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*Indian Practical  
Civil Engineers' Handbook*

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# Indian Practical Civil Engineers' Handbook

27763



Edited by \_\_\_\_\_  
P. N. Khanna, Chartered Engineer

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## PREFACE TO THE FOURTH EDITION

THE RECEPTION OF THE LAST EDITION OF THIS BOOK WAS FAR WIDER than I had anticipated and this edition also was exhausted within 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 Depts. 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.

Nov., 1957

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 reference 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.A.M. 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 omission or commission or give valuable suggestions for improvement of the book to enhance its usefulness.

P.W.D., PATIALA.  
Feb., 1953.

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.



## ACKNOWLEDGEMENTS

The author is indebted to the writers of the following publications from which great help has been derived in compiling this book :

1. American Civil Engineers' Handbook by Merriman and Wiggin.
2. Civil Engineering Handbook by Urquhart.
3. Kempe's Engineers' Year Book.
4. Civil Engineers' Reference Book by Trautwine.
5. Architects' and Builders' Handbook by Kidder and Parker.
6. Molesworth's Handbook.
7. Data Book for Engineers by Elwyn E. Seelye.
8. The "Practical Engineer" Pocket Book by Stuart.
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62. Sanitary Engineering by R.S. Deshpande.
63. Sanitary Engineering by V.H. Sadarangani.
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67. Small Sewage Works by F.C. Temple.
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69. Manual of Irrigation Practice (Punjab P.W.D.).
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92. The British Steel Piling Co., Ltd., London, Pocket Book.
93. Memorandum No. 575 on the Lay-out and Construction of Roads, Ministry of War Transport, England.
94. Memorandum No. 577 on Bridge-design and Construction of Roads, Ministry of War Transport, England.
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97. Electrical Earthing and Accident Prevention by *M.G. Say.*
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SOME OPINIONS ON EARLIER EDITIONS

11

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I am directed to acknowledge with thanks the receipt of "Indian Practical Civil Engineers' Handbook" by Shri P.N. Khanna.... I am sure this will be a valuable asset to Engineer-in-Chief's Library.

(Sd.)

for Major-General,  
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*Executive Engineer, P.I.D.C. Power Development,*  
Multan, West Pakistan

## SECTION 1

## WEIGHTS &amp; MEASURES

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The following abbreviations have been generally used:

c.,	cube	lbs.,	pounds
cm.,	centimetre	m.,	metre
cwt.,	hundredweight	md.,	maund
ft.,	foot or feet	mm.,	millimetre
gr.,	grains	oz.,	ounce
in.,	inch or inches	sq.,	square
kgm., or kilo.,	kilogramme	wt.,	weight
km.,	kilometre	yds.,	yards

## 1. WEIGHTS

**British—Commercial***Avoirdupois weights*

27.34 grains = 1 dram

16 drams = 1 ounce = 437.5 grs.

16 ounces = 1 pound = 7000 grs. = 0.454 kgm.

14 pounds = 1 stone

2 stones = 1 quarter

4 quarters = 112 lbs. = 1 cwt. = 1 md. 15 seers

2240 lbs. = 20 cwt. = 1 ton = 27.22 mds.

*Troy weights*—used for gold and silver (partly obsolete)

480 grains = 1 oz. troy

= 1.097 oz. avoirdupois

12 ozs. = 1 lb. troy

(obsolete)

= 0.82 lb. avoirdupois = 5760 grs.

175 lbs. troy = 144 lbs.

avoirdupois

100 lbs. troy = 82 $\frac{2}{7}$  lbs. avoirdupois

= 1 maund

*Apothecaries' weights*—used for medicine

20 grains = 1 scruple

3 scruples = 1 dram

8 drams = 1 oz. (troy)

12 ozs. = 1 lb. = 5760 grs.

The avoirdupois, troy and apothecaries grain are all of the same weight.

**Indian Bazar Weights**

8 ratties = 1 masha

12 mashas = 1 tola = wt. of rupee coin = 180 grs. = 0.41 oz.

5 tolas = 1 chhatak = 2 $\frac{2}{5}$  ozs. (avoirdupois)

4 chhataks = 1 powah = 8.2 ozs.

