferable to small culverts in hill sections but are unsuitable for use on steep gradients. A scupper is generally up to 60 cm span.

CAUSEWAY



Guide posts may be used in lieu of railing, if the submergence of the road surface is very frequent.

The depths should be increased in sandy beds.

Causeways may be built with deep upstream and downstream walls with no protection works or with shallow walls protected by boulder filled wire mattresses, or by a concrete slab (lightly reinforced at top and bottom) apron, below the downstream wall.

17. SUSPENSION BRIDGES

The simplest form of a suspension bridge comprises two sets of cables (may be one or more cables side by side) one on either side from which a deck is suspended by rope slings, wires or chains, spaced about 1.5 metres apart. The roadway consists of wooden planks placed transversely which are connected at ends by longitudinal wooden transoms which are supported by slings or suspenders. The whole stability of a suspension bridge is dependent upon anchorage of the cables at the ends. The cables are passed over towers and then anchored into the solid rock, if avilable, or into massive concrete blocks built underground in the banks at such a distance from the towers that the cables will make equal angles with the vertical on each side of the towers. When live load is applied the towers are deflected towards the centre of the bridge and are therefore strutted or guyed behind to prevent overturning. If cables are passed over rollers fixed on the tops of the towers, the top deflection will be minimized, or

alternatively the towers can be pinned at the bases so that they can deflect without bending. The dip of the cables at the centre of the span is about 1/10th to 1/12th of the span and the towers are 1 to 1.5 metres longer than the width of the bridge and may be of solid masonry or braced steel columns. Good deep foundations are provided for the towers. Advantage is taken of the high tensile strength steel wires which are formed into a rope. Hard drawn steel wires are available of strength of about 15 tonnes/sq. cm. For decking, stiffening girders are used as otherwise it is flexible under heavy moving loads. A temporary foot bridge is first made to help the erection af the main bridge. Stiffened type of suspension bridge should be preferred. Braced type of stiffering is better than truss type stiffening.

$H=\frac{wL^2}{8d}$; $T_1=\frac{H}{\cos\varnothing}$

$T=\sqrt{H + \frac{w^2L^2}{4}}$

Pressure on pier, per cable

$=\frac{wL}{2}-T_1 \sin\varnothing$,

Length of cable

$=L+\frac{8d^2}{3L}$

H = horizontal component of tension in cable,
w = load on cable per horizontal m on span,
d = dip of cable at centre of span,
T = max. tension in cable,
T_1 = tension in anchor cable,
Ø = angle of inclination of anchor cable to horizontal.

18. GIRDER BRIDGES

Plate Girders

Plate girders are suitable for spans from 12 metres to about 18 to 21 metres and trussed girders for larger spans. The depth of plate girders varies from 1/10 to 1/15 of the span: 1/12 is the most economical proportion. The width of the flange under compression should not be less than 1/30 to 1/40 of the span and should not be less than 1/18th of the distance between effective stiffeners if the edges are unstiffened and 1/24th of these distances if the edges are stiffened, or it will be liable to buckle sideways. The ratio of depth of web to its thickness shall not exceed 85, where it must exceed, the allowable shear stress has to be decreased. No plate less than 6 mm in thickness should be used. Girders should be connected by the web not the flanges, and should be built with a camber of about 1/480 of the clear span. For preventing, overturning, the width centre to centre of the main girders, should be not less than 1/15th of the effective span. Where two or more joists or channels are used side by side to form a girder, they shall be connected together at intervals of not more than 1.5 metres. Panel length should not exceed 1½ times its width.

Girders of over 15 metres span should have cast iron shoes upon the ends of expansion. Large girders should have one end supported on rollers working in a roller box. Working stresses for steel structures are given in Section 10. Shear in webs of plate girders is taken at 900 kg/sq. cm.

Approximate British formula for weight of a plate girder = WL/(900 − L) where:

W = total load on the girder in tons; L = span in ft.

19. TEMPORARY AND WOODEN BRIDGES

Wooden trestles are made in the channel bed which is of a firm material and where the velocity of the stream is not very high. The trestle bents are made of vertical posts or legs with their feet spread out at a slope of about 1:6. A transom is fixed horizontally across the legs near their tops and a ledger is fixed across their feet and diagonal braces are provided between them. The trestle bents may be two legged, three-legged or four-legged according to the load, span and the height. Longitudinal girders are supported over the horizontal transoms. Cribs are made instead of trestle bents in swift and shallow streams. (See under "Piers") Trestles can be erected at site in shallow waters or dry beds but for deep waters they are assembled on the bank and suspended at the site with their feet weighed. The wooden pieces are spiked together or tied with steel wires. Decking can be made of timber planks 5 cm thick for ordinary traffic and 8 to 10 cm thick for heavy and much wheeled traffic. 5 cm thick planks supported on road bearers or stringers at 60 cm intervals can carry axle load not exceeding 1½ tonnes.

20. CULVERTS

A culvert is a (small) bridge structure of less than metres span between faces of abutments and does not generally has two spans.

A culvert must be large enough to carry the flow without any heading up at the entrance. To provide for this, culverts should be assumed as flowing only half full when the approach channel is wide and shallow; if the banks are steep and the channel narrow 3/4th full may be taken. For arched culverts the top of the culvert for calculation purposes should be assumed as lying half-way between the springing and the crown. But on the other hand it is not economical to make a culvert unnecessarily high with extra approach embankments necessitating high abutments, headwalls and wing walls, for retaining deep over-fills. The depth of a culvert should be small, and it does not matter much if the opening stops appreciably below the formation level of the road and the inlet is sometimes submerged; instead the length should be increased suitably so that the road embankment, with its natural side slopes, is accommodated without high retaining headwalls. The heading up of the water at the inlet should not go higher up than predetermined safe level, nor overtop the road embankment. The fixing of this level is the first step in the design. (An opening running full gives less discharge than when running partially full.)

Where masonry abutments supporting arches or slabs are designed for culverts functioning under "head", bed pavements must be provided. And, in all cases, including pipe and box culverts, adequate provision must be made at the exit against erosion by providing curtain walls.

To get the best advantage of the capacity of a culvert the shape of the entrance should be such as to cause the least amount of restriction to the free flow of the water. This can be achieved in the case of plane face walled culverts by means of pitched aprons at both ends and pitched trained banks with outward curved chords which make an angle of 70 deg. with the face wall. Slight chamfering or bell-mouthing at the inlet ends of pipes or barrels will increase their capacity of discharge. (See under "Hydraulics"). Further increase in the rate of flow is obtained in pipe culverts by fixing the invert somewhat below the natural bed level of the stream.

Sometimes a road embankment is made across a flat country without any defined drainage channels which intercepts the natural flow of rain water which ponds up on one side of the road embankment. A simple method to remedy this is to provide *dips* or small *causeways* in the longitudinal profile of the road and let water pass over them. Cement concrete slabs for small dips will be very suitable. In wet cultivated or water-logged country, or where the embankment has to be taken high above the ground surface, dips or small causeways will not do a satisfactory job, shallow culverts will have to be built. Pipe culverts, or pipe barrels embedded in cement concrete for dips, functioning with the inlets submerged, can be provided.

For large culverts where considerable flows at high velocities may occur, a low bund up to about 15 metres down stream will usually prevent scouring round the foundations. In hilly reaches on nallahs where detritus, boulders, and much of sand or silt is brought down with the stream, it is usual to make a catchpit in front of the culverts to facilitate cleaning of such detritus. The width of this catchpit is the same as that of the culvert and the floor level deeper than the sill of the culvert by about 30 to 90 cm.

In the case of culverts or causeways on black soils or soft strata, a complete raft of concrete 1.0 to 1.8 metres thick should be laid below the neat work for the full width and length. Provide a curtain wall also at the downstream edge of the wing walls.

The arrangement of head walls and wing walls must be such that the embankment is protected and the flow of water facilitated. Wing walls may be parallel with or at right angles to the axis of the culvert, or or may be placed at an angle with the head wall, usually 30 deg. or 45deg. For hydraulic reasons, especially at the upstream ends, flared wings are best and the culvert is less likely to become choked than when either of the other types is used. Also see under "Wing Walls & Return Walls."

It is not generally convenient to make a culvert of a size smaller than 90 cm × 90 cm or 90 cm dia. and it is then called a *Ventway*. No opening should be less than 120 cm wide × 60 cm deep. To facilitate inspection and carrying out of repairs the min: vent dimensions shall not be less than 75 cm

In *pipe culverts* or other small culverts which should extend across the whole formation width, wing walls can be omitted, especially where there is good amount of earth filling over the culvert and a straight wall (continuation of the head-wall) is provided. The length of such a wall should be little more than sufficient to keep the earth of the embankment spilling around its end and from reaching the opening. There should be sufficient earth cushion over the top as explained under

'Abutments' with a minimum of 90 cm over a pipe culvert. The length of a culvert (equal to formation width) will be width of the road plus three times the fill over the top of the culvert for a 1½:1 slope of the soil.

Where a defined channel does not exist and the natural velocity of flow is very low, it is economical to design a culvert as consisting of a pipe or a number of pipes of circular or rectangular section (box) functioning with the inlet submerged.

Pipe Culverts

For small drainage crossings pipe culverts are often found in practice to be the most economical and easily constructed These culverts can be easily enlarged subsequently to take more discharge by the addition of one or more pipes. The pipes may be of corrugated galvanized iron sheets, earthenware, or cement concrete. Old tar drums are also sometimes used. Cement concrete pipes are most commonly used, reinforced or unreinforced. Reinforced concrete pipes are more economical for sizes above 45 cm and under heavy loads. The discharge through a circular opening is much more than through a rectangular opening of the same cross-sectional area, especially when running full; circular openings give about 25 to 30 per cent more discharge (See under "Hydraulics"). It is more economical to provide less number of vents of large diameter pipe culverts than more number of vents of small diameter for a particular discharge.

Bedding for Pipes

Pipe culverts should be laid on a firm bedding. (Bedding of pipes has been described in detail in the Sections on "Drainage and Sewerage" and "Water Supply"). If the soil furnishes a poor support, the pipes should be bedded in a layer of concrete; in the case of causeways, all pipes are embedded in concrete. Solid foundations may not be provided in good soils as some amount of settlement as a whole in the pipes can be tolerated.

Concrete Pipes (Manufacture)

Concrete pipes are now being manufactured by several firms, the main features of these pipes is the dense cement mortar obtained by the special spinning process employed. These pipes can be easily manufactured locally at the site of the work which can be made of the same strength if vibrators are available. Small stone aggregate or bajri with 1 : 1½ : 3 mix will make a satisfactory job. The moulds may consist of two concentric hollow cylinders enclosing between them an annular space according to the thickness of the pipe. The outer form may consist of two half cylinders which can be bolted together to form a full cylinder. As it would be defficult to consolidate concrete if the height to be poured at a time exceeds 75 to 90 cm, the form should be of such lengths that two forms jointed together make one pipe length. The arrangement for the inner cylinder has to be more elaborate as it is to be contracted to a smaller diameter to enable it to be withdrawn without damaging the partially set concrete. The inner cylinder should be of the full length of the pipe to be cast. (For fuller details, see IRC Journal vol. XVI-I, page 131 and IS:458). The pipes should be made in lengths of 1.22 to 1.83 metres. Pipe culverts are not generally made

more than 1.83 m in diameter. Tubes normally have 'ogee' joints and pipes have spigot and socket joints. (Also see under "Water Supply".)

Bending Moments in Circular Culvert Rings

A pipe is treated as an arch for calculating stresses. The vertical load on half the section of the pipe (treated as an arch) is taken as uniformly distributed and filling over haunches is neglected. The horizontal pressure is also ignored.

(*i*) $M = \frac{1}{16} p d^2$

M = bending movement at crown and at centre of base for a unit length of culvert; p = load per unit area; d = diameter of culvert.

(*ii*) $M = \frac{1}{4} r^2 (W - P)$

M = bending movement at the top, bottom and two sides of the pipe; r = mean radius of pipe; W = vertical pressure on the pipe per unit of area on the horizontal projection; p = lateral earth pressure per unit of area on the vertical projection.

(There is not much of difference in the above two formulae; (*i*) erring on the safe side.)

Reinforcement

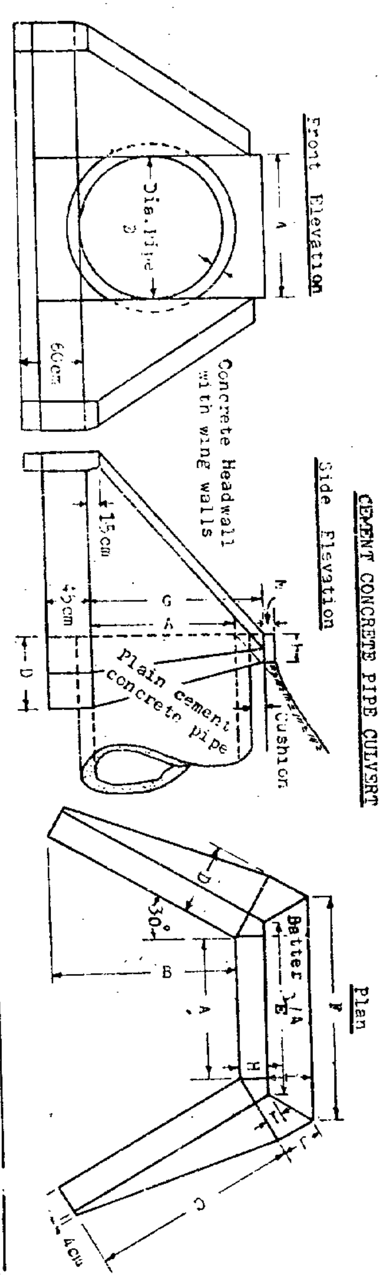
Reinforcement in pipes consists of hoops and longitudinal bars. Hoop reinforcement is provided to take up the full bending moment. Longitudinal reinforcement is about 1/4th to 1/5th of the helical reinforcement. 6 mm dia. bars at 23 cm centre to centre usually suffice.

It should be noted that at some locations the culvert pipes will be subject to longitudinal bending in which case sufficient longitudinal reinforcement should be provided in addition to the helical reinforcement to withstand this beam action; 2/3rd of the helical reinforcement is recommended for the longitudinal reinforcement.

Hoops are provided either: (*i*) concentric in one layer in the centre for pipes up to 60 cm dia., or (*ii*) in elliptical form for pipes 60 to 180 cm dia., as shown in the illustration, or (*iii*) in two concentric layers for large diameter pipes under heavy external loads or inernal pressure. The inner cage is placed close to the inside surface of the pipe while a lighter cage is placed near the outside surface. If the pipes are to be manufactured locally, it is preferable and easier to fix double layer of reinforcements in heavy pipes. The reinforcing bars should be staggered and a covering of 12 mm given on both the faces. The reinforcement should be arranged in the elliptical form as far as possible. Hoops should be welded to the longitudinal bars where practicable.

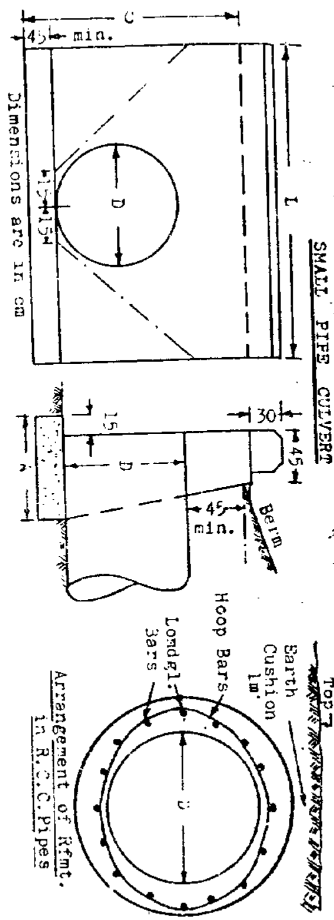
Pipes 1050 mm (42 ins.) or larger in diameter should be elongated vertically 5 per cent by field strutting. The struts shall be placed before the backfill is started.

High headwalls need not be provided for retaining deep overfills as they are costly; instead, the length of the culvert should be increased suitable so that the road embankment, with its natural side slopes, is accommodated without high retaining walls.



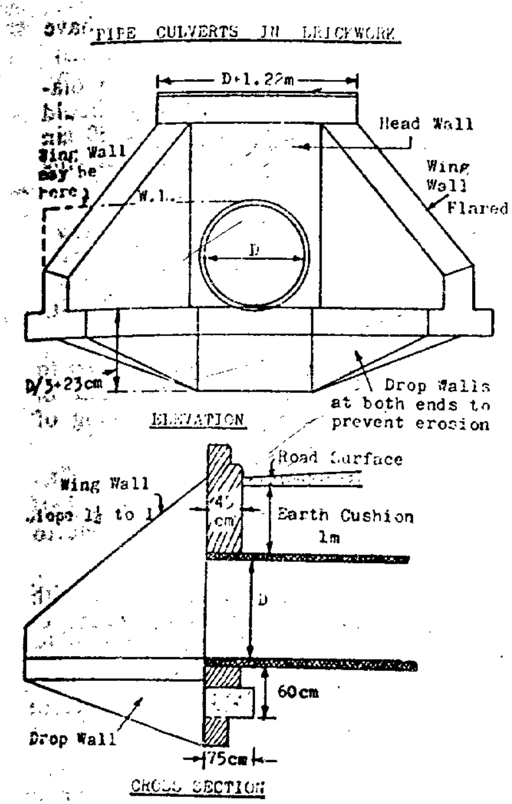
Dia* mm	A mm	B cm	C cm	D cm	E cm	F cm	G cm	H cm	I cm	J cm	K cm	L cm	M cm	T mm
1065	1065	152	176	60	136	175	137	25	15	34	25	34	9	114
1220	1220	168	194	64	151	195	152	25	15	37	25	38	10	127
1370	1370	183	212	70	170	218	168	28	17	40	28	41	11	140
1525	1525	198	229	76	188	240	183	30	18	44	30	46	13	152

Type Designs of Virginia Department of Highways (U.S.A) *Dia. is internal diameter of pipe.



Min Dimensions

D mm	L cm	C cm	A cm
305	183	122	61
380	206	129	61
460	229	137	61
610	274	152	69
915	366	183	76



Corrugated Galvanized Iron Sheet Pipe Culverts

Dia. of pipe in mm	up to 500	600 to 750	900 to 1350	1500	1800
Gauge No.	16	14	12	10	8
Thickness of sheet mm	1.60	2.00	2.50	3.15	4.00
Dia. of Rivets mm	8	8	10	10	10

Pipe Culverts

Dia.	Thickness of shell (barrel)			Reinforcement for concrete pipes					
	Bwk. cement	Plain cement conct.	R. C. conct.	Hoop bars			Longitudinal bars		
					Dia.	Spacing		Dia.	Spacing
mm	cm	mm	mm	No.	mm	cm	No.	mm	cm
300	10—4½″	75	—						
600	20—9″	100	65	1	6	7	1	6	10
900	25—9″	125	75	2	6	10	2	6	15
1200	30—13½″	150	90	2	6	7	2	6	15
1500	35—13½″	180	115	2	10	12	2	10	22
1800	35—13½″	200	125	2	10	10	2	10	20

Sheet culverts must be surrounded by concrete and should have adequate earth cushion. Use heavier gauge under heavy fills.

The longitudinal laps in all pipes 1050 mm (42 ins.) or more in diameter should be double riveted. Circumferential shop-riveted laps should have a max: rivet spacing of 15 cm; 6 rivets will be sufficient in a 30 cm pipe. The outside laps of circumferential joints should be pointing upstream and the longitudinal laps on the sides.

Rules for Construction of Pipe Culverts under Deep Fills

(Based on British Ministry of Transport instructions)

(*a*) All pipes and tubes with 6 metres of cover to be surrounded with at least 15 cm of concrete.

(*b*) Subject to (*a*) all pipes and tubes with over 4.27 metres of cover to be bedded on and haunched with at least 15 cm of concrete to a height of at least half the external diameter of the pipe or tube. Any splaying of the concrete to be above that level.

(*c*) Subject to (*a*) all pipes and tubes of 46 cm diameter and over to be bedded on and haunched with at least 15 cm of concrete to at least half the external diameter of the pipe or tube. Any splaying of the concrete to be above that level.

(*d*) Subject to (*e*) all pipes and tubes under 46 cm diameter and with less than 4.27 metres of cover may be laid without concrete if joints are of the socket or collar type, but concrete tubes with ogee joints are permissible when laid as in (*a*), (*b*) and (*e*).

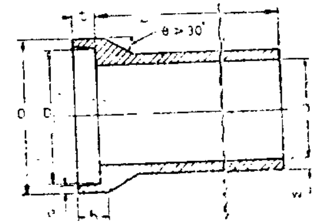
(*e*) All pipes and tubes with less than 0.91 metre of cover in fields or 1.22 metres of cover in roads to be surrounded with at least 15 cm of concrete.

(*f*) Every culvert under a highway should be so laid that the minimum distance from the finished surface of the road-bed to the top of the pipe is not less than one-half the diameter of the pipe with a minimum of 30 cm.

Loads on Pipes under Heavy Fills

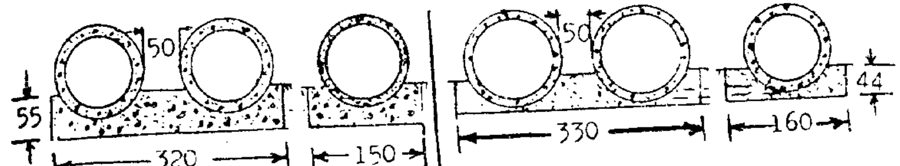
The exact amount and the nature of load under heavy fills still remains a disputed point and depends upon many factors. It is not like the arch action of masonry walls as taken for lintels. According to the results of some recent researches carried out in America it is considered that: (*i*) In the case of conduits laid in cuts or the linings of tunnels, no further increase in pressure on the pipe conduit is to be expected if the depth of trench exceeds nine times its width. (*ii*) For culverts under heavy filling the earth pressure is less than the total weight of the material over the pipe if the pipe is flexible and the top of the pipe deflects more than the adjacent soil. If the pipe is rigid, the total load coming on the pipe is more than the weight of the earth prism over the pipe depending on the stiffness of the pipe, the type of its bedding and the amount of compaction of the fill around it. Another point to be considered is the dispersion of the live load under the heavy fills. Also see "Approach Slab" under "Abutments".

Concrete Craddle Bedding for Heavy Loads **(Class A Bedding)** Minimum thickness under the pipe is one-fourth of its internal diameter and extending up the sides of the pipe for a height equal to one-fourth of its outer diameter. (Bedding over granular material for — first class bedding, is described in Section 16). It is very important to tamp well the side-fills of the pipe as the side-fills help very much the pipe in carrying vertical loads.



SPIGOT DIMENSIONS OF NP1 CLASS PIPES IS: 458

R.C.C. PIPE BEDDINGS FOR CULVERTS



1:2:4 c.c. cradle for heights of fill 4 to 8 m | Fine compacted granular material for fills up to 4 m

All dimensions are in cm. Longitudinal slopes of pipes are 1:1000. ISI class NP3 pipes, usually of 1 m dia., are used.