4 powahs = 1 seer = 2 $\frac{2}{5}$  lbs. (avoirdupois) = 32.9 ozs.40 seers = 1 md. = 82 $\frac{2}{7}$  lbs. (avoirdupois) = 0.73 cwt.

100 maunds = 3.673 tons

**Americans Units**

2000 lbs. = 1 ton = 907.185 kilos.

1000 lbs. = 1 kip

100 lbs. = 1 cwt. = 45.36 kilos.

2.2 lbs. = 1 kilo.



- 1 British ton = 1.12 U.S. tons  
 2000 lbs. = 1 short ton  
 2240 lbs. = 1 long ton (British ton)  
 The English and U.S. pound is about the same.

### Metric System

- 10 milligrams = 1 centigram = 0.15 grain  
 10 centigrams = 1 decigram  
 10 decigrams = 1 gram = 15.43 grs.  
 10 grams = 1 decagram  
 10 decagrams = 1 hectogram = 3.527 ozs.  
 10 hectograms = 1 kilogram = 35.27 ozs. = 2.2 lbs.  
 100 kilograms = 1 quintal = 1.968 cwts.  
 1,000 kilograms = 1 tonne = 0.984 ton  
 1 grain = 0.0648 gram = 64.8 milligrams  
 1 oz. (avoir) = 28.35 grams  
 1 lb (avoir) = 16 ozs. or 7000 grs. = 0.454 kgm.  
 1 cwt. = 112 lbs. = 50.802 kilograms  
 1 British ton (long ton) = 1.016 Metric ton (tonne)  
 = 1016.06 kilograms

(dekagram = decagram; gram = gramme;  
 kilogram = kilogramme; milligram = milligramme)

## 2. LINEAR AND SQUARE MEASURES

### British

### Gunter's Surveying Chain

- |                          |                         |
|--------------------------|-------------------------|
| 12 inches = 1 foot       | 7.92 inches = 1 link    |
| 3 feet = 1 yard          | 1 chain = 100 links     |
| 5½ yards = 1 rod or pole | = 4 poles = 22 yards    |
| 220 yards = 1 furlong    | 10 chains = 1 furlong   |
| 8 furlongs = 1 mile      | 100 sq. chains = 1 acre |
| 5000 ft. = 1 canal mile  |                         |

1 micro-inch = 1 millionth of an inch.

**Geographical and Nautical Measures.** A nautical or sea mile is the distance on the earth's surface at the sea level of one minute of arc of longitude of earth at the equator. This is about 6087 ft. (2029 yds.). The statute (or land) mile is 5280 ft. but the British nautical mile is 6080 ft. (5280 × 1.1515). The International nautical

mile is 6076.12 ft. or 1852 metres which is not accepted by the British Admiralty. The Admiralty use the relation:

6 ft. = 1 fathom; 100 fathoms = 1 cable length; 10 cables are approximately 1 nautical mile; 3 nautical miles = 1 league.

The Admiralty knot is a rate and not a distance: 1 knot is 1 nautical mile per hour = 1.1515 miles (land miles) per hour = 1.853 kilometres per hour.

United States Coast Survey Department take 1 nautical mile = 6086.07 ft. or 1.152664 statute or land miles.

### Square Measures

144 sq. ins.	= 1 sq. ft.
9 sq. ft.	= 1 sq. yd.
30¼ sq. yds.	= 1 sq. rod, pole or perch = 272¼ sq. ft.
40 sq. rods or poles	= 1 rood = 1210 sq. yds.
4 roods	= 160 sq. rods = 1 acre = 4840 sq. yds.
	= 43560 sq. ft. = 100 sq. chains (Gunter's)
640 acres	= 3097600 sq. yds. = 1 sq. mile

An acre is the area of a square measure 208.71 feet on a side.

### Indian Land Measures

#### Bombay Measure of Land

Linear		Square	
1 inch	= 1 tasu	1 kathi	= 39¼ sq. cubits
24 tasus	= 1 gaj		= 500 sq. ft.
1½ gajas	= 1 yard	20 kathis	= 1 pand
		20 pands	= 1 bigha
		6 bighas	= 1 rukeh
		20 rukeheys	= 1 chahur
1 anna	= 68⅓ sq. ft.	1 bigha	= 23 gunthas
16 annas	= 1 guntha = 1089 sq. ft.		= 2783 sq. yds
40 gunthas	= 1 acre	1½ bighas	= 1 acre

#### Assam Measure of Land

4 sq. cubits	= 9 sq. ft. = 1 kani
16 kanis	= 16 sq. yds. = 1 locha
20 lochas	= 2880 sq. ft. = 1 katha
5 kathas	= 14,440 sq. ft. = 1 bigha
4 bighas	= 1 pura; 3.025 bighas = 1 acre

*Bengal Measure of Land*

1 sq. cubit	=	1 ganda
5 sq. yds.=20 gandas	=	1 chhatak
16 chhatak=720 sq. ft.	=	1 katha
20 kathas	=	1 bigha = 1600 sq.yds.
3.025 bighas	=	1 acre
1961 bighas	=	1 sq. mile

*Madras Measure of Land*

2400 sq. ft.	=	1 ground or manai
24 ground=1 kani	=	1.32 acres (English)
484 kani	=	1 sq. mile
121 kani	=	160 acres

*U.P. Measure of Land*

20 invansi	=	1 kachvansi
20 kachvansi	=	1 bisvansi
20 bisvansi	=	1 bisva
20 bisvas=3025 sq. yds.	=	1 bigha = $\frac{5}{8}$ acre

*Punjab Measure of Land*

225 sq. ft.=9 karam or sarsi	=	1 marla
20 marlas = 4500 sq. ft.	=	1 kanal
4 kanals = 1620 sq. yds.	=	1 bigha
2 bighas	=	1 ghuma
9.68 kanals = 4840 sq. yds.	=	1 acre=43560 sq. ft.

**Metric Measures**

10 millimetres	=	1 centimetre	=	0.3937 inch
10 centimetres	=	1 decimetre	=	3.937 inch
10 decimetres	=	1 metre	=	3.28 ft.
10 metres	=	1 decametre	=	32.81 ft.
10 decametres	=	1 hectometre	=	328 ft. 1 in.
10 hectometres	=	1 kilometre	=	$\frac{5}{8}$ mile
10 kilometres	=	1 myriametre	=	6.2137 miles

1 micron = 1/1000 millimetre

$\frac{1}{8}$  inch = 1 millimetre

1 inch = 25.40 millimetres = 2.54 cm.

1 foot = 30.48 cm. = 0.3048 metre

1 yard = 0.914 metre

1 furlong = 0.201 kilometres = 201.168 metres

1 mile = 1.609 kilometres (km.)



1 sq. inch = 6.45 sq. cm.	1 sq. cm. = 0.155 sq. in.
= 645 sq. mm.	1 sq. metre = 10.76 sq. ft
1 sq. foot = 0.093 sq. metre	1 sq. km. = 0.386 sq. mile
1 sq. yard = 0.836 sq. metre	
1 sq. mile = 2.59 sq. km.	

### Sundry Measures

12 articles = 1 dozen	1 quire = 24 or 25 sheets of paper
12 dozens = 1 gross	20 quires = 1 ream
1 score = 20 articles	1 ream = 480 or 500 sheets

## 3. CAPACITY

### British

#### Dry Measure

1728 c.in. = 1 c.ft.
27 c.ft. = 1 c.yd.
= 21.02 bushels
1 bushel = 4 pecks
= 1.28 c.ft. = 8 galls.
8 bushel = 1 quarter
2 galls. = 1 peck

#### Liquid Measure

60 minims = 1 dram = 90 drops
8 drams = 1 ounce (oz.)
1 teaspoonful = $\frac{1}{8}$ oz.
1 desertspoonful = 2 drams = $\frac{1}{4}$ tola
1 tablespoonful = 4 drams = $\frac{1}{2}$ oz.
1 wineglass = 4 tablespoonfuls
= 2 ozs. = 16 teaspoonfuls
12 ounces = 1 pound
4 gills = 1 pint = 20 oz. = $\frac{1}{4}$ bottle
2 pints = 1 quart
4 quarts = 1 gall. = 8 pints
= 6 bottles
6.24 galls. = 1 c.ft.
31 $\frac{1}{2}$ galls. = 1 barrel

*The gallons used in India are the Imperial (English) gallons.*

### American Standards

1 U.S. gall. = $\frac{5}{8}$ Imperial gall. = 3.78 litres = 231 c. ins.
1 Imperial gall. = 1.20 U.S. gall. = 4.55 litres = 277 c. in.