Masonry Arch Culverts

Size of arches for IRC Class "A" loading has been tabulaled earlier. For Class "B" loading, the following thickness of arch rings for brick-work in cement mortar may be taken:—

For Spans up to 1.5 metres	20 cm — 9″
For Spans 1.8 to 3.0 metres	30 cm — 13½″
For Spans 3.3 to 5.5 metres	40 cm — 1′ 6″

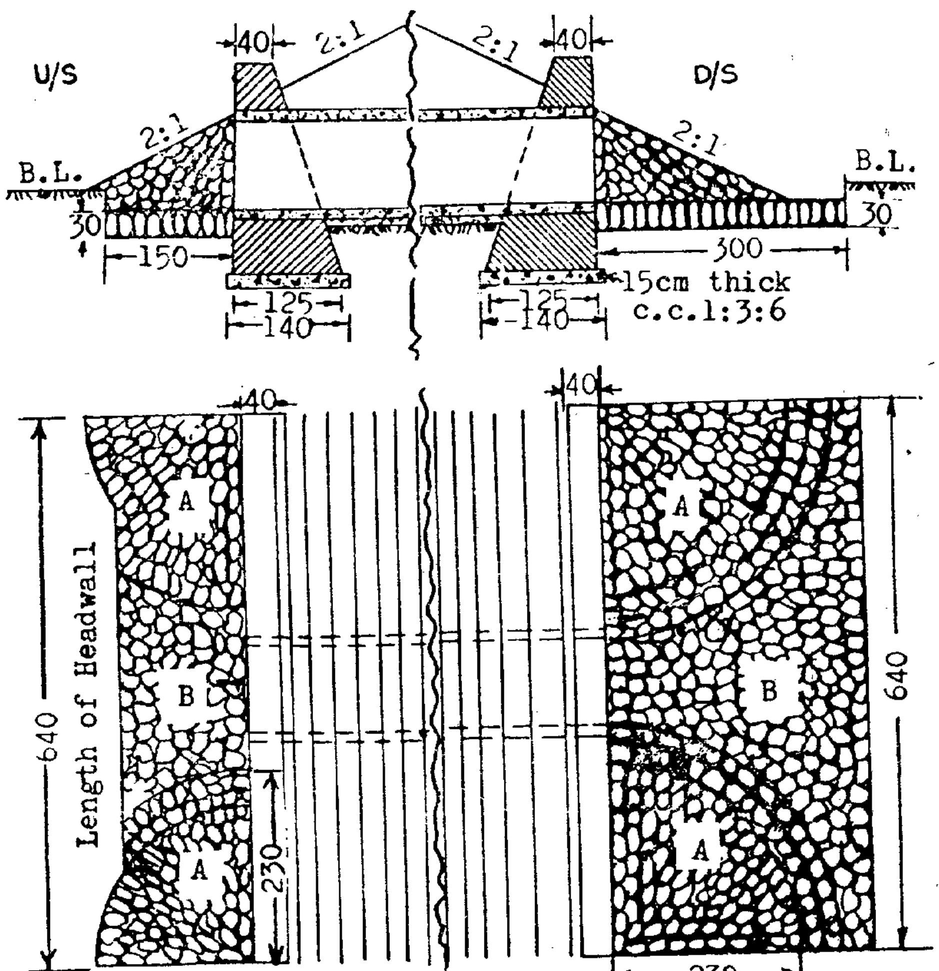
There should be an earth cushion of at least 45 cm. above the crown of the arch.

21. RIVER TRAINING WORKS

(Also see under "Irrigation")

Training works will usually be necessary for bridges and culverts where the waterway is contracted by the abutments or by the banks or where diagonal currents are produced. Training bunds should be parallel to the main current for a length upstream equal to the length of the bridge and downstream for 1/10 to 1/5th of this length. The exposed ends of training bunds should be curved off at an angle of 120 deg. to 140 deg.

R.C.C. PIPE CULVERT



PLAN

(a) All dimensions are in centimetres.

(b) "A" is 30 cm thick dry stone or brick revetment over fine granular material 15 cm thick.

(c) "B" is 30 cm thick dry stone or brick pitching for apron.

(d) Where two pipes are used, the length of headwall will be 810 cm instead of 640 cm. (IRC:13)

All obstructions in the river bed likely to divert the river current or cause undue turbulent flow or scour shall be cleared for a distance of not less than the length of the bridge subject to a minimum of 91 metres both upstream and downstream of the structure. Attention should be given to river training and protection of banks in the same length of the river.

Wire Crates & Mattresses for River Training Works

Wire crates for shallow or accessible situations can be about 3 x 1.5 x 1.2 metres in size. Where there is a chance of overturning, the crates should be divided into compartments. For deep and inaccessible situations, wire crates must be made smaller according to the situation. Wire mattresses built in situ should not be larger than 7.6 x 3.0 x 0.61 metres or smaller than 1.83 x 0.91 x 0.30 metres. Sides of large mattresses should be securely stayed at intervals of not more than 1.5 metres to prevent bulging.

The size of the netting will depend upon the size of the boulders available. For size of boulders 20 cm min: 15 cm mesh will be required and can be made from No. 6 gauge (5 mm) galvanized iron wire. This will require approximately 32 kg of wire per 10 sq. metres. For 15 cm boulders, a 10 cm mesh or 15 cm x 10 cm mesh can be woven with No. 8 gauge (4 mm) wire. Boulders 30 cm average diameter and weighing not less than 36 kg are best.

Stone Riprap for Fonndation Protection: Stone for pier and abutment protection shall range in size from the heaviest that can be handled and shall be graded from coarse to fine in such manner as to produce a minimum of voids. Where subject to ocean waves or heavy scour, specify stones from 1 to 6 tonnes.

22. ECONOMIC SPAN LENGTHS

The number of spans should be as few as possible, particularly in mountainous regions where torrential velocities prevail, since piers obstruct flow of water and multiply the difficulties met with in the foundations. The most economical span length is when the cost of the superstructure of one span is equal to the cost of one pier. The following approximate "thumb rules" are generally taken as a guide for determining the length of spans in small structures, where open foundations can be laid:

(*i*) Masonry arches: $S=2H$ or more

(*ii*) R C C slabs on masonry piers: $S=1\frac{1}{2}$

(*iii*) R C C beam and slab on masonry piers: $S=1\frac{3}{4}H$

(*iv*) R C C slab on pile bents: $S=\frac{3}{4}H$ to $1H$

(*v*) Steel truss spans on masonry pier: $S=3H$

where:

S=clear span length;

H=total height of abutment or pier from the undersides of its foundation to its top, and for arches to the intrados of the key-stone.

These rules will not apply where there is great disparity in the leads of the principal materials of construction, especially in the case of cement and steel when they have to be transported over long distances at heavy cost.

23. PREPARATION OF PROJECT ESTIMATES FOR BRIDGES

In the *Report* in addition to other descriptions reasons should be explained for selection of the particular site for the crossing. If necessary, other typical cross-sections of the stream at alternative suitable crossing places both upstream and downstream of the selected site should be enclosed. Possible behaviour of the river and the velocity of the stream should be detailed. A separate note should describe:—

The result of trial pits or bore holes showing levels of the various strata and the hard strata suitable for foundation and the intensity of pressure on the foundation soil. Trial pits and borings should be taken at least 1½ to 2 times the design depth. The estimated depth of scour with details of observations or any other special causes for scour.

Design:

The live load for which the bridge is to be designed should be detailed and the calculations for the design attached.

Drawings:

The following drawings should be prepared.

(*a*) *An Index Map* showing the proposed location of the bridge, and alternative sites, if any, and also the existing and proposed communications. Topographical sheets can be utilized with scale 1 cm to 500 metres or 1/50,000.

(*b*) *A site Plan* drawn to a suitable scale showing details of the site selected and extending not less than 100 metres upstream and downstream and covering the approaches to a sufficient distance which, in the case of a large bridge, shall not be less than 500 metres on either side of the channel.

(*c*) *A Contour Survey Plan* of the stream showing all the topographical features and extending to the distances shown below upstream, downstream and to a sufficient distance on either side to give a clear indication of all the features that might influence the location and design of the bridge.

100 metres for catchment areas less than 3 sq. km (scale not less than 1 cm to 10 metres or 1/1000).

300 metres for catchment areas of 3 to 15 sq. km (scale not less than 1 cm to 10 metres or 1/1000).

1½ km for catchment areas of more than 15 sq. km (scale not less than 1 cm to 50 metres or 1/5000).

Complete details about the catchment area on which the waterway of the bridge and its full design depends, are very essential.

(*d*) *A Cross-Secton* of the stream at the site of the proposed crossing and two other cross-sections at suitable distances, one upstream and the other downstream, all to an horizontal scale of not less than 1 cm to 10 metres or 1/1000 and with an exaggerated vertical scale of not less than 1 cm to 1 metre or 1/100. It should show: The highest flood levels, ordinary flood level, low water level, bed level, the nature of the soil and the various strata down below the depth of foundations.

(*e*) *A Longitudinal Section* of the stream extending to about 100 to 150 metres on either side of bridge site and showing at all the levels described for the cross-section. The horizontal scale shall be the same as for the survey plan and the vertical scale not less than 1 cm to 10 metres or 1/1000.

(*f*) *Boring Charts*

24. GLOSSARY OF TERMS

Abutment Pier—Is a heavier prier and is usually every fourth or fifth pier, built in an arched bridge, (with series of arches) designed to withstand heavy unbalanced inclined thrust from the superstructure (if some of the arches give way).

Afflux—See page 19/7

Air lock—In bridge caissons the chamber at the top of the shaft leading to the work and through which men or materials may be passed between the open air and the compressed air in the working space.

Apron—Is a layer of concrete, masonry, stone, etc. placed (like a flooring) at the entrance or outlet of a culvert or waterway, to prevent scour.

Backwater—See page 19/7

Buffle Wall, Dwarf Wall, Drop Wall or Curtain Wall—It is a thin wall used as a shield or protection against scouring action of a stream (as distinct from a retaining wall).

Boil or Blow—The phenomenon of the soil (quicksand) being forced into the cofferdam, caisson or an excavation from under the bottom by the upward water pressure. This is caused by the greater water pressure on the outside than on the inside.

Bowstring Girder—A girder consisting of a curved rib or arch having a horizontal tension member arranged as a chord and connected to the rib by hangers.

Buckle Plate Flooring—Steel plates bent to curved shape used for railway bridge flooring generally.

Buoyancy—The loss in weight of a body immersed partly or wholly in a fluid, due to the resultant upward pressure exerted on it by the fluid.

Caisson—See page 19/14

Catchment Area—(*i*) The area drained by a water course at any section (*ii*) Area from which the rainfall flows into (*a*) a drainage line at any pecified section or (*b*) a reservoir. See page 19/3

Catchment Drain—A drain excavated on the upper slope of a hill road o intercept and collect water flowing towards the road.

Causeway—Is a submersible small bridge. (*i*) A paved dip in a road across a shallow drainage course, at or about the bed level. (*ii*) A paved road crossing at or about the ground level, a water logged or marshy area. See page 19/42

Coffer Dam—See page 19/13

Cribs—The term has two meanings. In Bridge Engineering it is used for a temporary pier made in a river bed as explained earlier. In Highway Engineering, cribs are T sections made of precast RCC which are assembled into a wall for retaining stones and rubble, as a retaining wall. (Not generally used in India) See page 19/29,46

Cut-Off Wall—A wall, collar, or other structure intended to cut-off or reduce percolation of water along smooth surfaces, or through porous strata.

Cut-Water—See page 19/23

Deck Bridge—A bridge in which the carriageway is built at or near the top level of the main supporting members of the superstructure.

Dike—Earth dam or embankment. (Term used generally in America).

Dolphin—Cluster of piles driven in water for mooring purposes or for protection against floating objects.

Dumbbell Pier—A pier consisting of two RCC columns connected by thin RCC web for their full heights.

Ease-Water—See page 19/23

Fender—A replaceable device for protecting structure from damage caused by impact from floating bodies.

Flood Escape—A (lowered) section of a road specially designed to permit the escape over it of flood water rising over a specified level, without damaging any road or bridge structures nearby. Same as *Breaching* Section.

Flush Causeway—A causeway at bed level of a stream.

Free-broad—See page 19/7

Gorge—A narrow passage between hills.

Guard Rail—A side barrier or protection consisting of a rail supported on posts to constrain traffic.

High level Bridge—Is a bridge which carrie's the roadway above the highest flood level of the stream.

Head-room—Vertical distance between the highest point of a vehicle and the lowest point of any protuding member of a bridge.

Head-wall—See page 19/46,47

Irish Bridge—See page 19/43

Kerb Inlet—Apertures formed in a kerb to convey storm water to a gulley.

Lattice Girder—Is merely an I-beam with some of the web material cut away. The flanges are held apart and the shear resisted by diagonal and vertical bracings instead of by a solid web.

Over Bridge—A bridge that enables one form of land communication over the other.

Piping—The flow of water under or around a structure built on permeable foundations, which if not prevented or stopped will remove material from beneath the structure and cause it to fail.

Ramp Bridge—A temporary suspension bridge in which the roadway rests directly on the suspension cables and consists of wooden planks.

Revetment—Material, such as stones, concrete blocks or mattresses, placed on the bottom or banks of a river to prevent or minimize erosion.

Rip-rap—See page 19/55

Running Ground—Either water-bearing sand or very dry sand that will not stand up without support. (See under "Quicksand" in Section 6).

Scuppers—See page 19/43

Skin friction—The frictional resistance of the surrounding soil on surface of the caisson walls or the well steening.

Square Crossing—When the alignment of the centre line of a bridge is at right angles to the flow. When the alignment is at an angle, it is called *Skew Crossing*.

Submersible Bridge—A bridge designed to allow normal floods to pass through its vents but allowed to be over-topped during high floods.

Substructure—That part of a bridge or culvert which lies above the foundation and below the superstructure seats, or below the springing line of arches.

Through Bridge—A bridge in which the carriageway or flooring is supported or suspended at the bottom of main supporting members of the superstructure.

Trestle Bridge—A bridge composed of pilebents or towers carrying the deck.

Trough Flooring—Steel plates made in the forms of troughs used for railway bridge flooring generally.

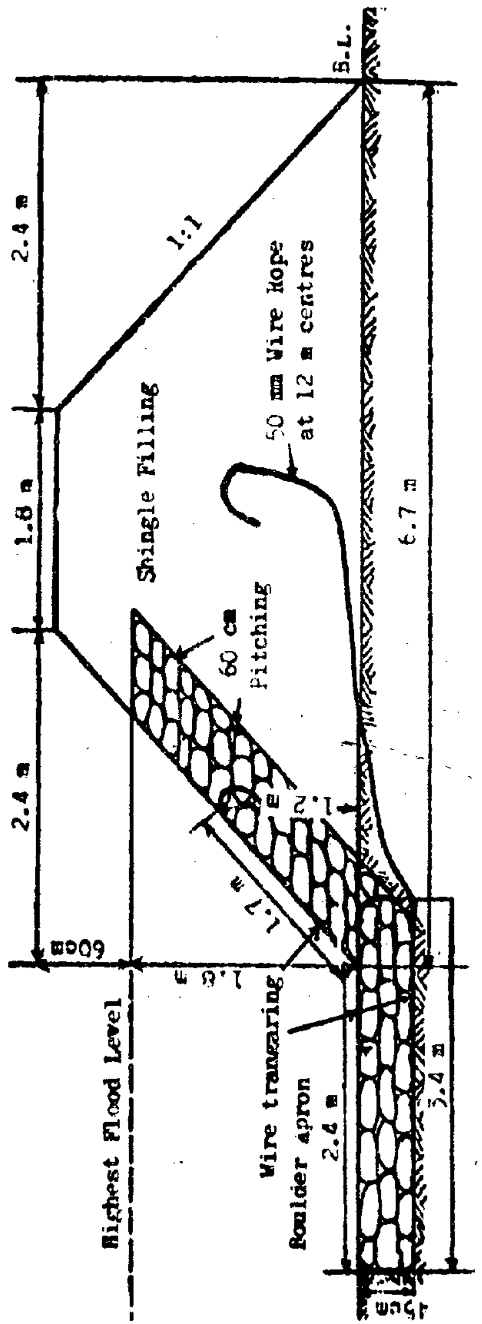
Under Bridge— A bridge constructed to enable a road to pass under another work or obstruction.

Vanes—Longitudinal walls usually built in continuation of piers to fan out the flow as desired.

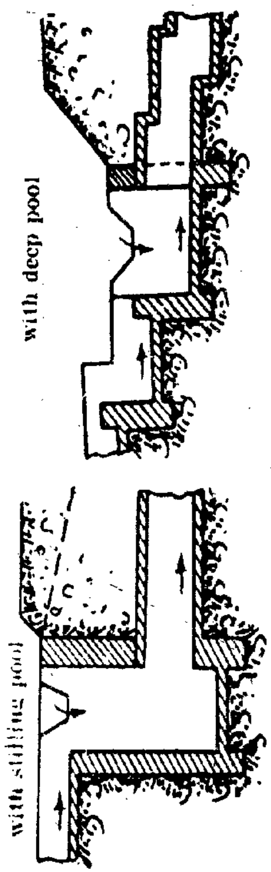
Viaduct—Is a long continuous structure which carries a road or railway line, a bridge over a dry valley (instead of over water) composed of a series of spans over trestle bents instead of solid piers.

Water Cushion—A pool of water maintained to absorb the impact of water flowing over a dam, chute, drop or other spillway structure.

Waterway—The area through which the water flows under a bridge superstructure is known as the waterway of the bridge,



AVERAGE SECTION OF GUIDE BUND



Connection of approach channel with culvert

SECTION 20

ESTIMATING & QUANTITIES

(For Water Supply and Road Works see under the respective Sections)

(Estimated Costs given are those prevalent at the time of writing)

In an ordinary type of building the cost of materials come to about 60% and labour about 40% of the total cost of the whole building. This may be further sub-divided as percentage of the total cost :—

Materials : Bricks—25%; Cement—13%; Steel—10%; Timber—12%.

Labour : Excavation—1%; Masons—25%; Carpenters—12%; Smiths—2%.

The costs for separate items of works may be roughly as follows, as percentage of the total cost of the building, for a single storey house :

Excavation and concrete in foundations ...	3%
Brickwork up to plinth	5%
Superstructure—brickwork	25%
Roofing	20%
Flooring	6%
Woodwork—doors and windows	15%
Internal finishes	6%
External finishes	3%
Water supply	4%
Sanitary work	8%
Electrification	5%
	100

The following figures may be taken for the materials required on plinth area basis for a single storey building, per square metre of the built-up area :

Bricks—350 nos.; Cement—1 bag; Steel—8 kg; Timber—0.09 cu m.

Note regarding labour :

The amount of work that an artisan or an unskilled labourer puts in is very variable depending on the skill of the man, his stamina, location, climate and several other factors. Therefore, the figures given for the labour items are more or less approximate.

1. EARTH-WORK & EXCAVATION

One labourer does in one day roughly the following tasks in ordinary soil from soft to moderately hard :

(*i*) Excavation in trenches for foundations of buildings, manholes etc., with 1.5 metres depth (or lift) and getting out the earth : 2.5 cu. m

(*ii*) Excavation in borrow pits and getting out the earth with lead up to 15 metres : 3.0 cu. m

(*iii*) Surface digging average 15 cm deep and getting out the spoil with 15 metres lead : 15 sq. m

(*iv*) Returning, filling in, including watering and raming in layers, 15 cm thick : 6.5 cu. m

(*v*) Forming embankments, watering and ramming in layers 1.5 cm high : 4.0 cu. m

(*vi*) Removing earth in baskets, distance of 50 metres and depositing : 2.5 cu. m

Labour involved in lifting a load through one metre height is taken equal to travelling a horizontal distance of ten metres.

Hard clay : That must be excavated with a pick, and cannot be remoulded in fingers.

Medium clay : That can be excavated with spade and remoulded with difficulty.

Soft clay : That can be excavated with shovel and easily remoulded in fingers.

Excavations in the following soils under above conditions will be :—

In soft moorum or hard earth ...	2.00 cu. m
In hard moorum ...	1.00 cu. m
In soft rock ...	0.55 cu. m
In hard rock ...	0·25 cu. m

Earth when dug from a pit increases in bulk by about 25 per cent, sand and gravel 10 per cent, sand 20 per cent, chalk 30 per cent.

Following allowances should be made in profiles of embankments for settlement of fresh deposited earth :

Compact earth fill ...	10 cm per metre height
Loose earth fill ...	15 cm per metre height
Black cotton soil ...	20 cm per metre height

2. MORTARS

Proportions of mortars for various types of works are given in Section 12. Dry ingredients when mixed together are less in volume than the sum total volumes of all the ingredients together. The volume is further reduced when water is added to the dry materials for making mortar, and this reduction may be anything from 20 to 33 per cent of the original volume.

Cement-Lime Mortars

Quantities per cubic metre of wet mortar

Proportion of mix Cement : lime* : sand	Cement kg	Lime* cu. m	Sand cu. m
1:1:3	410	0.284	0.852
1:1:6	250	0.170	1.020
1:1:7	220	0.153	1.071
1:1:8	190	0.132	1.056
1:2:9	170	0.263	1.062

* Lime putty or paste.

In the case of lime mortars, the volume of wet mortar is about two-thirds of the sum total of all the ingredients. Thus, 1 cu. m sand+1 cu. m lime+1 cu. m surkhi will make about 2.5 to 2.75 cu. m of dry and 1.75 to 2 cu. m of wet mortar. 2 cu. m lime+6 cu. m sand will produce about 8.25 cu. m of dry mix and about 6 cu. m of wet mortar. For practical purposes, to make one cu. m of wet mortar about 1.4 cu. m of dry ingredients (sum total of the all) should be taken.

For cement mortars, to make one cu. m of wet mortar, 1.42 to 1.21 of cement and dry sand are required for 1:1 to 1:6 proportions. (See Table). One cu. m of neat cement will produce only 0.87 cu. m of cement paste.

Lime Mortars—Quantities per cubic metre of wet mortar

Proportion of mix		1:1	1:2	1:3
White lime slaked	cu. m	0.700	0.475	0.357
Surkhi, sand or cinder	cu. m	0.700	0.950	1.070

Cement Motars—Quantties of cement and dry sand required per cubic metre of wet mortar :

Mix proportions cement : sand	Cement kg	Sand cu. m	Sand per 100 kg of cement cu. m
1:1	1020	0.71	0.07
1:2	680	0.95	0.14
1:3	510	1.05	0.21
1:4	380	1.05	0.28
1:5	310	1.05	0.34
1:6	250	1.05	0.42
1:7	220	1.05	0.48
1:8	200	1.05	0.54

The quantities of cement given are only average and inclusive of wastage, good for practical purposes. The actual quantity of cement varies according to the fineness of the sand used. Fine sands need more cement than coarse sands. Well graded sands need much less cement. Cement occupies the viods in the sand. Water also occupies space.

Where white cement and marble dust are used for decorative plaster, the same quantities of cement and marble dust, instead of sand, are used.

3. CONCRETES

(See Section 12 under "Mixtures for Mortars and Concretes".)

Breaking brick and stone metal: One man will break about the following quantities of metal in one day depending on the size of the brick-bats and the hardness of the stone.

Brick aggregate : about 0.80 to 1.00 cu. m—25 to 40 mm
about 0.50 to 0.65 cu. m—20 to 25 mm

About 390 to 420 9-inch bricks and 420 to 440 Indian Standard

Modular (IS) bricks make one cu. m of bricks aggregate. 280 to 350 9-inch bricks and 300 to 370 IS bricks make one cu. m of brick-bats. 125 cu. m of brick-bats make 100 cu. m of aggregate.

Stone aggregate : about 0.55 to 0.80 cu. m—25 to 40 mm
about 0.10 to 0.15 cu. m—12 to 20 mm
about 0.06 to 0.09 cu. m— 6 to 17 mm

Broken brick or stone metal when re-stacked and measured may show a difference in volume up to 7 per cent, while gravel or shingle may differ as much as 10 per cent. Metal loaded in a railway wagon settles down due to jerking during transit and will show a lesser quantity by about 7 to 12 per cent when re-measured.

(*a*) **Lime Concrete** in foundations—Per cu. m of finished concrete :

Materials		*Labour*
Aggregate—40 mm nominal size	1.10 cu. m	
Wet mortar—	0.4 cu. m	Mason—1/10
Dry mortars:		Mazdoor—1.75
kankar lime	0.42 cu. m	Bhisty—2/5
or		
white lime	0.17 cu. m	Materials available at site of work.
sand of surkhi	0.34 cu. m	
or		
white lime	0.18 cu. m	Ramming reduces the volume of the aggregate.
sand	0.18 cu. m	
surkhi	0.18 cu. m	

For concrete over terraced roofs or Jack-arches, 10 per cent extra mortar should be used with aggregate of 12 mm to 20 mm size. (Add extra for water-proofing concrete. See specifications in Sections 7 and 11.)

Labour : Mason—1/4; Mazdzoor—2; Bhisty—3/5.

Brick aggregate shall be broken from well burnt or slightly over-burnt and dense bricks or brick-bats. Brick aggregate must be soaked well in water before mortar is mixed with it. The brick aggregate should not absorb more than 25 per cent of its weight of water after 24 hours immersion.

"All-in" Aggregate *	Cement 50-kg bag	Combined Aggregate in cu. m	All combined aggregate to pass a 50-mm ring and to contain sufficient sand
1:5 mix	1	0.175	
1:6 "	1	0.210	* 1:6 "all-in" aggregate is not the same as 1:2:4 mix, which will be about 1:5
1:8 "	1	0.280	
1:10 "	1	0.350	
1:12 "	1	0.420	

Weight of Cement: One 50-kg bag of common grey cement is taken to contain 34.72 or say, 35 litres (or 0.035 cu. metre) and to weight 1440 kg/cu. m (nett). Weight of sack is extra. (It is 1300 to 1400 kg/cu. m when loosely packed and 1700 kg/cu. m well compacted.)

Rapid-hardening cement is taken to weigh 1200 kg/cu. m.

Cement Concrete

Quantities of material and labour required for various mixes, per one cubic metre of finished concrete: Exact quantity of the concrete produced depends upon the type, size and grading of the aggregate and sand, and also the quantity of water used in the mix.

Mix (Nominal)	Cement in kilograms				Dry Sand (av.)	Aggregate graded (av.) 12 to 25 mm
	Machine mixing		Hand mixing			
	Gravel Shingle	Broken Stone	Gravel Shingle	Broken Stone	cu. m	cu. m
1:1:2	550	580	570	600	0.40	0.80
1:1.5:3	370	390	380	400	0.42	0.84
1:2:3	360	380	370	390	0.54	0.81
1:2:4	290	310	300	320	0·45	0.90
1:2.5:5	250	270	260	280	0.46	0.92
1:3:6	190	210	200	220	0.46	0.92
1:4:8	140	160	150	170	0.47	0.94
1:5:10	120	130	120	130	0.48	0.96
1:6:12	100	110	100	110	0.49	0.98
1:6:18	70	80	70	80	0.35	1.00

Labour—Mass Work: Mason—1/4; Madoor—1.5 Bhisty—1/2

Hand mixing is for small jobs and needs more cement than machine mixing. Gravel agreegate need about 5 per cent less cement and about 2 per cent more sand and coarse aggregate in proportion to sand than required for crushed rock or broken stone.

Volume of aggregate is reduced by ramming. Cement concrete is not rammed heavily like lime concrete.

Cement for small jobs

One 50-kg bag of cement will make approximately:

0.133 cu. m of 1 : 1.5 : 3 concrete
0.166 cu. m of 1 : 2 : 4 "
0.192 cu. m of 1 : 2.5 : 5 "
0.250 cu. m of 1 : 3 : 6 "
0.333 cu. m of 1 : 4 : 8 "
0.400 cu. m of 1 : 5 : 10 "
0.500 cu. m of 1 : 6 : 12 "

The equivalent size of a box for 50-kg cement bag is 40 cm × 35 cm × 25 cm internally.

Cement should be measured by weight. Measurement by volume is very variable.

Very approximate figures of the quantities of ingredients of concrete may be obtained by considering the volume of all the ingredients at 1.5 times that of the resultant concrete.

Reinforced Concrete. *Reinforcement for Slabs and Beams:*

Take about 70 kg of reinforcement rods per cubic metre of concrete where only simple tensile bars are used, and about 110 kg where stirrups, transverse and shear reinforcements are also to be included. Binding wire 14 or 16 gauge (2 mm or 1.6 mm) may be taken 2 to 4 kg per tonne of bars.

In estimating the quantity of mild steel rods (where exact figures are required) add an allowance for both side hooked ends as under:—

Dia. of rod in mm	6	10	12	16	20	22	25	32	40
Allowance in cm	12	19	25	31	37	40	45	56	70

Or this may be taken roughly 15 per cent more than the total quantity required by design for anchorage, lappage and wastage.

Reinforcement for Columns:

Take about 160 kg of reinforcement rods per cubic metre of concrete, and binding wire about 1.5 kg per tonne of bars.

Labour for Cutting and Bending of reinforcement rods:

6 mm to 10 mm dia.	—120 kg	one man per day
12 mm	—150 kg	" "
16 mm and above	—220 kg	" "

Extra cement for finishing concrete surfaces: per 100 sq. m of surface.

(*a*) Smooth finishing of concrete surfaces with cement mortar 1 : 2 including floating with neat cement: 417 kg of cement

(*b*) Ditto. of RC work with 1 : 3 and ditto.: 367 kg of cement

(*c*) Finishing floor surfaces, etc., with neat cement: 220 kg of cement

(*d*) Applying cement slurry: 275 kg of cement

Formwork: The cost of formwork is estimated as between 1/3rd to 1/4th of the total cost of a concrete structure.

Approximate comparative costs of lime and cement concretes in foundations:

Lime concrete		Rs. 50/- per cu. m
Cement concrete	1 : 2 : 4	Rs. 110/- "
Ditto.	1 : 3 : 6	Rs. 90/- "
Ditto.	1 : 4 : 8	Rs. 78/- "
Ditto.	1 : 5 : 10	Rs. 68/- "
Ditto.	1 : 6 : 12	Rs. 62/- "

Labour for laying RC Roof Slab—Per 10 sq. m:

Mason —4
Carpenter —1/2
Mazdoor —6
Bhisty —2

Hire charges for forms and column bricks for supporting the forms should be added.

Pre-cast Concrete Works (in steel moulds) :

Making blocks in hand machine :

1 mason and 2 mazdoors will make in one day :

Hollow blocks—120 nos. ; Paving flags—35 nos. ;
Roofing tiles—100 nos.

4. BRICKWORK

The number of bricks, the quantity of mortar and the labour required depend upon the actual size of bricks, the thickness of wall, thickness of joints, if the bricks are with frogs or plain, and the composition of the mortar used.

Material and labour required for one cubic metre of brickwork—average and inclusive of wastage :—

Bricks—500 nos. (for both IS and 9″ bricks—actual size of 9″ bricks being variable, average quantity has been taken).

Mortar (wet)—0.23 cu. m for IS bricks
0.25 cu. m for 9″ bricks.

Mortar (dry)—sum total of all the ingredients :

Lime mortar—0.325 cu. m for IS bricks
0.350 cu. m for 9″ bricks

Kankar lime—0.275 cu. m for IS bricks
0.300 cu. m for 9″ bricks

Mud mortar—0.375 cu. m for IS bricks
0.400 cu. m for 9″ bricks

Cement mortar—Quantities of cement and sand required per cu. m of brickwork :

Mortar proportion	1:2	1:3	1:4	1:5	1:6	1:7
IS bricks—0.23 cu. m wet mortar						
Cement in kg	156	117	87	71	58	51
Sand in cu. m	0.220	0.243	0.243	0.243	0.243	0.243
9″ bricks—0.25 cu. m wet mortar						
Cement in kg	170	128	95	78	63	55
Sand in cu. m	0.238	0.275	0.275	0.275	0.275	0.275

Labour :—One cubic metre of brickwork

Foundations and plinth		Superstructure	
Mason	—4/5	—9/10	Add for scaffolding etc.
Mazdoor	—1	—4/3	
Bhisty	—1/3	—1/3	

(*i*) For *archwork* with uncut bricks, mortar will be 10 per cent more and labour 1.75 times than required for simple work. Add extra for centerings etc.

(*ii*) *Honey-comb brickwork* requires 1/10th mortar and half the labour of solid work.

(*iii*) For *half-brickwork*, mortar will be 2.5 cu. m for IS bricks and 2.4 cu. m for 9″ bricks per 100 sq. m.

(*iv*) For well steening work, mortar will be 0.33 cu. m where cut bricks are used, and 0.25 cu. m where special moulded bricks are used.

Approximate comparative costs of different types of brickwork, per cu. m :—

Brickwork in mud mortar		Rs. 64/-
,, in lime ,,		Rs. 83/-
,, in lime : cement mortar		Rs. 91/-
,, in cement mortar	1 : 7	Rs. 83/-
,, in ,, ,,	1 : 6	Rs. 86/-
,, in ,, ,,	1 : 5	Rs. 89/-
,, in ,, ,,	1 : 4	Rs. 95/-
,, in ,, ,,	1 : 3	Rs. 105/-

5. STONE MASONRY

Per cubic metre of finished work

1. Boulder filling dry, hand packed, as in pitching :
   Stones—1.05 cu. m; Mason—1/3; Mazdoor—1/3.
2. Uncoursed random rubble walling laid dry in superstructure :
   Stones—1.20 cu. m; Mason—1/2; Mazdoor—1.
3. Uncoursed random rubble walling laid in mortar in superstructure :
   Stones—1.20 cu. m; Mortar (wet)—0.30 to 0.35 cu. m
   Mason—1.2; Mazdoor—1.5; Bhisty—1/5.
4. Coursed rubble walling laid in mortar in superstructure :
   Stones—1.25 cu. m; Mortar (wet)—0.30 cu. m
   Mason—1.5; Mazdoor—2; Bhisty—1/5.
5. Ashlar masonry in superstructure :
   Stones—1.30 cu. m (undressed);
   Mortar (wet)—0.20 cu. m; Mason—2.8
   Mazdoor—3; Bhisty—1/5.
6. Ditto in cement mortar 1:6, with pointing in cement mortar 1:2 :
   Quantity of cement required—54 kg.
7. Ditto in archwork in cement mortar 1 : 3, with pointing in cement mortar 1 : 2 :
   Quantity of cement required—107 kg

(*i*) Add extra to all items for scaffolding and heights.

(*ii*) For work in foundations and plinth, labour will be about 15 per cent less.

(*iii*) The labour includes dressing and cutting of stones.

(*iv*) Stones measured in loose stacks.

(v) The quantity of stones is inclusive of bond stones.

(vi) For quantity of dry mortar that would make wet mortar see under "Mortars".

6. FLOORING

Brick flooring : Per 10 square metres :

(i) 9″ bricks laid flat (3″ depth) over 12 mm thick mortar bed :

Bricks—380 nos. | Mason—3/4
Mortar (wet)—0.28 cu. m | Mazdoor—1.5

For work in 1 : 4 cement mortar —103 kg cement + 0.28 cu. m sand
,, 1 : 6 ,, ,, —108 kg cement + 0.28 cu. m sand

Add for pointing 1 : 2 (See under "Pointing")

(ii) IS bricks laid flat (10 cm depth) over 12 mm thick mortar bed :

Bricks—500 nos. | Mason—7/8
Mortar (wet)—0.30 cu. m | Mazdoor—1.75

For work in 1 : 4 cement mortar —110 kg cement + 0.30 cu. m sand
,, 1 : 6 ,, ,, — 72 kg cement + 0.30 cu. m sand

Add for painting 1 : 2

(iii) 9″ bricks laid on edge (4.5″ depth) over 12 mm thick mortar bed :

Bricks—570 nos. | Mason—1
Mortar (wet)—0.39 cu. m | Mazdoor—2

For work in 1 : 4 cement mortar —150 kg cement + 0.39 cu. m sand
,, 1 : 6 ,, ,, — 96 kg cement + 0.39 cu. m sand

(iv) Brick tile flooring (30 cm × 30 cm) laid in cement mortar 1 : 6 and surface pointed with cement mortar 1 : 2 :

Tiles—110 nos.
Cement—48 kg + sand—0.22 cu. m for tile laying
Cement—15 kg + sand—0.02 cu. m for pointing.

Concrete flooring

(i) 25 mm thick cement concrete 1 : 2 : 4, finished with floating coat of neat cement—per 10 sq. m

Cement—102 kg (80 kg for concrete + 22 kg for floating)
Sand—0.12 cu. m
Aggregate—0.24 cu. m—12.5 mm nominal size
(For 40 mm thick and above use 20 mm size).

Labour : Mason—1, Mazdoor—4/5, Bhisty—1/5.

(ii) 25 mm thick cement concrete 1 : 1.5 : 3, and ditto. :

Cement—122 kg (100 kg for concrete + 22 kg for floating)
Sand—0.11 cu. m
Aggregate—0.22 cu. m—12.5 mm nominal size

Labour : as for item No. (i)

(iii) 40 mm thick cement concrete 1 : 3 : 6 with 12 mm thick wearing surface on top of fine grit 1 : 2—Per 10 sq. m :

(a) For bottom layer

Cement—85 kg = 0.06 cu. m
Sand—0.18 cu. m
Gravel—0.36 cu. m—12.5 mm nominal size.

(b) For top layer

Cement—85 kg
Grit—0.12 cu. m—3 mm to 6 mm size.

(iv) 40 mm thick red oxide flooring consisting of under layer of 30 mm thick cement concrete 1 : 2 : 4 and top layer of 10 mm thick of cement plaster 1 : 3, mixed with red oxide using 3.5 kg red oxide per 50 kg (1 bag) of cement, finished with a floating coat of cement/red oxide mix of the same proportion. Cement required per 10 sq. m is 201 kg.

(v) 10 mm thick layer of marble chips, cut and polished finished, using light colour and white cement—Per 10 sq. m :

Marble chips—180 kg = 0.11 cu. m—3 mm size | Mason—2
White cement—100 kg | Mazdoor—2
Colouring pigment—5 to 10 kg | Polisher—8
Add for carborundum stone and beeswax | Bhisty—1/2

To get whitish base 10 to 20 per cent of marble dust may be mixed with grey cement—3 cement and 1 marble dust is common.

Colouring pigment (or red oxide) can be added 7 to 10 kg per 100 kg of cement for the top marble chips layer or the floating coat.

Proportions of cement and stone chips are 1 : 1 to 1 : 3. Take about 1 mm extra thickness of terrazo for cutting and rubbing of the floor.

Weight of colouring pigment used in excess of 12 per cent of the weight of cement reduces the strength of the mortar. Where strength will not matter, a maximum proportion of 1 colouring pigment to 3 of cement can be used. The proportion should not be less than 1 : 12. Steel or iron floats should not be used on coloured floors.

Carborundum stone is used for polishing the floor. They are of—course, medium and fine grades. 1 part beeswas and 3 parts turpentine are well rubbed over the floor for high class polish.

The following materials are required to treat about 10 sq. m of floor surface :

Beeswax—60 grams ; Turpentine—0.20 litres ; Pigment—40 grams. (See also Section 8)

Approximate comparative costs of different types of common floors : **Per sq. metre**

1. 75 mm thick lime concrete floor with brick aggregate		Rs. 5/-
Add or deduct for each 25 mm over or under 75 mm		Rs. 1/50
2. 25 mm thick cement concrete 1:2:4, finished with a floating coat of neat cement	...	Rs. 4/-
Add for each 15 mm over 25 mm	...	Rs. 1/80

	Add to the above for coloured cement surface finish	Rs. 1/80
3.	40 mm thick cement concrete 1:2:4 over 75 mm cement concrete 1:5:10 finished with a floating coat	Rs. 11/-
4.	20 mm thick cement concrete 1:2:4 over 60 mm cement concrete 1:6:12 finished with a floating coat	Rs. 6/50
5.	10 mm thick layer of terrazo including grinding and polishing	Rs. 14/-
6.	Brick flooring laid flat in cement mortar 1:6, with pointing 1:3	Rs. 6/-
	Ditto. laid on edge	Rs. 8/-
7.	12 mm thick cement plaster 1:3 over brick bats well rammed. Total finished thickness 100 mm	Rs. 6/-
8.	40 mm thick tiles laid flat bedded and jointed in cement mortar 1:6, with cement pointing 1:3	Rs. 4/50

7. POINTING

Per 100 square metres

(*a*) On brick walls : Mortar (wet)—0.31 cu. m

(*i*) In cement mortar 1:3
Cement—155 kg ; Sand—0.32 cu. m

(*ii*) In cement/lime mortar 1:1:3
Cement—125 kg; Lime putty—0.085 cu. m; Sand—0.255 cu. m

(*iii*) In white lime and surkhi mortar 1:1
Slaked white lime—0.30 cu. m ; Surkhi—0.30 cu. m.

(Quantity of materials is about the same for all types of pointing.)

Labour—including raking joints and watering

Flush pointing : Mason—10 ; Mazdoor—5 ; Bhisty—2
Ruled pointing : Mason—12 ; Mazdoor—5 ; Bhisty—2
Weather-struck pointing : Mason—14 ; Mazdoor—6 ; Bhisty—2.

Add for use of scaffolding and sundries

(*b*) On brick tile walls—tiles 4 cm thick

Material required is 1.5 times that of brickwork given under (a) above

Labour for flush pointing will be about same as for weather-struck pointing.

(*c*) On stone walls

(*i*) Flush or ruled pointing on random rubble walls :
Mortar (wet)—0.60 cu. m
In cement mortar 1:3
Cement—306 kg ; Sand—0.630 cu. m

(*ii*) Ditto. on ashlar work : Mortar (wet)—0.23 cu. m
Cement—117 kg ; Sand—0.20 cu. m

(*d*) On brick flooring in cement mortar :
Mortar (wet)—0.20 cu. m
1:2—Cement—136 kg ; Sand—0.186 cu. m
1:3—Cement—102 kg ; Sand—0.205 cu. m
1:4—Cement— 76 kg ; Sand—0.205 cu. m
1:6—Cement— 50 kg ; Sand—0.205 cu. m

(*e*) Flat brick tile roofing, size 19×9×4 cm

(*i*) Grouting with cement mortar 1:3
Cement—311 kg ; Sand—0.653 cu. m

(*ii*) Pointing with cement mortar 1:3 (after grouting)
Cement—76 kg ; Sand—0.160 cu. m

Fine sand is used for pointing.

Proportions of various mortar mixtures are given in Section 12 and also in Section 7.

8. PLASTERING

(See also in Section 7 and 12)

Cement plaster on walls : Per 100 square metres :

Proportion of mortar Cement : Sand	12 mm thick 1.44 cu. m wet		15 mm thick 1.72 cu. m (wet)		20 mm thick 2.24 cu. m (wet)	
	Cement	Sand	Cement	Sand	Cement	Sand
	kg	cu. m	kg	cu. m	kg	cu. m
1:2	979	1.371	1170	1.638	1523	2.132
1:3	734	1.541	877	1.842	1142	2.398
1:4	547	1.532	654	1.831	851	2.383
1:5	446	1.516	533	1.812	694	2.360
1:6	360	1.512	430	1.806	560	2.352
Extra mortar	20 per cent		15 per cent		12 per cent	

(*i*) The total quantity of mortars taken is shown in the respective columns with extra percentage which is needed for filling up the joints in brickwork and uneven surfaces, and also for some wastage.

(*ii*) Where plaster is to be finished with a floating coat of neat cement, take 220 kg/100 sq. metres extra cement.

(*iii*) For plastering on ceilings 6 mm thick the quantities will be half of 12 mm thick on walls.

Lime plaster on walls—12 mm thick : Per 100 sq. metres :

(*a*) Kankar lime—1.65 cu. m (dry)

(*b*) White lime —0.68 cu. m } 1 : 2 (dry)
Surkhi or sand—1.36 cu. m }

(c) White lime —1.0 cu. m } 1 : 2 (dry)
Surkhi or sand—1.0 cu. m }

Chopped jute or hemp—1.2 kg.

Labour : Mason—8 ; Mazdoor— 12 (including raking joints) ; Bhisty—2 (including curing).

For ceiling work, take 10 masons. Add for scaffolding etc.

(d) Dubbing out, average 10 mm thick, on irregular wall surfaces, in lime mortar 1:2 : Per 100 sq. m

White lime—0.60 cu. m; Sand—1.20 cu. m

Chopped jute or hemp—1.0 kg.

Labour : Mason—4.5 ; Mazdoor 4 ; Bhisty—1.5.

Cement-lime-sand plaster on walls : Per 100 sq. metres :

Proportions of mortar Cement : Lime* : Sand	12 mm thick			15 mm thick		
	Cement kg	Lime cu. m	Sand cu. m	Cement kg	Lime cu. m	Sand cu. m
1:1:6	360	0.25	1.50	430	0.30	1.80
1:1:7	317	0.22	1.54	378	0.26	1.83
1:1:8	274	0.19	1.52	327	0.23	1.82
1:2:9	245	0.34	1.53	292	0.45	1.82

* Lime putty or paste.

Approximate comparative costs of plastering on walls—12 mm thick :

Cement : Sand			Cement : Lime : Sand			Lime : Surkhi or Sand	
1:3	1:4	1:6	1:1:6	1:1:8	1:2:9	1:2	1:3
Rs. 4/80	Rs. 4/10	Rs. 3/40	Rs. 3/90	Rs. 3/50	Rs. 3/60	Rs. 3/20	Rs. 2/90

Mud plaster on walls—25 mm thick : Per 100 sq. m :

Mud—3.0 cu. m ; Bhusa or Straw (2 cm long)—100 kg ; Gobar for gobri leaping—0.03 cu. m.

Labour : Mason—4 ; Mazdoor—15 ; Bhisty—2.

9. WHITE-WASHING & DISTEMPERING

(Specifications given in Section 7)

1 coat —12 kg white lime unslacked per 100 sq. m

2 coats—22 kg ,, ,, ,,

3 coats—32 kg ,, ,, ,,

Old surface will require 10 kg lime for the 1st coat.

Gum may be added where desired at the rate of about 4 grams to one kg of lime to the last coat. A small quantity of indigo (blue pigment) may be added to kill the white glaze.

Labour : Per 100 sq. m

White-washing 1 coat —white-washer + cooly boy—2/3 each
,, 2 coats — ,, ,, —1 ,,
,, 3 ,, ,, —1.5 ,,

For work on ceilings, add 25% extra labour.

Distempering : (Specifications given in Section 7) : Per 100 sq. m.

Dry distemper—6.25 kg for one coat—5 kg for 2nd coat
Priming Coat—8 litres.

Labour : Per 100 sq. m

1st coat or one coat —Painter + cooly boy—2 each
2nd coat or subsequent coat— ,, ,, —1.5 ,,
Priming coat — ,, ,, —1.5 ,,

Cement washing : (Specifications given in Section 8)

2 Coats painting per 100 sq. m—2 Painters.

Colour Washing : Colouring pigment 1.5 kg added with white lime solution for every 100 sq. m of surface for each coat in the last one or two coats.

Snowcem painting : Per 100 sq. m

30 kg for 1st coat—Painter + cooly boy—2.5 each
20 kg for 2nd coat— ,, ,, —2 each

10. WOOD-WORK FOR DOORS & WINDOWS

(Sizes of frames given in Section 9)

Sawing of planks from logs of wood has not been taken in the labour items. Ready sawn planks are considered available at site of work. Labour items are for soft-wood. Quantity of wood is inclusive of wastage. Add for Fittings.

(a) (i) 50 mm thick panelled and glazed door with frame—1.22 m × 2.13 m (4' × 7') :

Wood—0.164 cu. m (inclusive of frame)

Labour : Carpenter—4, Glazier—1/4, Mazdoor—1.5

(ii) Ditto. all panelled :
Wood—0.198 cu. m (inclusive of frame)

Labour : Carpenter—4.5, Mazdoor—2.

(b) (i) 38 mm thick panelled and glazed door with frame—1.22 m × 2.13 m (4' × 7') :

Wood—0.142 cu. m
(frame 0.048 cu. m + shutter 0.094 cu. m)

(ii) Ditto. all panelled :

Wood—0.170 cu. m (inclusive of frame)

(*c*) 38 mm thick wire-gauze door with frame—1.22 m×1.98 m (4′×6½′):

Wood—0.124 cu. m

(frame 0.048 cu. m+shutter 0.076 cu. m)

Wire-gauze—1.21 sq. m

Labour: Carpenter—3, Mazdoor—1.

If hardwood (teak) is used the thickness can be reduced to 25 mm instead of 38 mm, and that will reduce the cost (material and labour) by about 15%. 32 mm thickness will reduce the cost by about 9%.

(*d*) (*i*) 32 mm thick battened door with 25 mm braces complete with frame—1.07 m×1.98 m (3½′×6½′):

Wood—0.156 cu. m

Labour: Carpenter—3, Mazdoor—1.

(*ii*) 38 mm thick, ditto. 1.22 m×2.13 m (rebated) (4′×7′):

Wood for shutter only—0.147 cu. m.

(*iii*) 25 mm thick, ditto. 0.914 m×1.829 m (butt jointed) (3′×6′):

Wood for shutter only—0.071 cu. m.

(*e*) 38 mm thick venetian doors and windows:

Wood—about 0.55 cu. m : Per 10 sq. m

Labour: about the same as for panelled and glazed.

(*f*) 38 mm thick glazed windows with frame 1.22 m×1.52 m (4′×5′):

Wood—0.096 cu. m. Add for glass panes and putty.

Labour: Carpenter—3, Glazier—1/4, Mazdoor—2.

(*g*) Jafri or Trellis work : Per 10 sq. m :

Wood—about 0.213 cu. m (inclusive of frames)

Labour: Carpenter—3, Mazdoor—2.

(*h*) Framing labour—0.085 cu. m one man per day.

(*i*) Joinery (hardwood)—0.042 cu. m one man per day.

11. PAINTING

Covering capacity of paints: The covering capacity of a paint varies according to the composition of the paint, type of the surface to be covered and the skill of the painter. The following may be taken as a general guide :—

One painter will do about 45 to 65 sq. metres of surface per day, which can be up to 90 sq. metres if the surface is of iron sheets and the paint is running freely. For more difficult and careful work such as doors and windows, the same painter may do only about 20 sq. metres.

Labour required for scraping old work (which might come to about the same wages as of the painter), and for the use of brushes, and sundries etc., (which is about 1/15 to 1/20 of the cost of labour and material) should be added.

(*A*) **Painting woodwork with ready mixed paints—Per 100 sq. m of surface**:

(*a*) Preparing surfaces to receive paint :

Materials required—knotting, putty, sand paper, pumice stone, etc., and giving priming coat.

Primer ready mixed—7 litres ; Painter—4.

(*b*) Painting 1st coat over primer (or under-coat) :

Paint ready mixed—6.5 litres ; Painter—2.

(*c*) Painting 2nd coat or 3rd coat, or on old surfaces (as a finishing coat) :

Paint ready mixed—6 litres.

(*d*) Painting on new wood surfaces without primer :

Paint—7 litres.

Add to all items for use of brushes and ladders etc.

Preparing new wood surfaces and priming etc., vide item (*a*) above if properly done costs about 1.5 times that of one coat of painting.

(*e*) *Oiling* with double boiled linseed oil :

1st coat—6 litres oil ;
2nd or subsequent coat—5 litres oil.
Painter—2 : Cooly boy—1 for oiling one coat.

(*f*) *Varnishing* with Copal varnish :

1st coat—8 litres varnish ;
2nd and subsequent coat—6.5 litres varnish.
Painter—4 ; Cooly boy—2 for varnishing one coat.

(*g*) *Coal tarring* :

One coat—10 litres ; Two coats—18 litres.

(*h*) *Solignum painting* :

One coat—10 litres ; Two coats—17 litres.

(*i*) *Bitumen painting for Roof Surfaces or Damp-proof Courses, etc.* :

Bitumen—120 kg ; Sand—0.50 cu. m.

(*B*) **Painting Iron Work** : Amount of paint and labour required for a primer coat and other coats is about the same where red oxide paints are used. Cost of preparing iron surface for painting is about half that of preparing a wood surface.

Additions to get equivalent plain areas for painting done over irregular surfaces

(Based on C.P.W.D. Schedule of Rates)

Wood work : Doors, Windows, etc.

1.	Panelled or framed and braced	1.125	(for each side)
2.	Ledged and battened or ledged, battened and braced	1.125	„

3. Flush	1	(for each side)
4. Part panelled and part glazed or gauzed	1	,,
5. Fully glazed or gauzed	0.5	,,
6. Fully venetianed or louvered	1.5	,,
7. Trellis or Jafri work	2	(for painting all over)

Steel work : Doors, Windows, etc.

1. Plain sheeted steel doors	1.125	(for each side)
2. Fully glazed or gauzed	0.5	,,
3. Part panelled and part glazed or gauzed	1	,,
4. Corrugated sheeted	1.25	,,
5. Rolling shutters	1.25	,,
6. Collapsible gates	1.5	(for painting all over)

General work

1. Expanded metal, grill work and gratings, balustrade railings	1	(for painting all over)
2. Fencings and gates including standards, braces, rails, stays	1	,,
3. Corrugated iron sheeting in roofs, side claddings, etc.	1.14	(for each side)
4. A.C. corrugated sheeting	1.20	,,
5. A.C. semi-corrugated sheeting	1.10	,,
6. Nainital pattern roofing with plain sheets	1.10	,,
7. Ditto. with corrugated sheets	1.25	,,
8. Wire-gauze shutters including painting of wire-gauze	1	,,

Measurements shall be taken flat (not girthed) over all. In the case of doors and windows, the measurements include chowkats or frames.

12. ROOFING

Jack Arch Roofing—Per 10 square metres

(*a*)	9" bricks laid 4½" thickness of arch	—600 nos.
	Mortar (wet)	—0.27 cu. m
(*b*)	IS bricks laid 10 cm thickness of arch	—530 nos.
	Mortar (wet)	—0.18 cu. m
	Take labour for archwork	
	Concrete in haunches, 12 mm aggregate	—1.50 cu. m

Add for items according to the design :

R.S. Joists : Tie rods, 20 mm dia ; Angle iron wall plates, 75×75×6 mm ; Heads and nuts for tie rods ; Washer plates, 50×50×6 mm ; Cement concrete bed blocks under joists ; Tiles on top of concrete and pointing or other roof finishes ; Plastering or pointing and white washing under arches (soffit) ; Centerings for arches, etc.

Mangalore Tile Roofing on Wooden Battens—Per 10 square metres

Tiles—160 nos.	Carpenter—1
Wooden battens—50×40 mm—0.06 cu. m	Tile layer—1.5
Add for hire of ladders, etc.	Mazdoor—1.5

Corrugated Galvanized Iron (CGI) Sheet Roofing—Per 10 square metres

(15 cm end laps and two corrugations side laps) CGI sheets—22 gauge (0.80 mm thick)—12.8 sq. m. Seam bolts and nuts—25×6 mm size—30 nos.

J or L hook bolts and nuts—8 mm dia.—25 nos.

Bitumen and limpet washers—55 nos.

If sheets are fixed with screws and washers :

Screws 6 mm dia., 70 to 80 mm long—45 nos.

(See specifications in Section 11)

Smith—1 ; Mazdoor—1

Add for sundries and scaffolding etc., Woodwork and ridge is extra. (See under "A.C. Sheet Roofing.")

Corrugated Asbestos Cement Sheet Roofing—Per 10 square metres

A.C. sheets 7 mm thick—11.5 sq. m

J or L hooks or crank bolts and nuts 8 mm dia.—25 nos.

Bitumen and flat washers—25 nos.

Carpenter—1 ; Mazdoor—1

Add for ridge according to length where required and smith or carpenter and mazdoor at 1/24 each per metre run. No extra bolts or screws are required for fixing the ridge.

Single Allahabad Tile Roofing on Wooden Battens—Per 10 square metres

Flat tiles (inclusive of wastage)	—120 nos.	Carpenter—1
Half round tiles, ditto.	—120 nos.	Tile layer—1
Cement-lime mortar	—0.25 cu. m	Mason—1/4
Iron screws with washers, 50 mm	—1 doz.	Mazdoor—2.5

Add for ladders, also Wooden battens, 50×40 mm—0.06 cu. m

13. Water Assessment for Building Operations

Brickwork, masonry, concrete	2250 litres/cu. m
Plaster	85 litres/sq. m
Pointing	60 litres/sq. m
White-washing per coat	3 litres/sq. m

14. Burning Lime Stone in Kiln

10 cu. m of limestone or kankar when burnt will produce about 8 cu. m of burnt material. and 10 cu. m of burnt material will produce about 8.20 cu. m of ground lime.

Limes have varying properties. See Section 12.

15 to 20 quintals of coal or coal dust, or 40 to 55 quintals of fairly dry wood will burn about 10 cu. m of limestone. (This quantity is very variable.)

15. BRICK BURNING (Brick Kiln)

About 20 tonnes of coal is required to burn one lakh bricks in a Bull type kiln. Quantity of coal required for burning is very variable and depends upon the climate, soil, moisture, and the sub-soil water level. In dry climates, where the sub-soil water level is low, 13 tonnes of coal has been found sufficient. Where the sub-soil water level is high, a quantity as high as 25 tonnes may be required. Sandy soils also need more coal.

Where wood is used instead of coal, the quantity varies from 20 to 33 tonnes depending on the wood. If the wood is made into small pieces, lesser quantity will be required. Tamrind or Babul is the best wood for burning.

About 5.5 tonnes of coal is required for starting the firing of a 14-wall kiln and about 9 tonnes for a 24-wall kiln. For top firing good coal dust is required. Where wood is used the quantity required is about 12 to 14 tonnes for a 14-wall kiln. About 1.5 tonnes of wood is required for first firing of a small kiln.

Slag or dust (not very fine) of steam coal is considered best for kilns and need not be of a very good quality. But the percentage of first class out-turn is greater with wood than with coal dust. Coal with 'shining' surface is preferred.

About 2.8 cu. m of clay and 0.08 to 0.13 cu. m of sand are required for making 1000 bricks (IS).

It is not economical to burn less than 6 to 7 lakhs of bricks in a single kiln. The kiln is generally eliptical in shape. A typical economical size is :—

Length—61 metres, Width of each trench—8 metres

Gap between trenches—4.6 metres

(This will give a radius for a semi-circle of 10 metres)

Depth to have 16 to 18 layers.

A small size circular kiln with 3 lakhs capacity is :—

Outer dia.—22.25 m ; Inner dia.—14.63 m ; Trench—7.62 m wide ; Depth—2.13 m.

Size of Chimney :

Bottom—244 cm × 53 cm. Height of lower part—244 cm

Width at top of the lower part—115 cm (244 cm reduces to 115 cm.)

Height of upper part—6.7 m for big size ; 4.57 m for small size.

Total height of chimney is 10.67 m. When two chimneys are used they are 7.62 m high.

Top size—38 cm.

Lower part of the chimney is generally made of 12 gauge sheet and can be used for two kilns, while upper part is generaly made of old tar drums. Chimney is mounted on a cast iron base plate on wheels the axles of which radiate so that they travel round the kiln wall.

Approximate material and labour required for manufacturing 10 lakh bricks :

***Materials* :**

Moulding boxes	...	20 nos.
Flat wooden pieces for above		20 nos.
Sand ...	...	3 cu. m
Chimneys with drum and sheet		2 nos.
Feed hole covers	...	36 nos.
C.I. sheets 2.13 m long	...	18 nos.
Fuel wood for first fuelling	...	4 quintals
Steam coal ,,	...	8 tonnes
Dust coal ,,	...	4 tonnes

Earth should be kept heaped up for seasoning for 45 days before moulding. 7 to 15 days are required for drying.

***Sundries* :**

Cost of land ; Royality for taking earth ; Carriage of water and cost of tube well, etc. ; Drums ; Tins ; Baskets ; Rope for chimney 90 m ; Bamboos.

***Labour* :**

Moulders	...	...	5 per 1000 bricks
Loading in kiln	...	...	3 ,,
Unloading and stacking		...	2.5 ,,

4 firemen, 2 labourers, 3 loaders, 1 mate and 1 Mistry, all for 3 months.

16. Surkhi Burning

About 0.36 tonne of coal is required to burn 10 cu. m of *surkhi*.

17. Charcoal Burning

100 quintals of fuel-wood make about 20 to 25 quintals of charcoal. Wood is heated with a limited supply of air.

18. Blasting

In small blasts 1 kg of dynamite will loosen about 4 cu. m of rock, and in large blasts about half of this. It will need about 2.13 m of boring, half mazdoor, 1.83 m of fuse and 3.5 detonators. One man can bore 125 to 250 cm per day in granite and 750 to 1000 cm in lime-stone.

19. CARRIAGE OF BUILDING MATRIALS

Material	Load for a two-bullock cart on kachha roads	Load for a six wheeler motor truck on metalled roads
Bricks	200 to 300 nos.	2000 nos.
Brick ballast	0.50 to 0.85 cu. m	4.00 cu. m
Cement	12 to 15 bags	100 bags
Earth or soil	0.50 to 0.80 cu. m	4.25 cu. m
Gravel or bajri	0.50 to 0.80 cu. m	3.50 cu. m
Iron	0.50 to 0.75 tonnes	6 tonnes
Kankar	0 50 to 0.80 cu. m	4.25 cu. m
Lime-slaked	0.50 to 1.00 cu. m	4.25 cu. m
Moorum	0.50 to 0.80 cu. m	4.25 cu. m
Sand, surkhi, etc.	0.50 to 0.80 cu. m	4.25 cu. m
Stone ballast	0.35 to 0.70 cu. m	3.50 cu. m
Stone boulders	0.30 to 0.50 cu. m	4.25 cu. m
Tiles-Mangalore	250 to 275 nos.	2500 nos.

(*i*) A bullock cart will carry about 1/3rd more load on a metalled road, and a motor truck will take about 1/3rd less load on a kachha road.

(*ii*) The loads given are only approximate and will vary according to the carriage capacity of the vehicle.

(*iii*) The average load for a two-bullock cart may be taken about 0.75 tonnes for short distances and 0.50 tonnes for long distances (say about 8 km).

(*iv*) Loading, unloading plus traction time should be taken for calculating the number of trips. A truck takes about one hour to load bricks and about 3/4 hour to unload with 4 to 6 labourers on job.

(*v*) The average speed for a bullock cart may be taken as 3 km on kachha roads and 3.5 km on metalled roads per hour, and it takes about 3/4 hour for loading and unloading.

20. PREPARATION OF PROJECT ESTIMATES

Papers to be submitted with a project estimate for a work will generally consist of the following :—

(*a*) A *report* detailing scope of the work.

The report should contain the following :—

Reference of the authority ordering preparation of the estimate and other correspondence on the subject.

Previous history of the case and the work ; scope of the work ; cost how to be financed. The object to be gained by the excution of the work ; explain any unusual features which require elucidation.

The reason for adoption of the project or design in preference to others.

Availability of material and labour ; agency for work and time for completion.

(*b*) *Specification and Design*

Reasons why particular design chosen.
Attach calculations for the design.
Relative costs of materials.

(*c*) *Estimate* giving detailed statement of measurements and quantities.

A general abstract of cost.

In some departments rates are based on plinth areas or cubic contents of similar works previously built, but estimates framed on the basis of analogies from existing works are not very reliable and before this method is adopted the correctness or otherwise of the analogy should be curefully tested for selected portions of the works. In particular, analogies drawn from small works should never be relied upon for the preparation of large projects.

The total estimated cost should be given separately for each item.

Provision should be made in the estimates for all incidental expenditure which can be foreseen and also the following items where necessary:—

Cost of land, sheds for stores or hiring of godowns, huntments for workmen, pumping of water, etc.

Make the usual provision of 5 per cent for contingencies on the estimated cost of the work for unforeseen items.

Provision for workcharge or supervisory establishment and tools and plants.

Inclusion of lump sum provisions should be deprecated and costs should be worked out in detail as far as practicable.

Include for any temporary works or serveys to be done.

Rates :

Base the estimates on your departmental Schedule of Rates and where any of the rates are not available in the schedule, make your own rates based on the schedule items and attach analyses of the new rates with the estimates for check. Justify all rates which are higher than schedule rates. Where rates are based on actual market rates, add 10 per cent for contractor's profit.

Drawings :

Make line drawings or detailed drawings as required on suitable scales. Site plans will also be essential. Show North line on all site plans.

Index plans of 1.5 cm to 1 km will be required for most of the projects.

(More details are given at the end of each Section.)

Table of Daily Wages

Rs. Per Month	28 Days		29 Days		30 Days		31 Days	
	Rs.	P	Rs.	P.	Rs.	P.	Rs.	P.
1	0	04	0	04	0	03	0	03
2	0	07	0	07	0	07	0	06
3	0	11	0	10	0	10	0	10
4	0	14	0	14	0	14	0	13
5	0	18	0	17	0	17	0	16
6	0	21	0	20	0	20	0	19
7	0	25	0	24	0	23	0	22
8	0	29	0	27	0	27	0	26
9	0	32	0	31	0	30	0	29
10	0	36	0	34	0	33	0	32
11	0	39	0	38	0	36	0	35
12	0	43	0	41	0	40	0	39
13	0	46	0	45	0	43	0	42
14	0	50	0	48	0	47	0	45
15	0	54	0	52	0	50	0	48
16	0	57	0	55	0	53	0	52
17	0	61	0	59	0	57	0	55
18	0	64	0	62	0	60	0	58
19	0	67	0	65	0	63	0	61
20	0	71	0	69	0	67	0	65
25	0	89	0	86	0	83	0	81
30	1	07	1	03	1	00	0	96
40	1	43	1	38	1	33	1	29
50	1	79	1	72	1	67	1	61
60	2	14	2	07	2	00	1	94
70	2	50	2	41	2	33	2	26
80	2	86	2	76	2	67	2	58
90	3	21	3	10	3	00	2	90
100	3	57	3	45	3	33	3	23
150	5	36	5	17	5	00	4	84
200	7	14	6	90	6	67	6	45
250	8	93	8	62	8	34	8	06
300	10	71	10	34	10	00	9	58
350	12	50	12	06	11	67	11	19
400	14	29	13	79	13	33	12	90
450	16	08	15	51	15	00	14	51
500	17.	86	17	24	16	67	16	13
550	19	65	18	96	18	34	17	74
600	21	42	20	68	20	00	19	16
650	23	21	22	40	21	67	20	77
700	25	00	24	13	23	33	22	48

SECTION 21

MISCELLANEOUS SPECIFICATIONS & GENERAL TECHNICAL INFORMATION

Page

Athletic Fields

Billiard :

Size of Table	Size of Room
2.44×1.22 m	5.18×3.96 m
2.74×2.70 m	5.79×4.27 m
3.05×1.52 m	6.10×4.57 m
3.66×1.83 m	7.31×5.49 m

Croquet Lawn :

Size of Room	32×25.6 m

Ball Room :

Allow a minimum floor area of 1.5 sq. m for each person exclusive of ante room.

Deck Tennis :

Singles size	12.2×3.66 m
Doubles size	12.2×5.49 m

Badminton :

Singles size	13.4×5.18 m
Doubles size	13.4×6.10 m

Basket Ball :

Max. court	28.6×15.2 m	
Ideal court	27.4×15.2 m	22.6×12.8 m
For Schools	25.6×15.2 m	18.3×10.7 m

Black-Boards for Schools

Heights :

Primary class room	53 cm	from floor to base of black-board
Elementary class room	64 cm	
High school class room	76 cm	

Height of Black-Boards 1.06 m, 1.22 m, 1.37 m, 1.22 m is best.

Specifications :—Under coat of 12 mm cement plaster of 1 cement, 2 sand and 1 charcoal powder. Finishing coat of 1 cement and 1 charcoal, 3 mm thick.

Paint : Dissolve 1/2 kg of shellac in 5 litres of methylated spirit and add 1/2 kg of ivory black, 75 grams of finest flour emery and 1/4 kg of ultramarine blue. Mix and put in stoppered bottles. Shake well when using.

(See also under "School Buildings" in the pages following.)

Churches

Size :

Accommodotion including passages, communion table etc., for every person	0.46 to 0.65 sq. m
Width of pews ..	84 to 91 cm
Length of seats for each person ...	46 to 61 cm
Height of seats from floor ...	46 cm
Width of seats ...	33 to 38 cm
Book boards, height from floor ...	81 cm

Cinemas, Theatres and Public Assembly Halls—Design Requirements

(*i*) For approximate seating capacity in cinema halls allow 0.55 to 0.65 sq. m per person inclusive of passages with a minimum of 0.45 sq. m per person exclusive of passages when seats are arranged in straight rows. This will be 0.65 to 0.75 sq. m per person inclusive of passages when seats are arranged on a curve. Smaller dimensions apply to larger rooms. For small concert halls or narrow rectangular rooms, 0.55 sq. m per sitting will usually be sufficient allowance.

While rectangular halls have certain accoustical advantages, the fan and horse-shoe shapes have the added advantage of drawing the audience nearer to the stage, reducing thereby the length of the hall. The side walls should preferably be arranged to have an angle of not more than 100 deg. with the curtain line.

The minimum height of an auditorium shall be 6.0 m. The volume per person should be 3.5 to 4.5 cu. m.

The seats should preferably be staggered sideways in relation to those in front so that a spectator in any row is not looking directly over the head of the person in front him.

If steps have to be inserted in a gangway or passage, there shall be not less than three steps at any one place.

The minimum headway of a passage under the landing shall not be less than 2.1 m at any point.

(*ii*) The area of all doors, windows and ventilators abutting on an open air space shall not be less than 1/5th of the total floor area. If exhaust fans are installed or the hall is air-conditioned the ventilating area can be reduced.

(*iii*) The maximum rake of the floor of an auditorium shall not be more than 1 in 20. Gradually rising towards the rear. Where possible, gradients or inclined planes shall be used instead of steps, which shall not be steeper than 1 in 10.

(*iv*) The width of seats (with arms) shall be between 48 and 56 cm. The back to back distance of chairs shall be 85 to 106 cm. There shall be an intervening space of at least 35 cm between the back of one seat and the front of the seat immediately behind measured between perpendiculars. Also recommended the clear distance between the backs of two successive seats should be 90 cm for seats with fixed backs and 80 cm for seats with rocking backs.

Where chairs are set on steps in balconies, the steps are made 70 to 80 cm wide and 40 to 50 cm high. Front and rear steps are made 90 cm wide, and the front step may be lesser in height. These steps are sometimes made 90 cm wide and 35 cm high.

(*v*) Gangways and passages should not be more than 6 m apart and no seat to be more than 3.8 m from a gangway or passage.

(*vi*) A gangway or passage must be at least 1.2 m wide. The minimum headway of a passage under a landing shall be not less than 2.1 m.

(*vii*) The height of the balcony ceiling or the gallery shall not be less than 3.7 m from the floor of the auditorium and the projection of the balcony into the hall should not be more than 3 times the clear height of the opening of the balcony recess.

(*viii*) The maximum slope of a balcony or gallery shall not exceed 35 deg. The elevation of the balcony seats should be such that line of sight should not be inclined more than 30 deg. to the horizontal.

(*ix*) In the case of a cinema the farthest seat shall not be more than 45 m away from the screen. (Farthest seat for natural voice is 24 m.)

(*x*) The angle of seating shall not be less than 60 deg. with the screen. The front row shall not be nearer to the screen than half the width of the screen.

(*xi*) The position and height of the screen shall be so regulated that the maximum angle of the line of vision from the front seat (eye level taken at 1.7 m from ground level) to the top of the screen shall not exceed 35 deg. The level of stage is kept 0.9 to 1.2 m from the lowest floor level.

(*xii*) No corridor leading to any staircase or exit passage shall be less than 1.5 m in width.

(*xiii*) Entrance and exit doors shall be provided of size and less than 1.5×2.1 m at the rate of one door for every 250 persons and one for any less number in excess provided that from every upper floor or gallery, there shall not be less than two exits. All such doors shall open outside.

(*xiv*) Two independent staircases shall be provided for excess to the gallery or auditorium if on the upper storey and such stairs at no place shall be less than 1.5 m clear in width. No staircase shall have a flight of more than 15 steps or less than 3 steps, and the tread of each step shall be not less than 30 cm and rise not more than 15 cm.

The farthest point in a cinema building from where the staircase affords access, shall not be more than 20 m distance from such staircase.

Common Names of Chemical Substances

Aqua Fortis	... ...	Nitric Acid
Blue Vitriol	... ...	Sulphate of Copper
Chalk	... ...	Carbonate of Calcium
Common Salt	... ...	Sodium Chloride
Green Vitriol	... ...	Sulphate of Iron
Galena	... ...	Sulphate of Lead
Lime	... ...	Calcium Oxide
Limestone	... ...	Calcium Carbonate
Nitre or Saltpeter	... ...	Nitrate of Potash
Oil of Vitriol	... ...	Sulphuric Acid
Plaster of Paris	... ...	Calcium Sulphate
Quicklime	... ...	Calcium Oxide
Red Lead	... ...	Oxide of Lead
Rust of Iron	... ...	Oxide of Iron
Sal Ammoniac	... ...	Chloride of Ammonium
Salt of Tartar	... ...	Carbonate of Potassium
Slaked or Hydrated Lime	...	Calcium Hydrate (Hydroxide)

Soda	... ...	Hydrate of Sodium
Soda-ash	... ...	Sodium Carbonate
Spirit of Salt	... ...	Hydrochloric or Muriatic Acid
Vermilion	... ...	Sulphide of Mercury
Vinegar	... ...	Acetic Acid (dilute)
Water Glass	... ...	Sodium Silicate Solution
Washing Soda	... ...	Sodium Carbonate
White Presipitate	... ...	Ammoniated Mercury
White Vitrol	... ...	Sulphate of Zinc
Whiting	... ...	Calcium Carbonate

For provision of "Sanitary Fittings" see under "Drainage" page 16/48.

Public lecture halls and auditoriums may have a volume of 2.8 to 3.7 cu. m per person.

If the hall is rectangular in shape, its width should be 16 to 24 metres. The length of the hall should not be more than twice the width. Long halls are prone to long delayed sound reflections from the rear walls. The ceiling height should be about 1/3rd to 2/3rd of the width of the room – the lower ratio for very large rooms and the higher ratio for small rooms.

(Based generally on IS:4878).

Seats should be raked and staggered so that there is a clearance of about 10 cm between the sight lines from two consecutive rows. In an auditorium the angle of elevation of the inclined floor should not be less than 8 deg.

Docks and Marine Works

Dock—An artificial basin (water tank) for the use of vessels (ships).

Dockwall or Quaywall—A marginal wall on a wharf or pier.

Dry Dock—A dock from which water can be excluded.

Wet Dock—A dock in which vessels remain afloat while loading and unloading. It is often provided with gates to retain water at low tides.

Wharf—A structure at which vessels may load and unload cargo and passengers.

Marginal Wharf or Quay—A wharf parallel to the shore.

Pier—A wharf projecting from the shore.

Sea wall or Bulkhead wall—Retaining wall for a marginal warf.

Slip—Space between two piers ; sometimes called "dock".

Breakwaters : (Sea and River Protection Works)

Breakwaters are a sort of "bund" built out from the banks into sea or deep waters to form a basin to protect the shore from wave actions. A breakwater should preferably form a converging angle of intersection not greater than 60 deg. A simple form of breakwater is the rubble mound and can be constructed by dumping rubble, riprap, etc., into the sea. The sides are allowed to be formed to get a natural slope. The interior is built with smaller rip-rap and the sea side of the breakwater with heavier and massive stones. In deep waters smaller stones are used than at higher level as the wave action is less in the former case, and (as a matter of safety) massive stones or concrete blocks are used on the sea side and above the mean sea level, the slopes and faces are paved with blocks of stone or concrete. The stones used should be well graded. Stones of 1/2 kg weight to be about 3 per cent and stones of 25 kg to be not more than 12 per cent, and 1 tonne stones not more than 50 per cent. Steel dowels are used to prevent the large blocks from sliding. The top width of the mound should not be less than the depth of water. The base width should be at least 7 times the depth of water. The slope at the harbour side should be 1 to 1½ to 1 to 2. If a wall is to be constructed over the top of this breakwater it should be allowed to settle for about 2 to 3 years.

EARTHQUAKE-PROOF BUILDINGS' DESIGN

No reliable means of predicting an earthquake as to the time of occurrence, place of occurrence or its **intensity** have yet been discovered. It is, however, considered that in the localities where earth-quakes have occurred once they are likely to re-occur again at a future date. An earthquake map of India has been prepared by the Meteorological Department showing zones liable to severe, moderate and minor earthquakes.

Earthquakes consist of vertical and horizontal wavelike motions of the ground. The horizontal motions are much greater than the vertical—from five to ten times and may be in any direction. The most destructive force is therefore the horizontal motion.

General Principles of Design

Buildings should be as light as engineering considerations and considerations of health and comfort permit. Continuity and lightness of structure are of more importance than thickness of walls or low height. The maximum height of any building shall not exceed 27 m. Excessive length in proportion to width is undesirable. A square or a compact rectangular plan should be adopted. A ratio of length to breadth should not exceed 3 to 1 normally. A closed shape, square or nearly so, is preferably to U or L shapes.

All parts of a building should be firmly tied together and stiffly braced at corners in such a manner that the whole structure will tend to move as a unit. Parapets, cornices, contilevers and projections exceeding 75 cm should be avoided. Chimneys should be of RCC or metal and well tied with the main structure. Exterior bearing walls and other walls of masonry should be adequately tied together at the level of each floor line from outside wall to outside wall of the structure by continuous metal rods or RC ties of adequate strength and should be tied to all intervening partition walls. Cement mortar should be used in all masonry structures. A rigid structure comes to rest very quickly and is preferable to a flexible structure.

In the case of a rigid frame with rigid joints made by means of knee braces and gusset plates, the horizontal girders by means of transverse walls serve to stiffen the buildings at the expense of adding to the bending moments of the horizontal girders and vertical columns. Diagonal braces are most effective in increasing the rigidity of the structure.

(*i*) The centre of gravity of the whole structure should be as low as possible. Symmetry in the arrangement of cross-walls is desirable. The

overturning moment due to seismic forces of the structure shall not exceed 50 per cent of its moment of stability both calculated using the same loads.

(*ii*) Adjacent buildings or parts of the same building differing in mass or stiffness shall be separated by a sufficient gap or distance to prevent hammering of one another, or must be rigidly inter-connected. The width of the gap wall shall be at least 10 cm for single storey building and 20 cm for double storey buildings and should be filled with some fragile material which can fracture in an earthquake without causing any damage. The separation should be carried down to the top of the foundation which may be continuous for the entire group.

(*iii*) The maximum foundation pressure under dead and live loads combined with seismic forces shall not exceed by 10 per cent of the normal safe bearing pressure allowed. In the Codes of some countries it is provided that where the allowable bearing pressure is 10 tonnes/sq. m, the foundation pressure may exceed by 10 per cent, and where the bearing pressure is 20 tonnes/sq. m, by 15 per cent, and where it is 40 tonnes/sq. m the bearing pressure may exceed by not more than 33 per cent. For the live load is taken the transient live or floor loads equal to one-third of the equivalent dead floor load which the floor is designed to bear, subject (except in the case of store-rooms) to a minimum allowance of 100 kg/sq. m.

(*iv*) Every building and every portion of a building exterior and interior should be so designed and constructed as to withstand the bending moments due to a continuously applied horizontal force in any direction equal to 10 per cent or less in minor earthquake regions to 20 per cent in severe earthquake regions (according to the intensity of earthquakes expected) of the weight of the building inclusive of live and dead loads combined at the plane under consideration. For the live loads, the superimposed or live loads on roofs may be neglected and for floors in residential buildings they may be taken equal to only 1/3rd of the load for which the floor is designed with a minimum of 100 kg/sq. m, and 2/3rd for assembly halls, schools, and for warehouses full load shall be taken. In multi-storey buildings the structure is tested at each floor for the horizontal forces. For elevated water tanks, chimneys or other tower supported structures not supported by a building, a higher co-efficient for horizontal seismic force is taken, which may be half or equal to the weight of the projecting part of the structure according to the rigidity and importance of the structure and the intensity of the seismic forces expected. Both wind and seismic forces shall be investigated and the higher value taken in calculating stresses. Wind loads when less than the loads due to the above horizontal seismic forces are neglected. Horizontal shear should also be checked.. The stresses are computed in a similar manner as for the determination of wind stresses. The design should be such that the moments due to seismic forces do not cause an uplift in any part of the foundation. The horizontal earthquake force increases with the mass of the structure ; therefore the lighter the material used, the smaller the horizontal force. Roofs and upper storeys of buildings in particular should be designed as light as possible.

Working Stresses. An increase of 33.3 per cent of the allowable working stresses (under combined vertical and horizontal forces including seismic forces) over those for vertical loads alone, in the case of buildings including columns etc. with a framework of steel, and an increase of 25 per cent for RC framework, provided a section shall not be less than that required for ordinary dead and live loads, be permitted.

Foundations. It has been observed that earth movement is of more destructive character on soft, mobile or unstable ground than on rigid ground in the same vicinity. Loose fine sand, soft silt and clay may be considered as unsuitable sub-grades for earthquake resistant buildings. The wave amplitude may be increased several fold in alluvial or marshy material and in lands where the sub-soil water table is high. Common structures built on filled land or underlain by very soft materials suffered more than those on rock hills. But the result is quite different with a strong rigid building built on soft alluvial ground. The elimination of any kind of fixity between the foundations and the soil underneath decreases considerably the amplitude transmitted to the structure. The degree of acceleration generally decreases with increasing depth below the surface.

It has been proved that a 0.9 m bed of sand or gravel decreases the amplitude transmitted above by half, as these materials serve to cushion the blow. If the building rests on piles on soft ground, the soft ground may suffer distortion in common with the piles and so relieve the stresses and motion of the superstructure, but when the piles rest on a rigid sub-strata, the safety of the structure may be impaired. Therefore, a large number of short piles are preferable to long piles driven down to bed-rock. All foundations on piles or soils with less than 20 tonnes/sq. m bearing value shall have footings interconnected with ties in two directions at right angles to each other.

In highly plastic or soft cohesive soils, like silt and some types of clays, and in very loose cohesiveless soils, such as fine sand with very low bearing capacity, the foundation should be carried to a firm stratum by means of piles, piers or wells and where such a foundation is adopted the thicknesses of the soft stratum above the firm stratum should not exceed the smaller width of the area of the entire base of the building. If the thickness exceeds this width the site is unsuitable for an earthquake resistant structure. The heads of piles, piers or wells should be connected by a reinforced concrete slab, or a grillage of beams.

The foundations need not be deeper than made under normal conditions. Overturning moments due to earthquake forces increase vertical pressure on the foundations. Footings should be so designed that the pressure on the soil per unit area shall, as far as possible, be uniform under all parts of the building. Eccentric vertical loading will cause additional pressure and the footings with the greatest eccentricity will determine the design bearing pressure for the whole building. All foundations and footings of columns and piers of buildings should be completely inter-connected in two directions approximately at right angles to each other ; it is an important means of securing rigidity and a moving together of the structure as a unit. It is very desirable that the bottom surfaces of all parts of the foundations of one structure should be on the same level, and no stepping should be done. If the ground is not level, the slope shall be excavated to a horizontal plane for the entire area of foundation. Sites near abrupt changes in the surface level or strata should be avoided.

A raft foundation designed to withstand load evenly over its whole surface is a very suitable type but is expensive and may be used for very high buildings or in very poor soils. Continuous inverted T—beam foundations may be adopted where the weight of the building is too heavy to be carried on column footings or the bearing pressure of the ground is low. The beams should be designed to resist direct pressure or compression caused by seismic forces in addition to the bending stresses. For design it is assumed that the loads on the various parts of the building are distributed so that there is a uniform upward pressure on the beams which are considerd as supported by the columns. Any load such as that of a wall directly bearing on the beam is deducted from the upward load on the beam. The depth of the beam is in most cases determined by shear. The flanges are designed as cantilevers. Care should be taken to see that the reinforcement bars are small enough to develop their strength in bond and that the thickness of the flange is sufficient to resist shear and compression due to bending. For small buildings it will be found that a rectangular beam will give sufficient bearing surface, for heavy structures it is usually more economical to adopt the invertd T—beam section. When beam foundations are used, the whole area for the foundations should be excavated.

Isolated footings inter-connected may be adopted. The design of footings is based on the same principles as explained in Section 6. When the outer edges of the footings are within the normal angle of dispersion of 45 deg., there is no bending in the footings, a nominal reinforcement, say 12 mm bars at 15 cm centres, may be provided. The cross-section of the ties or footings should be 30×30 cm (minimum : 20×20 cm) when built of reinforced concrete and should be reinforced with at least four 12 mm (minimum : 10 mm) dia. rods, two in the top and two in the bottom, one at each external angle, with not less than 6 mm round stirrups spaced not more than 30 cm apart. If the width of the footing exceed 30 cm one similar rod should be added at the bottom for each additional 15 cm width or part thereof of the footings. The reinforcement should have covering of 5 cm. The junction of the tie and the footing should be as low as possible and the bottom of the tie should be on the solid ground. All angles of the footings should be adequately reinforced and if required, diagonal rein-forcements splaying at the corners should be provided. In plan, the junctions of columns and beams are usually splayed at the re-entrant angles in the vertical plane. Each tie shall be capable of resisting in direct compression or tension the seismic force on the heavier of the two footings.

Beam and slab mat (solid raft) is considered to be the ideal seismic foundation in preference to braced inter-connections. The walls of the buildings are supported on continuous beam footings and a slab is used as connecting tie which also serves as the basement floor slab. The slab shall be cast integrally with ribs or beams. The slab should have a thickness of at least 1/48th of the clear distance between the connected foundations with a min. of 15 cm. The reinforcement of this slab should not be less than 0.2 per cent of the gross area of slab in each direction.

The bottom of the slab should not be more than 30 cm above the tops of the footings. The inter-connecting ties and slabs should be built monolithic with the footings and the reinforcing bars should be anchored into the footings. Since the bottom of the slab is above the bottom of the footings the ends of the footing joints are splayed up.

Masonry Buildings of Bearing Wall Construction

All masonry walls, in the case of masonry buildings with bearing wall construction, should have foundations of reinforced concrete, laid at least 30 cm below surface level upon solid ground. All buildings constructed of masonry or hollow concrete blocks should have continuous bands of reinforced concrete carried round all external, party and cross-walls at foundation level, lintel and below the roof slabs or floor joists and at the level of the tie beam or feet of the rafters of the roof. Where RC slab is provided for the roof, the band at eaves level may be omitted in case of one-storey building. The lintels of all openings should be kept at the same level. Such concrete bands should be of the full width of the walls and not less than 15 to 30 cm deep. The construction of a floor slab in the basement forms a most effective tie. Where walls are thickened by piers, the concrete bands should be of the full width of the wall and pier combined. Where rafters, verandahs, or other roofs terminate against walls at intermediate positions between adjoining floors, such walls should have a continuous band reinforced to resist lateral forces. Continuous raking concrete bands should be provided at the top of all masonry gable walls. Such bands should not be less than 15 cm (prefer 23 cm) deep and not less than the full thickness of the top of the wall in width. The steel reinforcement for the bands may be two rods at the top and two at the bottom of 12 mm dia. for 9 in (23 cm) walls, 16 mm dia. for 13.5 in (35 cm) walls, and 20 mm dia. for 18 ins (45 cm) walls, with 6 mm stirrups 30 cm centres.

The height of a masonry wall should in no case exceed 1.5 times the total width of the building at its base. The total height of bearing wall should not exceed 12 m, the height being measured from ground level to the main tie level or the foot of the rafters or to half-way up a gable.

The minimum thickness of walls in unframed buidings for one-storey building may be 9 ins. (23 cm), and for two-story buildings 13.5 ins. (35 cm) in ground floor and 9 ins. (23 cm) in first floor. For buidings more than one storey, the following heights should not be exceeded :

Thickness of wall	18" (45 cm)	13.5" (35 cm)	9" (23 cm)
Storey height	4.5 m	3.5 m	3 m

In no case the storey height of an external wall should exceed 4.5 m except that for a single storey building with trussed roof where a max. height of 6.7 m may be permitted provided the wall thickness is not less than 1/15th the height of the wall. No brick wall should exceed 15.2 m in length between centres of intersecting walls. Unsupported walls of more than 4.9 m length should be stiffened by reinforced brickwork or concrete columns spaced at not more than 2.4 m centres.

Hollow wall constructions (two-leaf walls with air space) are not made in seismic regions.

Practical considerations limit the thickness of reinforced concrete walls to a minimum of 125 mm. A thickness up to 200 mm will usually cover most of the requirements. Walls are generally built monolithic with the floor slabs and may be treated as fixed beams. Vertical reinforcement of 10 mm dia. bars at 30 cm centre to centre and horizontal reinforcement of 6 mm bars at 30 cm centre to centre will be adequate for practically all storey heights encountered. The thickness of concrete hollow block masonry load bearing walls shall not be less than 20 cm for single storeyed buildings and not less than 30 cm for ground floor for two storeyed buildings. In the case of concrete masonry walls of the type of construction mentioned above, the clear height of the wall shall not be more than 18 times the thickness for walls with lateral support both at top and bottom and not more than 14 times with lateral support at top only. Reinforced floors and roof slabs are considered as affording lateral support.

Cross-walls. The thickness of any cross wall shall not be less than 2/3rd of the thickness prescribed for the external wall into which is bonds with a minimum of 23 cm. Intersecting walls should be tied by means of concrete bands at the foundations and at each floor level from the wall intersected to another external or parallel wall on the opposite side of the room or building. Cross walls generally possess greater rigidity than frames extending in the same direction.

Roofs. Arched roofs of any type shall not be permitted. RC slabs shall be reinforced in both directions. All roof trusses should be anchord to the concrete band or top frame. Slotted plates at one end of the steel trusses may be provided to allow for temperature changes. No gable ends should be permitted and ends of all sloping roofs should be hipped.

Openings in Walls

Buildings with open fronts (such as stores, garages, and shops) and many openings in walls lack structural bracings and have often suffered severely in earthquakes. All openings in the walls should as far as possible be placed away from outside corners of the building. Single doors should be used as far as practicable. In external walls the minimum distance from an external corner to the side of any opening should not be less then 1/4th of the height of the opening or 1.5 times the thickness of walls in which the opening occurs, whichever is the greater. The total width of all the openings in any wall should not be more than 50 per cent of the length of the wall, and the combined length of the solid portions in the wall should not be less than 3/8th of the combined height of all the openings, or the wall should be proportionately thickened so that the area of any horizontal section through the wall apart from the openings is the same as if there had been only 50 per cent openings. Auxiliary bars parallel to the sides of the openings and within 5 cm of the openings continuous or extended at least 80 diameters beyond the openings should be provided. Added safety against damage can be obtained by providing additional diagonal bars at least 60 cm long and 12 mm dia. at each side of the corners of the openings. Large window openings should be avoided.

R C Framed Buildings. The load should be taken on the frames and no portion may be considered as transmitted to the walls. The main beams should be reinforced at top and bottom with 0.7 per cent reinforcement on each side, and stirrups provided throughout the length.

The slenderness ratio of columns should not be greater than 15 and the main longitudinal reinforcement should not be less than 1.25 per cent of the effective area. Particular attention must be paid to the tying in of reinforcement at the ends of beams and columns. Floors and roofs should be cast integrally with the supporting beams and the exterior corners should have diagonal braces either of beams or through the provision of diagonal reinforcement in the slabs. The structure should be made as rigid as possible. Construction joints to prevent cracking due to shrinkage or temperature changes in slabes may be provided whenever considered necessary. Wall panels may be of bricks, concrete blocks or any type of special material and the panels should be adequately tied to the frame at points of support. Brick wall panels or hollow concrete blocks should be reinforced with two 6 mm bars at every 6th course for panels in external walls and every 8th course for panels in internal walls for spans up to 3.6 m and for longer spans the reinforcements should be placed after fewer courses. The reinforcements should be tied to the frame at either end.

Staircases

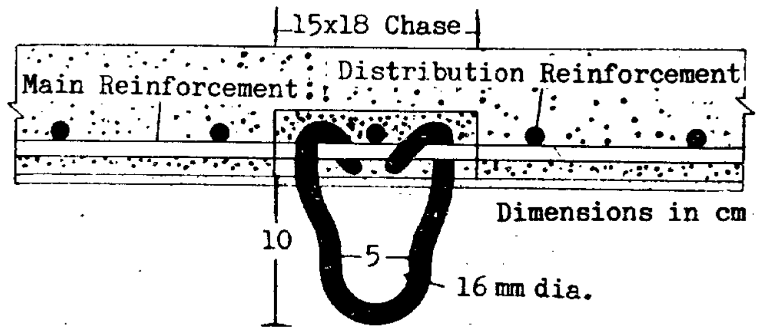
Staircases should, whenever possible, be winding staircases, and in any case the steps should be rigidly connected on either side to the walls or frame of the building. Cantilevered stairs are not advisable in seismic areas. Staircases built in such a manner so as to act as diagonal bracings between the two connected floors have been damaged by earthquake shocks in most of the cases. Therefore, for the interconnection of adjacent floors by means of stairway, either sliding joints should be provided at the stairs so that bracing effect is eliminated or where stairs are built monolithically with the floors the stairs should be enclosed by two rigid partition walls at the stair opening and in such cases joints should not be necessary; this is for when the two flights are made parallel to each other for one storey height.

Timber Structures

Small buildings of one or two storey height with a ground area of not more than 350 sq. m with well braced timber frames on sound foundations can be safely constructed to withstand the stresses. Such buildings should not be more than 7.5 m in height or if two storeyed, not more than 9 m in height. Openings must be kept back, as far as possible, from all corners. It is preferable to make timber structures with masonry foundations. If so constructed, the foundation walls should not be less than 23 cm thick on a footing of 30 cm width for a foundation wall up to 2 m in height for a single storey dwelling. Where the foundation wall exceeds 2 m in height, 12 cm should be added to the wall thickness for every 2 m or part thereof, with a proportional increase in the footings. If the foundation wall is of reinforced concrete, the thickness may be reduced to 15 cm. All corners of walls should be diagonally braced and if the length exceeds 7.5 m additional bracings are necessary.

Buildings in seismic regions must be made fire-proof as fire frequently follows an earthquake and its ravages are often greater than those of the earthquake itself.

Fan Clamps



FIXING FAN CLAMPS IN EXISTING R.C. ROOF SLAB

Fencings and Railings

See also under "Reinforced Concrete"—Section 8 and "Bridges and Culverts"—Section 19.

Line Posts are intermediate posts forming the majority in a fencing system and are intended to carry the fencing wire between the strainer posts.

Strainer Posts and Terminal Posts are corner or end posts or those used at intermediate positions in a line of fence and which are strutted or braced.

Strut or Brace is the member used in inclined position for supporting the strainer post.

Fencing posts may be of the following sizes:

Line Posts: Pipes—50 mm dia.; or L or T—40×40×6 mm.

Terminal Posts: Pipes—65 mm dia.; or L—55×55×6 mm with two stays 40×40×6 mm or L for corner posts and one stay for end post.

Gate Posts: Not over 1.8 m wide single or 3.6 m wide double gate opening:

Pipe—75 mm dia.; or L—75×50×10 mm; or T—60×60×6 mm.

Over 1.8 m wide to 4 m wide single or 3.6 m to 7.9 m wide double gate opening:

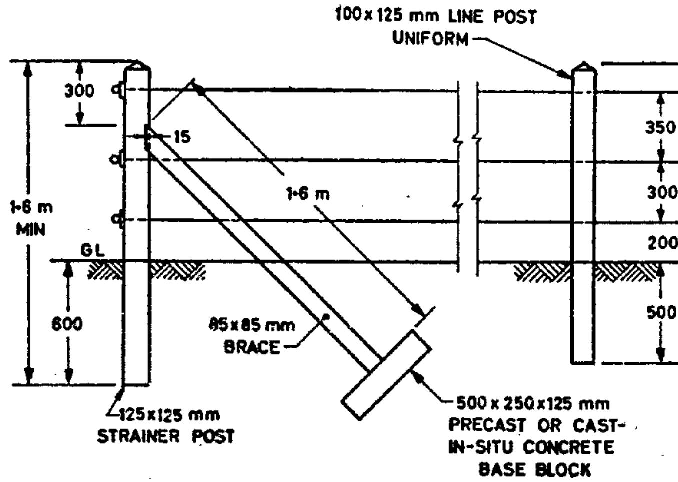
Pipe—100 mm dia.; or L—125×75×10 mm; or T—75×80×6.8 mm.

Wires are fixed to end posts by straining bolts either an hooked or eyed end, complete with washer and nut. The sizes of such bolts are about 230×10 mm: 300×10 mm; 300×12 mm; 350×12 mm; 450×10 or 12 or 15 mm etc. Wires are fixed to line posts by gal: staples of size 12 mm to 45 mm.

Wires generally used for fencing are of different gauges up to 14 SWG; usually those are of 4, 5 and 6 gauge in 7 strand (or 7 ply). Or 4 point 75 mm apart borbed-wire. A coil of barbed wire of 1000 metres weighs about 100 kg.

Welded steel wire fabric to be 0.90 m wide with rectangular mesh of 75 mm × 25 mm size, weighing not less than 7.75 kg/sq. m.

Sal ballies post should have minimum girth of 300 mm, and fixed 2.5 m centre to centre.



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FENCING INTENDED FOR HOMES AND HOUSING ESTATES

Railing Pipes for Embankment, etc.

May be of galvanized (heavy) pipes of 40 to 50 mm dia., in 4 or 5 rows, 33 cm apart. Height above ground level is 0.91 to 1.4 metres. Posts are 1.83 m to 2.44 m apart ; they are sunk into the ground from 45 cm to 75 cm and fixed in lime concrete or lean cement concrete foundaiions. Posts can be of Ts 80 × 80 mm (also called stumps or standards). The foundation block of concrete may be of size 30 × 30 × 45 cm high. Posts at angles can be strutted by bending the second pipe from the bottom and carrying it into the concrete block at the base which will be increased in size by about 60 cm.

Ferro-Type Printing (Blue Prints)

Method of sensitizing paper :

(*i*) Citerate of Iron and Ammonia	...	6.5 grams
Water	...	30 millilitres
(*ii*) Ferric cyanide of Potassium*		
(Red Prussiate of Potash) ...	...	4.6 grams
Water	...	30 millilitres

Make the above two solutions separately and mix at the time of use. The mixed solution should be applied with a sponge on a thick (glazed)

* and not the ferrocyanide of yellow prussiate.

paper and done twice crossways. The paper is hung up to dry. The process should be carried out in a dark room.

About 120 millilitres of the solution will suffice for eoating 10 sq. m of paper.

The vessels in which the solutions are made must be scrupulously clean.

The potassium salt should be broken up fine. The iron salt dissolves very rapidly. It may be kept indefinitely in a solid state if perfectly dry, but it readily absorbs moisture and then becomes sticky and unfit for use and the solution is apt to become mouldy after a few days. The solutions should be prepared and kept in coloured bottles in a dark room in small quantities as required.

The solutions must be dissolved thoroughly and filtered. Take care that no undissolved particles of the red prussiate get into the double solution. It must be rejected when its brown colour changes to bluish green.

If a few drops of strong ammonia solution be added to the citrate solution, until the odour is quite perceptible, the addition of a saturated solution of oxalic acid in water to the double solution will hasten the printing in cloudy weather. 10 per cent. of the oxalic acid solution will increase the rapidity of printing about 2.5 times ; 20 per cent., 5 times ; 30 per cent., 10 times ; but with more than 20 per cent, it is difficult to get clear white lines.

After washing, the application of a solution from 1 to 5 per cent, of hydrochloric acid, or of oxalic acid in water, intensifies the blue colour, and is therefore useful in bringing out pale or under-exposed prints ; but the prints then must afterwards be washed again in pure water.

To erase a (white) line on a blue print, go over the line with the sensitizing solution. This should be done in a weak light. Then expose the entire print and rewash.

Blue colour may be removed showing the white paper beneath by applying a saturated solution of concentrated carbonate of soda (either washing soda or bicarbonate-baking soda, will also answer) or carbonate of potash. If red, instead of white, lines are desired, mix with the soda or potash solution ordinary carmine writing-ink.

White correction lines can also be made on a blue print with the following solution :

Oxalate of Potash	10 grams
Gum solution	40 minims
Water	30 millilitres

Flag-Poles—Wooden

For 10 m to 20 m extending above the roof—diameter at the roof should be 1/50 of the height above the roof and top diameter 1/2 of the lower. (Size for hard-woods.)

Heat Conducting Powers of Substances

Slate	1000	Chalk	560
Lead	5210	Asphalt	450
Brick (av.)	650	Wood	336
Firebrick	620	Cement	200

The following table gives the absorption of heat by different colours when exposed to the Sun's direct rays :—

White	100	Turkey red	165
Pale yellow	102	Dark green	168
Strong yellow	140	Light blue	178
Light green	155	Black	208

Hospitals (Design)

Design for Wards : (See also under "Siting of Buildings and Ventilation" Section7.)

Allow 2.5 m spacings between wall and passage for bed. Allow 3.2 m minimum gangway between beds.

X-Ray rooms require special type of floors, plastering, and also doors and fitting, etc. for which details should be obtained from the Medical Department.

Light

Reflection of Light :

REFLECTION FACTORS FOR VARIOUS MATERIALS AND PAINT COLOURS

	Per cent		Per cent
White	84	Sky blue	30—47
White tile glossy	80	Sea green	38
White-wash	80	French grey	36
White paint	70—84	Brick, yellow, clean	35
Pale cream	73	Golden brown	31
Aluminium paint	72	Light brown	30
Deep cream	70	Blue, (turquoise)	27
Primrose	70	Brick red, clean	25
Lemon	69	Dark battleship grey	15—25
Golden yellow	62	Yellow dirty	20
Light buff paint	61	White-wash dirty	20—40
Light stone paint	58	Middle brown	20
Medium buff paint	54	Grass green	19
Concrete unpainted	45	Post office red	16
Light battleship grey	44	Peacock blue	16
Salmon pink	42	Galvanized iron	16
Yellow clean	40	Dark brown	12

(Thus, apart from the sanitary advantages of white-washing of walls and ceilings, there is a considerable improvement in lighting.)

Reflection Factors of Various Metals :

Silver plate	...	...	92	per cent
Silvered glass	...	...	82—88	,,
Mercury-back glass	...	...	70	,,
Chromium plate	...	...	65	,,
Polished aluminium	...	...	62	,,
Nickel plate	...	...	55	,,

Silver-plated metal reflectors are the most efficient when new but they quickly tarnish. Silvered glass is very efficient. Chromium is free from corrosion and can stand rough usage which would ruin glass reflectors.

Motor Garages

Min. size	4 m (prefer 4.9 m) × 2.7 m	Floor slope 1 in 36
Usual size	5.5 m to 6 m × 3 m to 3.6 m	
Height	2.1 m to 2.7 m	

Size of Motor Pit :

Length	3.6 m	Steps can be made on one side.
Breadth	0.84 m	Bottom drain should be provided
Depth	1.22 m	if possible.
Planks for covering		40 mm thick
Approach Ramp		1 in 8 to 1 in 10

(Dimensions of Road Vehicles are in the Section "Roads and Highways.")

Garrage Doors of Mild Steel sheet.

Frame L—40 × 40 × 6 mm, 3.5 kg/m and two diagonal braces. MS sheet 1 mm thick.

Repairs of Buildings

Removal of Walls in Existing Building to Increase the size of Room

Keeping the existing roof as it is, if the load bearing wall is to be removed, then it is necessary to find out the weight of the roof coming on the wall and also the weight of the wall if any on the next floor. After calculating the weight, the size of the steel girder that could support the weight is decided. It is necessary to verify if the side walls on which the girder would be resting will be able to carry the load from the girder. In case of doubt, it is necessary to romove at least 45 cm width of existing wall at the point of support and erect new brick pillar in cement mortar for a width of 45 cm and thickness equivalent to the thickness of the wall or 35 cm whichever is greater. Before removing the wall the roof should be supported with runners and wooden posts. It is always safer to provide two steel girders of equal size in the place of the wall so that for placing one girder only half the thickness of wall abuting the roof need be broken and the girder placed in position. The next day, the other half thickness of the wall is broken and the second girder is placed in position. This method of working ensures safety to the building.

Repairs to Damaged RCC Roof Slab

Where concrete with plaster below reniforcement of RCC roof has fallen down and the exposed reinforcement has become rusty—this may be due to leaky roof, where water has entered concrete and rusted the reinforcement. Or, it may be because of salty water of sand used in the concrete. Whatever the reason, the best remedy is to provide either wooden or precast RCC battens at 45 cm centres supporting the roof with 23 cm bearing on walls. (The size of the battens should be calculated to take the roof and slab loads.) The roof slab in-between rafter shall be plastered with 1:3 cement mortar 5 cm thick. This will ensure a long life to the roof slab. To prevent further leakage of water, a new weather proofing course shall be laid on the roof slab. Cracks in the slab should be grouted with cement mortar.

Repairs to Leakage in Lavatory Rooms

Where roof slab of a lavatory is found to be leaky, it may be that joints between soil pipes and the accessories are not done properly due to which waste water leaks through the joints. But in many other cases, it is found that the joints are all properly laid and yet the waste water leaks through the roof. On minute observation, it is found that the porcelain P or S traps are cracked, through which the waste water leaks through the roof. The reason for the porcelain trap to get cracked, is that when the waste matter does not pass through the soil pipe properly and the waste gets accumulated in the water closet, the cleaner tries to clean the trap with a wooden stick or an iron rod and during this process the trap gets cracked. The cracked traps should be replaced with new ones.

Repairs to Damaged RCC Columns and Beams

The structure which has cracked, or where reinforcement has deteriorated can be repaired by guniting. Any existing plaster on RCC work is removed and surface roughened with foot marks at least 6 mm deep at close intervals not more than 1 cm centres to form a band for the cement gunite plaster. All cracks are opened out to maximum depth possible in V shape and surface cleaned of all loose mortar and foreign matter. Reinforcement bars are cleaned to remove all scales and rust by wire brushing and by rubbing with emery paper. A coat of neat cement slurry is applied on the existing reinforcement after cleaning it just before the guniting is done.

Guniting is a mixture of cement and sand deposited in the form of cement plaster ranging from 12 to 50 mm thick for a walls and 100 mm for floors, ejected under a pressure of 2 to 3 kg/sq. cm from a 'cement gun'. The cement and sand are mixed almost dry and screened through a sieve, and the mixture is blown dry through a hose and water just sufficient for the purpose of hydration added at the nozzle. A separate hose carries the water to the nozzle under a pressure of 1 to 1.5 kg/sq. cm, and the nozzle man regulates the quantity by means of a hand operated valve.

Repairs to Plaster

The masonry joints which become exposed after removal of old plaster are raked out to a minimum depth of 10 mm in the case of brickwork and

20 mm in the case of stonework. The loose mortar is dusted off and the surface thoroughly washed with water and kept wet till plastering is commenced. Concrete surfaces are thoroughly scrubbed with wire brushes after the plaster had been cut out and pock marked; surface cleaned and thoroughly washed and kept wet till plastering is started.

School Buildings

Design of Class-rooms

The ratio of dimensions of class-room should be about 3 : 2 or 4 : 3. It is recognized that a square is the best area for teaching purposes. The floor area of a class-room per pupil is taken 0.93 to 1.4 sq. m average for primary schools and 1.8 to 2.3 sq. m for high schools. The height of the rooms should be 3.6 m minimum and if the floor area exceeds 55 sq. m, the height should be 4.2 m.

The class-room blocks should be so arranged that the rooms derive light mainly from the north side and no verandahs are provided on that side. Extensions to class-rooms should not be made by adding at right angles to the main block. Two blocks should be separated by a distance of not less than the height of the higher block for light.

For light, the glass area of the class-room should not be less than 1/6th of the floor area and most of the effective light should come from windows in the north wall. The windows should be placed at regular distances so as to ensure uniformity of light. The edge of the last window in the north wall should be behind the last row of pupils and not more than 0.9 m from the west wall. No windows admitting light should be placed so that pupils when seated will be facing them. In rooms where pupils are seated on desks, the height of the window sill should be 1.06 to 1.22 m, but where the pupils are seated on the floor, the height of the window sill should be 0.75 to 0.90 m from the floor. While fixing the width of a class room, it should be kept in view that it is difficult to light efficiently any portion of a room which is more than 7.3 m from the window wall.

Ventilators should be fixed as close as possible to the ceiling 15 to 23 cm, and allow an area of 325 sq. cm of open ventilation per each pupil.

Blackboards should never be fixed on the same walls with the windows used for lighting purposes; and no seat should be more than 9 m distant from the blackboard-take 1.82 to 2.43 m. Distance between rows of seats should be 75 cm, min: 45 cm.

Space between rows and walls to be at least 30 cm.

For sanitary arrangements see "Standards for Public Sanitary Conveniences" in Section 16. For each closet there should be a window with a minimum area of 0.6 sq. m. Doors should be being so as to leave a space of at least 7 cm at top and bottom.

Hostels

In the case of single rooms or cubicles the minimum floor space should be 9 sq. m. Rooms for accommodating 3 or 4 stutdents should provide 6 sq. m of floor space per head and those for 5 or more students, a minimum of 5.6 sq. m per head.

Class rooms should be so planned that the light is admitted from the left. The school building should be so oriented that adjacent buildings do not cut off the light coming through its windows. This distance should be such that the angle of light coming in is never less than 18 deg., *i.e.*, a distance of 9.14 m from an adjacent two storey building.

The Solar System

The earth is an oblate spheroid. That is, it is somewhat flattened at the poles, rather than being a perfect sphere. It revolves around the Sun at 107159 km per hour and spins on its axis at 1670 km per hour (at the equator).

A solar year is the time in which the earth makes one revolution around the sun. Its average time (mean solar year), is 365 days, 5 hours, 48 miniutes and 45.6 seconds nearly, or about 365.25 days. A mean lunar month (or lunation of the moon) is 29 days, 12 hours, 44 minutes, 2 seconds and 5.24 thirds or 29.53 days average, which is the time the moon takes to travel round the earth.

Leap year. If the number of a given year is divisible by 4, it is a leap year and contains 366 days. (In a leap year February has 29 days.) Centuries not divisible by 400 are not leap years. Thus 1896, 1904, 1600, 2000 are leap years, but 1897, 1901, 1800, 1900 are not leap years.

A.M. is before-noon (L. Ante Meridiem).

P.M. is after-noon (L. Post Meridiem).

Equatorial radius of the Earth	6377.4	kilometres
Polar „ „ „	6355.6	„
Circumference „	40064.1	„
Mean distance of the Sun from the Earth	149460000	„
Max: „ „ (1st July)	151165550	„
Min: „ „ (1st Jan.)	146338550	„
Radius of the Sun	683020.5	„
Radius of the Moon	1739.3	„
Distance of the Moon from the Earth	384321.0	„
Length of 1 degree in latitude:		
Equator	110566	metres
Pole	111687	„
Length of 1 degree in longitude:		
Equator	111309	„

One degree of a great circle of the Earth at the equator =96.643 nautical or geographical kilomeres=111.333 statute kilometres.

Speed of sound is taken as 1800 km per second.

Swimming Pools

Size (m)	Depth (m)	Suitable for
15×7.5	Max : 1.7 ; Min : 0.76	Schools : 0.46 depth for Children
23×9	1.8 to 2.1 deep at one end and 1.7 deep at the other end	Average capacity pools
30.5×13	Ditto.	Swimming associations
40×15	Ditto.	Contests and races
50×18	1.8 min : depth and 2.7 min : for diving	Championship

Water area may be taken at 2.5 to 3.3 sq. m per each person using the tank. Or, 1.1 sq. m×max : daily attendance at any one time.

Tanks are made with sloping floors towards the diving side and the depth allowed under the diving board should be half the height of the highest board with a minimum of 2.7 m. Copper railings 40 mm diameter to be provided all round.

Expansion joints are provided at about 12 m spacing.

Toilet Arrangements :

Lavatories :

1 WC and 1 urinal for every 60 men

1 WC for every 40 women

1 basin or every 60 bathers

Dressing Boxes :

1.06×0.91 m to 1.22×1.22 m according to availability of space.

One dressing box per 15 to 20 bathers.

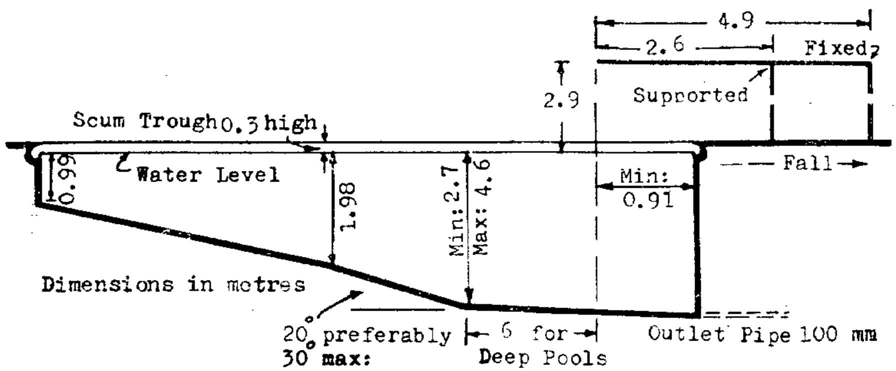
Schools : (Boarding Houses)

Allow for baths	8 per cent. of boarders
Allow for WCs	8 per cent. of boarders
Allow for urinal	4 per cent. of boarders

Purification of Water : Can be done by the addition of bleaching powder or chlorine. The free chlorine actually present in the water of the pool should not amount to less than 0·2 and not more than 0·5 parts per million parts of water. (See under "Water-supply".) Heavily chlorinated pools can cause scratchy feeling in the eyes. The condition usually clears up in a few days without treatment. Cold packs may help reduce the irritation. Some people are more sensitive to chlorine's effects than others.

It has been found that a continuous maintenance of 1 mg. per litre of free chlorine residual provides adequate protection against bacteria and viral agents. The bacteriological quality of water is supposed to be as near the standards prescribed for drinking water as possible.

Walls of the tanks should be designed to resist earth pressure with live load surcharge of 30 cm. Where tanks are built above ground level, there should be enough of earth well rammed all round. Top width of walls to be 45 cm. Foundations of the long walls should preferably be stepped to make up for the sloping floor. Floor can be 15 cm cement concrete over 15 cm of lime concrete ; but must be sufficiently strong to resist the water pressure. There should be expansion joint all round between the floor and the walls (can be 20 mm) filled with bitumen. Scum trough should extent around the whole area of the pool and a scum gutter outlet provided. In small pools where diving boards are not provided, floor will be with one single slope and not as shown in the sketch. Inlet pipe should be at one end of the pool and outlet on the opposite end. The various depths of water should be clearly indicated. There should be a pacca non slip surround of 2.4 m (min:) width all round the pool for open air pools and given outward fall of 1 in 40 ; or, alternatively, a kerb 23 to 30 cm high can be constructed at the edge of the pool in order to prevent surface water draining back into the pool.



Spring Boards :

Are generally 51 cm wide and fixed not less than 1.8 m apart and not more than 2.9 m above water level, except for out-door pools which have boards as high as 9.7 m. The board should project 0.9 m minimum from edge of tank. Where a number of boards are provided one above another, each should project at least 0.61 m further than the one next below it. The international one meter spring board is 4.3 m long and 3 meter board is 4.9 m long. Fixed running boards should be 4.3 m to 5.5 m long or more and should give a clear run of at least 4.9 m. The height of the diving board should not exceed twice the depth of the water. The ladder shall have a max: of 5 steps.

Times

Variation of Local Times

Earth rotates at 15 deg. an hour, or 1 deg. in 4 minutes, therefore local times of two places with distance of 1 deg. east or west differ by 4 minutes.

Vibrations

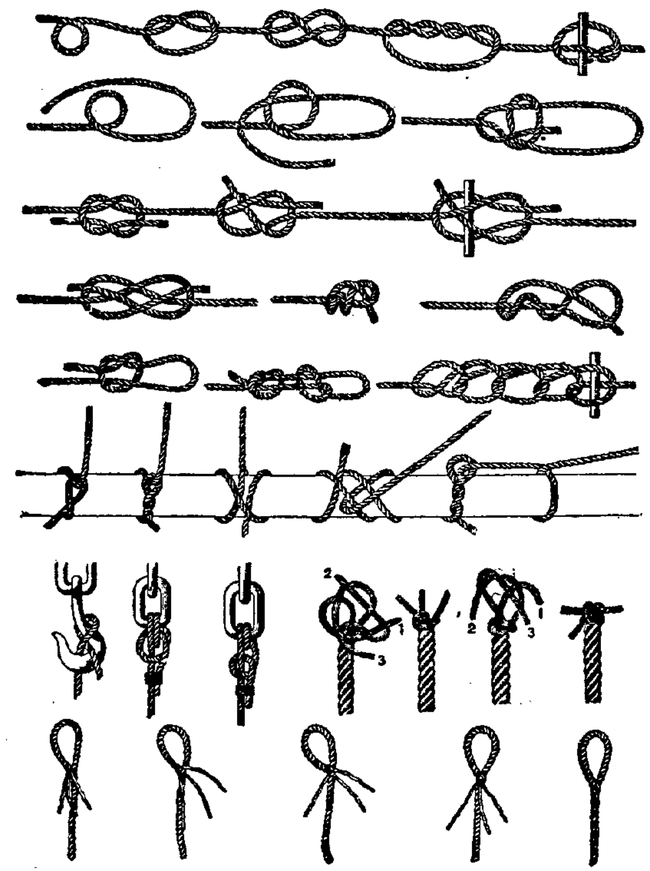
For structures subject to vibrations such as due to working machinery, allowance for dynamic effect can be made by reducing the working stresses by about 20 per cent or by increasing the live and dead load effect by the same amount.

Wire Ropes

Approximate breaking load for flexible steel wire ropes, in tonnes $=1/250\ C^2$; for iron ropes $=1/400\ C^2$; for plow steel ropes $=1/150\ C^2$. C is circumference of rope in mm.

Working load should be only 10 to 25 per cent, of the breaking load. Fastening reduces the working load 60 to 75 per cent.

Various Types of Knots for Ropes



SECTION 22

ELECTRIC SERVICES

TERMS AND MEASURES

AC (Alternating Current): Is a current which alternatively reverses its direction in a circuit in a priodic manner. A complete set of these changes is called a Cycle. The number of times the current goes through these changes during each second is called the Frequency of so many cycles per second.

DC (Direct Current): Is a current flowing in one direction only and of uniform strength free from pulsation.

The *volt* is the practical unit of electric pressure, the force that would carry one ampere of current against one ohm resistance.

The *ampere* (amp.) is the unit of electric current or quantity flowing. A current of one amp. flows when a potential difference of one volt is applied to a resistance of one ohm.

The *watt* is the unit of power and is equal to 1 volt × 1 amp. *watts/volts* = amperes.

The *ohm* is the electrical resistance of a conductor in which a constant current of 1 amp. flows under a constant voltage of 1 volt.

The Board of Trade Unit (BTU) is the kilowatt-hour (k w h), which is the equivalent of 1000 watts taken for a period of 1 hour.

Ohm's Law: The fundamental law in electric circuit theory which states that the current through any circuit element is proportional to the voltage across it.

Example: A 200 volts lamp giving 40 candle-power and consuming 40 watts, requires a current $I = 40/200 = 0.2$ amp.

Ammeter: An instrument for measuring current passing through a conductor (wire).

Armature: Portion of a dynamo in which the electric current is induced.

Commutator: Copper bars at the end of the armature coils which rub against carbon brushes ; current is collected through the carbon brushes.

Series Wiring: A system of wiring in which the same current travels through two or more lamps before completing its circuit.

Short Circuit: An accidental connection between the positive and negative conductors and due to which the current is cut short and does not complete its circuit.

Transformer: An instrument for reducing or transforming a high pressure to a low one by induction.

Indicated Horse-Power: (IHP) is the power developed at the cylinder, as registered on the indicator diagram.

Horse-power is a rate of expenditure of energy.

Brake Horse-power: (BHP) Is the power available at the engine shaft and is the indicated horse-power minus the power dissipated in frictional losses within the engine.

Shaft Horse-power : (SHP) Is the same as BHP, but is the term used for large engines where the output is measured by a torsion meter rather than by a brake.

Nominal Horse-power : (NHP) Is used by insurance and classification societies as a measure of the size of an engine as distinct from its power, for the determination of survey fees.

Mechanical Efficiency : Is the ratio between the work got out of an engine at the shaft and the work put in at the cylinder by the steam = BHP/IHP. The mechanical efficiency varies greatly, usually it lies between 0.75 and 0.95. A mechanical efficiency of 0.85 to 0.90 is a usual allowance ; it increases with the load on the engine.

Electrical HP = IHP × engine efficiency × generator efficiency. (For conversion to Metric, see page 1/14).

One HP motor consumes about 500 watts.

One HP (British) = 33,000 ft. lbs./min. = 550 ft. lbs /sec.
= 0.745 kilowatts = 746 watts = 1.014 metric HP

1 metric HP = 0.986 HP (British)

1 kilovolt = 1000 volts

1 kilowatt = 1000 watts = 1.341 HP = 737 ft. lbs./sec.

1 kilowatt-hour = 1000 watt-hours = 1.341 HP-hours.

1 millivolt = one-thousandth of a volt.

1 milliampere = one-thousandth of an ampere.

1 megohm = one million ohms.

E = IR : To find the voltage, multiply amperage by number of ohms resistance.

W=IE : To find number of watts, multiply voltage by amperage.

W=IR : To find the number of watts, multiply the square of the number of amperes by the resistance in ohms.

To find the number of kilowatt hours, divide the number of watts by 1000 and multiply by the number of hours.

I = Current in amperes ; R = Resistance in ohms ; E = Electromotive force in volts (e.m.f.) ; W = Watts. Z is impedance in ohm.

In a DC circuit R = E/I
In an AC circuit Z = E/I } Ohm's Law

Advantages of AC Over DC :

(*i*) Greater simplicity of dynamos and motors.

(*ii*) Feasibility of obtaining high voltages by means of transformers for cheapening the cost of transmission.

(*iii*) The facility of transforming from one voltage to another, either higher or lower, through transformers.

DC current is necessasy for some industrial processes, such as electroplating, and also for the charging of storage batteries. DC is also required for cinema work. AC is converted to DC through *convertors*. Speed control is simpler with DC than with AC, and DC motors in general operate with less noise.

AC Constant-Potential Systems

There are two systems :

(*i*) Single-phase system. The term phase is used in connection with AC systems only in the sense of circuit. Thus a single-phase system means a system sending out power from one circuit only of the generator.

(*ii*) Three-phase system has three circuits and three or four wires are used. This is the most universally employed system. Domestic consumers are generally given a single-phase service and three-phase service is given to large power consumers.

Current Taken by DC Motors

Approx. full load current in Amperes assuming average efficiency :

HP	Voltage			HP	Voltage		
	110	220	440		110	220	444
0.26	3.0	1.5	—	20	154	77	38
0.50	5.4	2.7	—	25	193	97	48
1	10	5	2	30	227	114	57
2	18	9	4	40	300	150	75
3	26	13	6	50	375	187	93
5	42	21	10	60	452	226	113
6	49	23	13	75	560	280	139
7.5	60	30	15	80	590	295	148
10	80	40	20	100	740	370	195
12	95	48	24	150	—	550	275
15	117	59	29	200	—	—	365

Current Taken by AC Motors

Approx. Amperes per phase taken by modern induction motors, allowing reasonable efficiencies and power factor :

BHP of Motor	Single Phase	Three Phase		BHP of Motor	Single Phase	Three Phase	
	230 Volts	400 Volts	440 Volts		230 Volts	400 Volts	440 Volts
0.25	1.8	0.7	0.6	5	24	8	7.5
0.50	3.5	1.2	1.0	7.5	36	12	11
0.75	4.8	1.7	1.4	10	47	15	14
1	6.2	2.0	1.7	12.5	59	19	18
1.25	7.4	2.5	2.2	15	70	22	21
1.50	8.7	2.8	2.5	20	91	29	28
1.75	10.0	3.2	2.8	30	135	42	39
2	11.8	3.5	3.2	40	183	56	53
2.25	14.0	4.3	4.0	50	227	70	66
3	17.5	5.0	4.5	75	—	104	94
4	20.0	6.5	6.0	100	—	136	125

To find the current taken by each terminal of a three-wire, three-phase AC motor, divide the current taken by a single-phase AC motor of the same size and voltage by 1.73.

Quantity by Heat

The British Thermal Unit (BTU) is the quantity of heat required to raise the temperature of 1 lb. of water by 1 deg. F. Similarly the caloric or the gram clorie (cal.) is the quantity of heat required to raise the temperature of one gram of water by 1 deg. C.

Power Available from Water Falls

HP available in kilowats = QH × 10

Q = discharge of water in cumecs, H = fall of water in metres

Supply Requirements : An estimate is made of the probable *peak demand* (in kw or kVA) and *annual consumption* (in kwh). The whole of the light and other electrical services in a building are never in simultaneous use, so that the maximum demand does not equal the total installed load. This may be taken : Lighting - 1/2 to 2/3 of installed load ; other services - 1/3 to 1/2 of installed load ; for fans, pumps etc., it should be assumed that all the appliances will be in service together. This is called *diversity factor*. The number of units taken in a period, expressed as a percentage of the total number which would have been taken had the maximum demand been maintained continuously throughout the period, is called the *load factor*. The maximum demand and load factor are important in establishing the economics of any installation.

Installation of Small Generating Sets : Sufficient room should be provided all round the set for ease of access. A distannce of about one metre from the set to any obstruction is about the min: Room is required for storage of oil and for a small bench ; shelves for spares are convenient. The room must have ample ventilation to take the heat of the engine from the cooling water into the outside atmosphere. The engine foundation should be a heavy block of concrete which should extend 75 to 150 mm all round the bedplate of the generating set. The block should continue above floor level for the plinth. A suitable height is one that will bring the crank-shaft centre to about 70 cm above floor level. The exhaust pipe should slope downwards from the engine, preferably to an expansion chamber in the floor. Where possible cooling tanks should be in a separate compartment to the engine. Water pipes should not be cemented into the floor, and cables should normally be run in trenches in the floor. The switchboard should be supported about 40 cm for single panel and not less than 75 cm for two or more panels, away from the wall, where it is necessary for an operator to get behind the board. Rails and guards should be provided to ensure prevention of accidental contact with flywheels.

Light and Fans

Candle-power : (CP) is the strength or intensity of a light source, but is not a direct measure of the light output.

Lumen : This is the unit used to measure the rate of flow of light.

Foot-Candle : The measure of illumination, *i.e.*, the useful result obtained from the lighting system.

1 CP = 1.7 to 2 watts for metallic filament lamps.
3 to 3.5 watts for carbon filament lamps, except in the case of gas-filled lamps.

The majority of lamps in general use are of gas filled type ; the tubular lamps are of vacuum type. Vacuum bulbs are cooler than gas-filled ones.

Mercury Lamps. These give a characteristic bluish-green or bluish-white light with noticeably poor rendering of red colour.

Tubular Fluorescent Lamps. Available in three colours—"day light", "warmwhite" and "natural". Due to their greatly increased area of radiating surface compared with the equivalent gas-filled lamps, these lamps are of a much lower brightness, thus reducing the possibility of glare. They give over three times the light at the same time radiating only one-quarter of the heat.

Fluorescent Mercury Lamps. These have a fluorescent powder coated on the inside of the outer bulb to glow with an orange red light and thus improve colour rendering. The surface temperature of fluorescent lamps is about 49 deg. C.

Apart from discharge aad fluorescent tubular lamps, standard type gas-filled are the most efficient and the larger sizes more than the smaller.

Current taken by a lamp in amperes : = Watts/Volts.

A 100 watts lamp consumes about 1 unit of electric energy in 10 hours, or a 10 watts lamp will consume about 1 unit of electric energy in 100 hours.

Ceiling fans can be rated at 100 watts and table fans at 60 watts in the absence of any actual values known.

All glow lamps may be hung at a height of 2.4 m above the floor level and fans 2.7 m above the floor.

Fuses :

Fuses are a safety device for protecting conductors against overload and fires. Fuses and automatic circuit breakers prevent generators or batteries from having too great a load imposed upon them and also protect current consuming devices, such as electric motors, from the effects of overload. The blowing of a properly rated fuse is an indication of fault in the installation as these fuse elements are rated to melt when the current is approximately double the normal rating of the fuse. It is dangerous to put in a stronger wire, as is often done, to make the fuse to stay. The cables between the main switch-fuses and the distribution boards which they control should also be large enough to carry the max: current likely to flow. and the fuses which control these cables must on no account have a higher rating than that of the cables.

The following table which is based on I.E.E. regulations 202A, is a useful guide as to the correct size of fuses, cables and flexible cords :—

Rating of Fuse	Size of Fuse wire	Min. size of Cable	Min. size of Flex Cord
Amps.	SWG		
3	38	1/.044	14/.0076
5	35	1/.044	40/.0067
10	29	3/.036	70/.0076
15	25	7/.029	110/.0076
20	23	...	162/.0076

If the wire is of Standard Alloy (63 per cent tin and 37 per cent lead) the same can be used as follows ;

Current Rating Amps.	1.8	3	5
Size of wire-SWG	27	23	21

Fuse wires should not be kept un-enclosd and fuse boards should not be located in corners where wood, paper, oil, petrol, or any other inflammable articles are kept. This ensures safety from fire.

Generally the minimum size of fuse for the branch circuits in consumers' premises is 5 amps. The load on lighting sub-circuits should be generally restricted to not over 3 amps. For lighting circuits, fuses of 50 per cent over the max. working current are enough and for motor circuits generally 75 per cent in excess is enough.

The size of cables should be a little larger than given in the various tables for resistance of copper conductors which are based on a temperature of 15.6 deg. C. to allow for the higher temperuture due to overload and sun. Allowance should be made for increase in resistance based on the max: temperature at the hour of peak load. A rise of temperature of say 48.9 deg. C. will involve an increased resistance of about 30 per cent.

Stay wires should be fixed to the pole as near as possible to the point at which stress is applied and should make as large an angle with the pole as practicable. The lower end of the stay should, if possible, be fixed in the ground at a distance away from the pole not less than 1/3rd of the height at the point at which it is affixed to the pole. Where a line changes direction stays are necessary not only at the top of the pole but as far down as half-way.

Poles with cast iron base for the portion buried in ground are considered very satisfactory. In general the use of steel transmission line poles with feet buried deep in the earth will void the need for separate down leads for earthing.

Weight of Single Copper Wires per 1000 metres in kilograms

Gauge No.	SWG	BWG	B & SG	Gauge No.	SWG	BWG	B & SG
1	339	339	315	16	15.4	15.8	9.75
2	287	304	250	17	11.8	12.7	7.72
3	239	253	198	18	8.67	9.05	6.11
4	203	213	157	19	6.01	6.68	4.86
5	169	183	124	20	4.90	4.60	3.85
6	139	155	98.8	21	3.85	3.86	3.05
7	117	122	78.4	22	2.95	2.95	2.42
8	96.2	114	62.2	23	2.17	2.36	1.93
9	78.0	82.6	59.4	24	1.82	1.82	1.53
10	61.8	67.6	49.1	25	1.50	1.50	1.21
11	50.6	54.4	31.0	26	1.22	1.22	0.96
12	40.7	44.8	24.7	27	1.01	0.96	0.76
13	31.9	33.9	16.5	28	0.82	0.74	0.59
14	24.1	25.9	15.5	29	0.69	0.63	0.48
15	19.5	19.5	12.3	30	0.58	0.55	0.39

(Wire gauges are given at Page 4M/17)

The buried ends of steel poles should be lapped with tarred gunny as a preservative. Special consideration must be given to the earthing of RC poles. In soft soils a small base plate is usually necessary under the foot of the pole.

Copper for electric wires has three grades : (*i*) soft or annealed wires, (*ii*) medium hard drawn wire, reduced in section by cold drawing, (*iii*) hard-drawn wire, which is cold drawn from a rod which is anealed at the same stage of its being drawn and reduced in section. The cold drawing increases the tensile strength but reduces elongation.

SYSTEMS OF WIRING & THEIR SUITABILITY

Screwed Steel Conduits. Are made up of comparatively heavy gauge mild steel tubes of varying thicknesses. No conduit less than 15 mm in diameter (inside) should be used. These tubes are threaded by means of dies to screw into appropriate fittings or accessories. The walls of the tubes are considerably thinner than gas or water pipes and the threads finer. For ordinary installations welded conduits are quite satisfactory. The finish of steel conduits and their accessories is black enamel ; for use in damp or exposed positions steel conduits are usually galvanized but this results in very nearly doubling the cost ; or otherwise two coats of iron oxide paint may be applied. Conduit wiring is the most resistant to mechanical injury as may cause shocks and even if a fire starts it cannot spread rapidly. This system of wiring is now generally adopted in all superior buildings. It is the best suited for warehouses and godowns, heavy engineering shops, timber and such other stores. Ordinary steel conduits (if exposed) are not suitable for factories where chemical fumes are produced (such as, in breweries, dye works, silk factories).

Prefer to use a fine tooth hacksaw for cutting conduits ; pipe cutters leave a burr inside. After cutting remove all burrs or sharp edges with a reamer, which may damage the insulation. Use tallow as a lubricant for the dies ; if a lubricant is not used the dies wear rapidly and soon result in tight or uneven threads. All joints must be perfectly tight otherwise it will be imposible to ensure proper earthing, and also control of moisture.

Earthing of the metal conduits is of utmost importance and also of cutting away every thread of the tape where it is stripped from the rubber at terminals. Conduits must be prevented from touching gas or water pipes. The completed installation of screwed conduits must be mechanically and electrically continuous across all joints so that the electrical resistance of the conduit together with the resistance of the earthing lead, measured from the connection to the earth electrode and any other point in the completed installation, must not exceed one ohm.

The lengths of conduits should be joined by means of screwed sockets. Threads should be free from grease or oil and no material of this nature should be allowed to come in contact with the conductors. All conductors used in conduit wiring should be stranded. The radius on the inner side of any bend should not be less than 75 mm.

CTS or **TRS** (tough-rubber sheathed) cables with stand dampness if the junction boxes are sealed with a plastic insulated compound. They will also resist acid and chemical fumes, including those of ammonia, but the wiring should be with all insulated fittings and all joints should be sealed properly. These cables should not be exposed to direct sunlight or high temperatures. TRS cables have a tendency to crack and open after some time where the wires have been bent to a sharp angle.

PVC cables stand chemical fumes very well but are unsuitable in high temperature localities.

Pyrotenax is suitable for high temperatures and where chemical fumes are produced.

Ordinary Twisted Flex cables are not suitable for rough handling, damp or hot locations ; catch fire rapidly. The heat from large gas filled lamps in often enough to rapidly deteriorate the insulation of a flex. Tough rubber flexible cables should be used in such locations.

Wooden Casing is the most common system in India. Casing is highly inflammable. It should be fixed at a distance from the walls in damp locations by means of moisture proof distance pieces. There is considerable risk of fire if wires of opposite polarity are crowded together in the same groove and the insulation becomes damaged.

Open Wiring on Cleats. This system is not suitable for ware-houses and godowns, or where the cables are likely to be exposed to mechanical damages, or places where inflammable materials are stored. Cleated wiring is not suitable for permanent domestic installations. VRI (vulcanized—rubber insulated) cables on cleats should be used for temporary works. It should be employed only where the pressure is not more than 250 volts. It is a cheap but inferior system of wiring and can be installed very rapidly.

Lead-sheated Wiring. Is very suitable for domestic premises and moist places but not so much for godowns and stores. It is unsuitable where liable to exposure to ammonia fumes, as in cattle sheds. It needs more care and skill and costs more to install than tough rubber sheathed cables.

Joints in all types of wiring are a source of weakness causing over-heating or even fire. Except for joints in flexible wires, soldering or proper mechanical joints should be made as far as possible. Any wiring should not be fixed close to hot water, stream or gas pipes.

Maximum Intervals between Poles : The intervals shall not be greater than necessary for ensuring the safe limits of breaking loads of conductors. In over, along or across any street the interval shall not exceed 67 m (I. E. Rule 67); 45.7 m is the most common span.

According to I.E. Rule 62, no conductor of an aerial line erected over, along or across any street shall be less than 6 m from the ground, and in private grounds or at a consumer's premises, it shall not be less than 4.6 m. In the case of outdoor sub-stations in public places, proper fencing to a height of 2.4 m and a minimum clearance to the live conductors of 1.5 m horizontally from the fencing upwards, should be made.

No service line or tapping shall be taken off an aerial line except at a point of support. (I. E. Rule 78).

All conductors should be of copper. No insulated conductor shall have a cross-section less than that of one No. 18 SWG and every such conductor of greater cross-section shall be stranded. Flexible conductors must be made up so that the total cross-sectional area is not less than equivalent to No. 22 SWG and they must be composed of wires twisted together on a short lay, no wire being smaller than a No. 40 SWG.

Where conductors pass through floors, they shall be carried in a heavy guage insulated conduit or a porcelain tube. The floor tube shall be carried 30 cm above the floor line and 25 mm below ceiling line. Where the supply is AC the conductors of the conduit must be bunched in the tube.

All casing and capping should be served with two coats of varnish before erection, internally and on the back, also after erection.

Insulated wires exposed to sun and rain deteriorate quickly though the weather proof quality has over double the life of VIR wires. Lime and distemper gradually ruin the insulation of VIR wires and oil attacks the rubber covering of cables. PVC cables stand up well to oil and petrol.

Plugs (for fixing wires) for ordinary walls or ceilings should be of well season teak or other hardwood not less than 50 mm long by 25 mm square on the inner and 20 mm square on the outer end, except for use with metal sheathed wiring. They should be cemented into the walls to within 6 mm of the surface, the remainder being finished according to the nature of the surface ; and to give the cement a hold on the plug, there should be on each of two opposite sites two counter-bores not less than 12 mm diameter and 32 mm deep. Woodens plugs used for metal-sheathed wiring should not be less than 40 mm long and 6 mm in diameter on the outer end. "Rawl plugs" or other special plugs may be used.

Where wiring has to be carried along the face of rolled steel joists, a wooden backing should first be laid on the joist and clipped to it as inconspicuously as possibble.

External and road lamps should have weather-proof fittings to prevent the admission of moisture. An insulating distance piece of moisture-proof material must be inserted between the lamp-holder nipple and that of the fitting.

With 3-wire or 3-phase installations the loading or circuit should be balanced. Circuit on opposite side of a 3-wire system or on different phase of a 3-phase system should be kept, as far as possible, apart where bringing them into the same room is unavoidable.

There should be one main switch and one main fuse on each pole of each main circuit (other than the netural conductor of a 3-wire circuit) at the point of entry of the supply.

In installations supplied from a 3-wire system all branch switches should be placed on the "outers."

All single pole switches are inserted in live conductors only.

All circuits are properly protected by fuses or circuit breakers.

If any point has a current rating of over 15 amps. it must be wired on a separate circuit. If any circuit has a current rating of over 15 amps. it must not consist of more than one point. This means that three 5 amps. sockets may be wired on one 5 amps. circuit, but two 10 amps. points must be wired on separate circuits. Provided the total current rating of a circuit does not amount to more than 15 amps., an unlimited number of points may be connected.

It is usual to arrange lighting points on 5 amps. circuit, in which case they would be wired with 3/.029 cables. Where large lamps are installed it may be necessary to wire for 10 amps. circuits using 3/.036 cables, or 15 amps. circuits using 7/.029 cables.

Connections of all circuits carrying more than 15 amps. shall be made by means of cable sockets.

Distribution Boards. Main distribution board should be provided with a switch and fuse on each pole of each circuit. Branch distribution boards should be provided with one fuse on each pole of each circuit. Switches and fuses of opposite polarity should be mounted on separate bases. The main switch board for medium or high pressure supply shall have a clear space of not less than 90 cm in front, and either less than 23 cm behind or a gangway of over 76 cm width and 180 cm height behind. (I.E. Rule 61).

Testing Polarity of Single-Pole Switches. All single-pole switches should be inserted in the live conductors only. If single-pole switches were inserted in the neutral or middle wire of a circuit there would be a considerable risk of person receiving a shock due to being mislead in thinking that switching off rendered any appliance safe. A shock with the switch off would also be much more severe than if the switch had been left on. The best way to test whether switches are on the live side is to use a test lamp. With the switches in the "off" position, if they are correctly connected, the lamp will give a full light when connected between earth and the switch feed. If they are incorrectly connected there will be no light between the earth and the switch feed but a reduced light would be the result if the circuit lamps were left in the lampholders.

Testing an Installation

Test to Earth—Is made with all fuse links in place, all lamps in position, and all switches on. The result must be not less than 50 megohms divided by the number of outlets, (*i.e.*, points and switch positions), except that it need not exceed 1 megohm for the whole installation. Control, rheostats, heating and power appliances, etc., may if desired, be disconnected for this test, but, if so, their insulation resistances must, in each case, be not less than half a magohm. When PVC cables are used the values of insulation resistance may be relaxed to 1/4.

Earth Continuity Test—In the case of cables encased in metal, (whether conduit or metallic sheathing) the total resistance of the conduit or sheathing from the earthing point to any other position in the completed installation shall not be more than 1 ohm.

Test between Conductors—Where practicable, a test should be made between all the conductors connected to one pole or phase conductor of the the supply and all the conductors connected to the middle wire or neutral or the other pole or phase conductors of the supply. While undertaking this test, all the lamps should be removed and all switches on. The result must be 50 megohms divided by the number of outlets (points and switch positions), but need not exceed 1 megohm for the whole insthllation.

Leakage at cable ends is one of the commonest causes of low readings. Therefore, all stray ends of cotton thread should be carefully removed when connecting up to switches, ceiling roses, etc.

Polarity of SP Switches—Tests should be made to verify that all non-linked SP switches are on an outer or phase conductor and not on the neutral or earthed conductor.

Proper Earthing of Electrical Installations

Earth Resistance of Various Soils: (Journal of the Institution of Electrical Engineers, Enland, Vol. 87):

Marshy ground	220—270
Clay	400—2,700
Brick clay	2,600—2,800
Chalk	6,000 – 40,000
Sand	9,000—8,00,000
Sand gravel	30,000—50,000
Rocky mountain area ...	1,00,000

For resistivity in ohms.—ft., divide by 33.

Perfectly dry earth is very nearly an insulator, and the conduction of the current is by means of the moisture contained in the soil and is effected by the manner in which the moisture is held. Temperature also has its effect on resistivity and, in general reduction of temperature increases the figure with a sharp increase at 0 deg. C. Therefore, it is evident that one of the major factors determining the resistance of an electrode is the specific-resistance of the soil surrounding it. The effect of the earth resistance of a plate or pipe can be very marked and variations as high as 14 to 1 have been recorded due to seasonal variations (from wet to dry), but pipes or plates buried beyond 1.8 m may not be affected much.

When looking out for positions for good earths the following ascending order of resistancy of soils may be kept in mind: (Journal of the Institution of Electrical Engineers, Vol. 72).

(*i*) Wet marshy ground and ground containing refuse such as ashes, cinders and brine waste. (*ii*) Clay, loamy soil, arable land, clavey soil and loam mixed with small quantities of sand. (*iii*) Clay and loam mixed with varying proportions of gravel and stones. (*vi*) Damp and wet sands. (*v*) Dry sand. (*vi*) Gravel and stones.

The resistance of the (*ii*) class of materials is twice that of the (*i*). The resistance of the (*vi*) class varies generally from 20 to 40 times that of the (*i*). Soil of fine texture freed from stones and compactly pressed round electrodes improves the earth resistance. The drier the earth the higher its resistance. In common soils more than 15 to 20 per cent. moisture is not necessary for good conductivity, but in sand and gravel water logging is found to be essential.

Pure water is a poor conductor, but the addition of a small quantity of common salt increases the conductivity to a large extent. The presence of a small quantity of salt in the water may reduce soil resistance by as much as 80 per cent. The conductivity of sea water is found to be 50,000, while that of some river waters 100 and of rain water (before falling to ground) only about 6.

Methods of Earthing:

(*a*) A separate earth-continuity conductor may be run throughout the premises and connected to the neutral at the supply end. The conductor to which exposed metal is connected is termed the earth continuity conductor.

(*b*) The individual cases may be connected to the neutral at the point of supply of the apparatus itself.

The first method is considered better. For the second method reliance is placed on the correct wiring of the system and on the integrity of the neutral conductor. A broken neutral coupled with a faulty piece of apparatus would produce a dangerous condition.

Pipe or rod electrodes have been found to be more efficient than plates and give lower resistance for the same surface area and cost. Rod electrodes can be either of copper or wrought iron and pipe electrodes of galvanized iron or cast iron. With pipes the connections of earthing wire can be above ground where the electrode is fenced, or below ground level in a small pit, and can always be examined for breakage, and is also less susceptible to corrosion. If at any time the pipe or rod is found not giving sufficiently low resistance, it can be driven deeper, but this is not practicable with a plate for which excavation is necessary. For the common purposes, a 20 to 12 diameter GI pipe is enough, driven down to at least 1.5 to 1.8 m; greater depths are necessary in dry or sandy soils; increase in length has a much greater effect. In areas liable to frost the pipe has to go deeper than the level to which frost can penetrate. Plates, where used, should also be buried to a depth of not less than 1.8 m below ground and should not also be less than 1.8 m from any building. Size need not be more than 2.3 to 2.8 sq. m.

In rocky areas where it is not possible to drive down the earth rods to sufficient depths to good moisture level, copper or GI strips or wires are used in long lengths, which are buried as deep as possible (but not less than 60 cm) GI strips can be about 25×3 mm and copper strips 25×1.5 mm and having the same surface area as that of an earth plate.

Whenever current passes through an earth electrode into the ground there is danger of electric field at the surface of the ground in the vicinity of the electrode, which can be fatal to animals they can be killed at much lower voltages than human beings. Therefore, the ground near an earth electrode should be fenced off.

Materials for Earthing

Copper stands corrosion better than iron but iron has greater mechanical strength and is also cheaper. Rust on pipes does not change the resistance of soil but dry rust on joint may add substantially to the joint resistance. Therefore, joints must be cleaned of rust. The earthing wire and the connections with earth electrodes should be of the same metal as far as possible, whether of iron or copper. Electrolytic corrosion will occur if dissimilar metals are in contact in the ground and exposed to the action of moisture. If it is absolutely necessary to connect dissimilar metals, the junction must be protected by painting it over thoroughly with a moisture resisting bituminous paint or compound. There is not much danger of corrosion with deep driven rods as corrosion elements in the soil are usually confined to the surface soil down to a depth of a few metres only due to the absence of free oxygen below. The earth wire is connected best by twisting round the cleaned surface of the pipe tightly three or four times and its end clamped to the pipe by GI bolts and nuts using GI washers. (See also Lightning Conductors in the Section on "Masonry Structures"). Not less than the equivalent (solid or stranded) of one No. 8 SWG (No. 12 min:) copper or GI wire should be used for making an earth connection. (See also "Electrical Equipment of Buildings"—Extracts from Regulations of the Institution of Electrical Engineers). The point in aiming at low resistance earths is that in the case of earth faults, the fuse at the supply end may blow and clear the same. The earth resistance to be aimed at, for any installation therefore, depends on the fuse size protecting it.

Where good earths are difficult to get, the system can be earthed with water supply mains (with at least 30 m of buried pipe) or well pipes. Such earthing is not objectionable with alternating currents but with direct currents the flow of fault currents in pipes produces electrolysis and results in heavy corrosion of pipes and also makes the water harmful to some extent. Gas mains or hot water pipes should not, however, be used, nor pipes conveying inflammable liquids, to obtain an earth connection, as there can be possibility of explosions. Connections to water mains for earthing purposes should be at the main water supply pipe and made on the supply side of the main stop cock. The water pipes must consist of metal-to-metal joints. A perfect electrical connection is essential. The surface of the pipe should be carefully cleaned and connection made with a screwed clamp.

It is always safer to connect the outer metal covers of all equipments like table fans, table lamps, electric irons, electric kettles, motor pumps, pipes carrying electric wires, and also any flexible metallic covering of the conductors, with wire to earth, to ensure small faults blowing circuit fuses and ensuring safety. I.E. rule 54 enjoins that this be done. Metal baths and metal sinks where electrical equipments are installed, should be electrically bonded to the cold water pipe. Earthing connections to portable apparatus are difficult to maintain effectively, therefore, all electrical apparatus should be of all-insulated design wherever practicable.

I.E. rule 57 requires that all motors and regulating and controlling equipments, generators, transformers, etc., shall be connected with earth by two separate and distinct connections.

Earth Resistance Test. For such tests, direct-reading self-contained sets—Evershed and Vignoles Megger Earth Testers, are now generally used. 12 mm dia. iron rod 0.9 m long and pointed at one end can be used as electrodes.

Testing Electric Motors. The insulation resistance between windings and frame of an electric motor should not be less than one megohm. Load tests should be made with an ammeter when the motor is working under full load.

Earthing af Overhead Lines and Poles etc. All metal-work of high voltage overhead lines shall be efficiently earthed. A continuous earth wire may be provided and connected with earth at four points in every mile; earthing of individual poles is unreliable. The second earth wire may be continuous or it may extend only to about 1.5 km from each sub-station. In the second case it reduces the possibility of damage to out-station equipment by nearby lightning strokes. Continuous earth wire or neutral conductor situated below the phase conductors guard against danger from a falling live conductor. The use of wood-pole lines with unearthed pole-top metal work largely eliminates transient faults and reasonable immunity from lightning. The stays should be properly insulated both above and below the portion at the level of the live conductors, to ensure the fuses blowing when these get accidently energised, is most important. Interference is often reduced and reception improved on *radios* by using an earth rod instead of a connection to the earth pin of a socket-outlet or a water pipe.

Causes of Fire. Failures of insulation of wires and cables is responsible for the heaviest proportion of all electrically-caused fires. The other major cause is the earth leakage flow over inadequate earthing arrangements. Short-circuit or breakage igniting rubber insulation cause fires. Circuit protective devices should be capable of operation on the occurrence of a dangerous earth leakage, shot-circuit and over-current. (See also matter following).

PRECAUTIONS TO AVOID ELECTRICAL ACCIDENTS*

Voltages of 250 or less are called low pressure, above 250 up to 650 volts medium pressure, and above that high pressure.

The average safe voltage with dry hands is 30 volts and with wet hands 22.5 volts, while for a person in a bath with the whole surface of skin contacting current, even 10 or 11 volts may prove dangerous. The effect of shock is lessened as voltage is reduced. Voltages of about 200 to 500 AC applied even a tenth of a second may prove fatal. With high voltage, the current often flows even before the victim touches the conductor, and is violently thrown off. The heavy current rush and the short period of contact are both not so dangerous as less heavy and longer duration contacts likely with modern AC voltages. The max: safe DC voltage is about 5 times that for AC and DC is less fatal than AC until about 1000 volts when it is more fatal. With DC systems there are fewer accidents with distribution voltages of about 220, but fatalities have, however, occurred with DC even at 120 volts.

The passage of electricity through the human body, particularly through the heart, is very dangerous to life. An electric current proves fatal when it passes through the body and for it to be able to pass through the body, the body should lie in the electric path between the two electrodes of an electric supply source. No harm would occur if a man contacted one electrode only, standing perfectly insulated from the other. Rubber shoes, rubber mat, dry wooden board or stool are good insulations, but not trees, metal poles, earth or buildings. Touching one wire accidently can be far more frequent than touching both simultaneously.

Generally for small houses, one neutral wire and one phase wire connections are brought in by well insulated wires to all the points needed. If we touch the neutral wire we may get no shock but if we touch the phase wire we get a shock. For supply to every large houses and factories using motors where all the three outer wires are taken in so as to give 400 volts between phases, as also the neutral wire, the danger if any two of the phase wires are touched is far greater than if any one of the phase wires is touched. Greater precautions are therefore prescribed in such cases than where single phase supply is used.

*This is based generally on "Electrical Accidents by K.V. Karantha."

The min: safe clearance from power lines shuld be 2.4 m vertical and 1.2 m horizontal for LT lines and 2.5 m and 1.5 m for HT lines from portions of any buildings to which persons can have access. Such precautions should also be taken with trees, especially fruit bearing trees. Fallen electric supply lines should not be touched except with dry poles unless declared to be dead. When a person has touched a live wire and cannot extricate himself, the current should be switched off atonce, but if that would involve more time than pulling the victim away, do the latter by taking the following precautions : (*i*) Use a dry wooden pole to push away the wire; (*ii*) stand on a wooden board, stool, chair, books, rubber mats, rubber shoes, coir matting, three or four layers of cloth or paper, to pull off the victim from the live equipment ; (*iii*) hands wrapped in a few layers of dry thick cloth or papers or rubber gloves can be used for releasing the victim; (*iv*) pull him by his clothes, if dry, or (*v*) use a rope. All the articles used must be dry. It is equally dangerous to handle the situation without precautions. In the case of H T equipments or lines, extra care and precautions will be necessary.

All the labourers engaged on work should be made to understand the necessary precautions for handling wires and other equipments and permanent gangs should be employed as far as possible. Use of safety belts, rubber gloves, gauntlets should be encouraged and proper lighting arrangements made for working at night. No person should work on lines or poles within 1.8 m of any overhead HT lines unless the following additional precautions have been taken :

The circuit to be worked on should be switched off or links or fuses opened or locked in the "off" position. After switching off the supply and before touching the lines, all conductors should be tested for pressure by a discharge rod ; the discharges wires should be kept at least 60 cm away from the man. All the conductors should then be short circuited together and adequately earthed ; this shall be done at two points one on each side of the place where the work is carried out. Poles on which work is to be actually carried out should also be earthed. All tools and equipments such as gloves, belts, gauntlets, ladders, earthing devices should be periodically examined for fitness. (Rubber goods should be preserved in French chalk.)

Common sources of danger from main supply lines :—

(*i*) Failure due to rusting of galvanized wires exposed to sea breeze.

(*ii*) Use of an insulator of an indequate size in an HT line ; it can have a flash-over melting the copper of the conductor at the point of flash and breaking the conductor.

(*iii*) Accidently energised poles and stays due to insulated wires dangling from the lines, losing their insulation and making accidental contact with the poles or stayes. Stay wires passing between live conductors are liable to cause danger should they get slack in course of time. Poles not erect, causing slackness in service wires or rusty or rotting poles and dangling aerial fuses which make fuse replacement difficult.

Electric Shocks and treatment. Artificial Respiration :

If no immediate medical aid is available and the patient is not breathing, artificial respiration should be given and continued until the patient revives or is diagnosed as dead. Most recoveries are apparent in the first ten minutes and a good proportion in twenty minutes, but fewer thereafter. But recoveries have been known even after 3 hours. It is more important to continue artificial respiration than loosing the precious time in taking the patient to a hospital.

The easiest method of artificial respiration is to press the lower portion of the chest inwards from the two sides and release alternatively, in the same way as happens when normally breathing to the rhythm of the breath of operator thus forcing air out and in until normal breathing is fully restored. The patient is laid down prone—on his abdomen with one arm extending straight along his head and the other bent at the elbow forming a rest for his head so that his nose and mouth are free to breathe. Kneel over the patient with your legs astride stradling his thighs and place your hands on the small of the back with fingers spreading down to the last rib, with arms held straight swing forward gently bringing your weight to steadly bear on the patient, thus compressing his chest and abdomen. Then swing back quickly removing the pressure entirely but not removing your hands. The above should be done about twelve to fifteen times a minute swinging forward when you breathe out and backward when you breathe in, making the patient also to do the same. Avoid violent jerky swinging forward.

Fires caused by Electricity

Fires start with the unsual heat caused by the passage of electric current under abnormal circumstances. Bad installations, poor materials, deterioration of insulation or accidental damage to wires or apparatus resulting in shorts, sparks, leakage or sustained overloads are the common causes of the unusual heat. Exposure to excessive heat, moisture or mechanical damage should be avioded. Joints in wiring cause overheating and sometimes fire. Presence of readily combustible materials or explosive gases near "faults" are very dangerors. Special precautions are necessary in garages.

Petrol vapour is about two and a half times heavier than air and collects on floors and inspection pits. Switches, plugs, light or other fittings should be fixed at not less than 40 cm above the floor. Main switches and cutouts of dwelling houses should not be fixed in garages. Conduit wiring and CTS flex (instead of ordinary flex) should be used in such localities.

In all dust laden atmosphere the lamps should be enclosed in dust-proof outer glass globes. Explosions have occured with the dust of cork, linseed, dextrin, sugar, cocoa, charcoal, coal, lampblack, malt, sulphur, wood flour and such other materials, and of fibrous materials like cotton-wool, kapoc, paper-pulp etc. All electrical fittings should be excluded from such rooms and vacuum bulbs which are cooler than gas filled ones should be preferred. The deposition of dry cellulose spray paint on even flame-proof lamp fittings may lead to fire.

Measures to Curb Electric Fires

Of many factors leading to fires in buildings, one is due to faulty electrical installations. Broadly, the electrical installation in a building can be split into three parts namely :

1) Electrical fittings for lighting, air coolers, air conditioners, refrigerators and other electrical fittings like plugs, sockets and switches :

2) Electrical wiring.

3) Main Board comprising Meter, Switch, Fuse unit and Sub distribution boards.

Fittings :

Regarding fittings, plugs, sockets, switches, the quality of these products plays an important role. Invariably small sparks start from these fittings due to poor quality and workmanship and even though the sparks are of small intensity they are liable to ignite inflammable materials in the vicinity. Some times these tiny sparks may not trip the safety systems. To avoid frequent tripping of fuses due to overloads, the tendency is to put higher rating fuses, thereby, overloading the entire wiring system. The termination points give rise to hot points. This practice is most prevalent and users are hardly aware whether the cables leading to the fittings are capable to carry that much load. The fires can be avoided in choosing better quality fittings and also the cables suitable for the type of load. Also at the distribution board, miniature circuit breakers of the required current rating should be fitted. It is not possible to increase or decrease the current ratings of the 'MCB' installed as no fusewire is needed. As compared to fuses where the possibilities of putting a higher ratings fuse wires are there in the 'MCB' this possibility is not there. This would avoid overload of cables. Similarly while connecting the fittings, care should be taken that termination of cables in the fittings should be properly done and no loose wire should be left. In case of fittings with loose screws, the same should not be used. It is also advisable to check termination regularly after some interval of time to ensure that no loose connections exist In case of aluminium cables, proper terminations becomes very important. The aluminium cable strands have a tendency to get loose after tightening and also formation of aluminium oxide which is a bad conductor of electricity. It is a fact that aluminium wires have excellent cold working properties and because of this type wire in the points and termination cannot be tightened permanently as aluminium flows out. Both these factors combined give rise to hot points on joints even on normal current loads and cause fires. This can be avoided by providing proper lugs on aluminium wire. The best solution, of course is to use copper cables in buildings.

Cables :

In number of buildings aluminium wires of small cross section are used. As described above these give rise to hot point and loose terminations. Also during installation these wires are pulled badly in conduits thereby damaging the insulation during pulling of wires as well as the conductor sizes get reduced. These cables are liable to cause fires specially when higher rating fuse wires are put on fuses. For buildings the use of copper cable is recommended. Also the cable should be protected by miniature circuit breakers and Earth Leakage Breakers.

Distribution Boards :

The electricity boards and utilities provide electricity connection to the buildings through a rewireable fuse. From the fuse the wires are brought to the meter, the consumer provides a main switch and fuses for individual circuits of light and power. The fuses of meter and also the main switches are put on a wooden board speciallty for small installations where the main fuse is replaced by higher rating fuse thereby overloading of meter and wires and also the workmanship is poor, this causes fires at the distribution board itself and is evident from the increased number of meter burnings. Not only the Electricity Board loses revenue on connecting load, but it makes their entire system weak. The supply should be connected by means of sealed miniature circuit breakers of appropriate ratings and the entire board should be made of sheet steel. The outgoing circuits for light and power should be connected through 'MCB'. This avoids the fuse replacement altogether and the system can rémain sound.

Safety for Human Life :

In an A.C. supply system the earthing of electrical equipment is necessary to avoid shocks and earth leakage fcr safety of human beings and fires. In our present system though the provision of earthing is there but sufficient care is not taken to maintain a proper earthing and to connect all electrical gadgets to earth wire. In most of the cases, the earth wire is ignored.

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SIMPLE CONVERSION TABLES
Metric Measures into English & Indian

Commercial Weights

	Tonnes to Tons	Quintals to Cwts.	Quintals to Mds.	Kilos. to Seers	Kilos. to Lbs.	100 Grams to Paus	100 Grams to Tolas	100 Grams to Ounces
1	0.98	1.97	2.68	1.07	2.20	0.43	8.57	3.53
2	1.97	3.94	5.36	2.14	4.41	0.86	17.1	7.05
3	2.95	5.90	8.04	3.21	6.61	1.29	25.7	10.6
4	3.94	7.87	10.7	4.29	8.82	1.71	34.3	14.1
5	4.92	9.84	13.4	5.36	11.0	2.14	42.9	17.6
6	5.89	11.8	16.1	6.43	13.2	2.57	51.4	21.2
7	6.90	13.8	18.7	7.50	15.4	3.00	60.0	24.7
8	7.87	15.7	21.4	8.57	17.6	3.43	68.6	28.2
9	8.86	17.7	24.1	9.64	19.8	3.85	77.2	31.7
10	9.84	19.7	26.8	10.7	22.0	4.29	85.7	35.3

Lengths / **Areas**

	Km. to Miles	Metres to Yds.	Metres to Ft.	Cm. to Ins.	Hec. to Acres	Sq. Metres to Sq. Yds.	Sq. Metres to Sq. Ft.	Sq. Cm. to Sq. Ins.
1	0.62	1.09	3.28	0.39	2.47	1.20	10.8	0.15
2	1.24	2.19	6.56	0.79	4.94	2.40	21.5	0.31
3	1.86	3.28	9.84	1.18	7.41	3.59	32.3	0.46
4	2.49	4.37	13.1	1.57	9.88	4.78	43.1	0.62
5	3.11	5.47	16.4	1.97	12.4	5.98	53.8	0.77
6	3.73	6.56	19.7	2.36	14.8	7.18	64.6	0.93
7	4.35	7.66	23.0	2.76	17.3	8.37	75.3	1.08
8	4.97	8.75	26.2	3.15	19.8	9.57	86.1	1.24
9	5.59	9.94	29.5	3.54	22.2	10.8	96.9	1.39
10	6.21	10.9	32.8	3.94	24.7	12.0	108	1.55

Volume or Capacity

	Cu. Metres to Cu. Yds.	Cu. Metres to Cu. Ft.	Cu. Cm. to Cu. Ins.	Litres to Gallons
1	1.31	35.3	0.06	0.22
2	2.62	70.6	0.12	0.44
3	3.92	106	0.18	0.66
4	5.23	141	0.24	0.88
5	6.54	177	0.30	1.10
6	7.85	212	0.37	1.32
7	9.16	247	0.43	1.54
8	10.5	282	0.49	1.76
9	11.8	318	0.55	1.98
10	13.1	353	0.61	2.20

SIMPLE CONVERSION TABLES

English & Indian Measures into Metric

Commercial Weights

	Tons to Tonnes	Cwts. to Quintals	Mds. to Quintals	Seers to Kilos.	Lbs. to Kilos.	Paus to Grams	Tolas to Grams	Oz to Grams
1	1.02	0.51	0.37	0.93	0.45	233	11.7	28.3
2	2.03	1.02	0.75	1.87	0.91	467	23.3	56.7
3	3.05	1.52	1.12	2.80	1.36	700	35.0	85.0
4	4.06	2.03	1.49	3.73	1.81	933	46.7	113
5	5.08	2.54	1.87	4.67	2.27	—	58.3	142
6	6.10	3.05	2.24	5.60	2.72		70.0	170
7	7.11	3.56	2.61	6.53	3.18		81.6	198
8	8.13	4.06	2.99	7.46	3.63		93.3	227
9	9.14	4.57	3.36	8.40	4.08		105	255
10	10.2	5.08	3.73	9.33	4.58		117	283

Tons in British ; Tonnes is Metric ; Cwt. is hundredweight
Kilos. is Kilogram (or Kg) ; Mds. is Maunds

Tons × 27.22 = Maunds
Seers × 2.057 = Lbs. (av.)
Lbs. (av.) × 0.486 = Seers
Ounces × 2.43 = Tolas
Cwt. × 1.36 = Maunds

Tons/sq. ft. × 10.94
= Tonnes/sq. m
Tonnes/sq. m × 0.091
= Tons/sq. ft.
Lbs./ft. × 1.488 = Kg/m

Lengths / **Areas**

	Miles to Km.	Yds. to Metres	Ft. to Metres	Ins. to Cm.	Acres to Hec.	Sq. Yds. to Sq. Metres	Sq. Ft. to Sq. Metres	Sq. Ins. to Sq. Cm.
1	1.61	0.91	0.30	2.54	0.40	0.84	.093	6.45
2	3.22	1.83	0.61	5.08	0.81	1.67	.186	12.9
3	4.83	2.74	0.91	7.62	1.21	2.51	.279	19.3
4	6.44	3.66	1.22	10.2	1.62	3.34	.372	25.8
5	8.05	4.57	1.52	12.7	2.02	4.18	.464	32.3
6	9.66	5.49	1.83	15.2	2.43	5.02	.557	38.7
7	11.3	6.40	2.13	17.8	2.83	5.85	.650	45.2
8	12.9	7.32	2.44	20.3	3.24	6.69	.743	51.6
9	14.5	8.23	2.74	22.9	3.64	7.52	.836	58.1
10	16.1	9.14	3.05	25.4	4.05	8.36	.929	64.5
11	—	—	—	27.9	—	—	—	—
12	—			30.5				

Furlong × 201 Metres

Sq. km × 0.39 = Sq.
Sq. miles × 2.59 = Sq.

Km. is kilometres ; Hec. is hectares

Volume or Capacity

	Cu. Yds. to Cu. Metres	Cu. Ft. to Cu. Metres	Cu. Ins. to Cu. Cm.	Gallons to Litres
1	0.76	.028	16.4	4.55
2	1.53	.056	32.8	9.09
3	2.29	.085	49.2	13.6
4	3.06	.113	65.6	18.2
5	3.82	.141	81.3	22.7
6	4.59	.170	98.3	27.3
7	5.35	.198	115	31.8
8	6.12	.226	131	36.4
9	6.88	.255	147	40.9
10	7.65	.283	164	45.5

1 pint = 20 fl. oz. = 0.568 litre
1 fl. oz. = 0.028 litre
1 litre = 1.76 pints
1 fl. lb. = 1/3 litre
1000 litres = 1 cu. m = 220 galls.
(fl. oz. is fluid ounces ; lb. is pound)

one cubic foot = 28.32 liters

Lbs./sq. in. × 0.07 = Kg./sq. cm
„ / „ × 703 = Kg./sq. m
„ /sq. ft. × 4.88 = Kg/sq. m
„ /cu. ft. × 16.02 = Kg/cu. m
„ / „ × 0.016 = Grams/cu. cm