### Imperial, Metric and American Equivalents

1 c.in. = 16.4 c. cm. (cc)	1 c. cm. = 0.061 c. in.
1 c.ft. = 0.028 c. metre	1 c. metre = 35.31 c.ft.
1 c. yd. = 0.764 c. metre	
1 litre = 1.76 pints (Imperial) = 2.11 pints (U.S.)	
= 0.22 gall. (Imperial) = 0.26 gall (U.S.) = 61.02 c.ins.	
For capacity and weight of water see under "Hydraulics"	

## 4. CONVERSION FACTORS

<i>To Convert</i>	<i>Multiply by</i>	<i>To Convert</i>	<i>Multiply by</i>
inch to mm.	25.40	sq. in. to sq. mm.	645.2
mm. to inch	0.039	sq. in. to sq. cm.	6.451
inch to cm.	2.54	sq. cm. to sq. in.	0.155
cm. to inch	0.3942	sq. ft. to sq. metre	0.092
inch to metre	0.025	sq. metre to sq. ft.	10.764
metre to inch	39.37	sq. yd. to sq. metre	0.836
ft. to metre	0.305	sq. metre to sq. yd.	1.196
metre to ft.	3.281	acre to sq. metre	4046.9
yd. to metre	0.914	sq. metre to sq. acre	0.00025
metre to yd.	1.093	sq. mile to sq. km.	2.59
miles to km.	1.609	sq. km. to sq. mile	0.3861
km. to miles	0.621		
		c. inch. to c. cm.	16.387
oz. to grams	28.349	c. cm. to c. inch	0.061
grams to oz.	0.035	c. ft. to c. metre	0.028
lb. to kgm.	0.453	c. metre to c. ft.	35.31
kgm. to lb.	2.204	c. yd. to c. metre	0.764
cwt. to kgm.	50.80	c. metre to c. yd.	1.038
kgm. to cwt.	0.02		
tons to kgm.	1016	pints to litres	0.568
kgm. to tons	0.00098	litres to pints	1.76
U.S. tons to kgm.	907.18	Imp. galls to litres	4.546
kgm. to U.S. tons	0.0011	litres to Imp. galls.	0.22
tons/sq. in. to kgm./		U.S. galls. to litres	3.785
sq. mm.	1.575		
seers to lbs. (avoir)	2.06		
maunds to cwt.	0.73		
tons to maunds	27.22		

**Specific Gravity**

Specific gravity of solids and liquids is usually expressed as the ratio of the density of the substance to that of water. The density of water is 62.426 lbs. per c.ft. The weight of a material is its specific gravity  $\times$  62.426.

## 5. THE SOLAR SYSTEM

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 minutes and 49.7 seconds or nearly  $365\frac{1}{4}$  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.

Equatorial radius of the Earth	..	3963.36 miles
Polar " " " "	..	3950.04 "
Circumference of " " "		24,900 "
Mean distance of the Sun from the Earth		92,890,000 "
Max: " " " (1st July)		93,950,000 "
Min: " " " (1st January)		90,950,000 "
Radius of the Sun	..	424,500 "
" " Moon	..	1081 "
Distance of the Earth from the Moon		238,857 "
Length of 1 degree in latitude:		
Equator	..	362,750 ft.
Pole	..	366,396 "
Length of 1 degree in longitude:		
Equator	..	365,186 "

One degree of a great circle of the Earth at the equator  
 =60.064 nautical miles=69.194 statute miles.

## 6. WEIGHTS OF MATERIALS

Alum	106	lbs./c.ft.
Aluminium, sheets	167	"
" cast	160	"
Asbestos	153—187	"
" board	75	"
" cement flat sheets $\frac{1}{4}$ " thick	2 $\frac{1}{2}$	lbs./sq.ft.
" " corrugated $\frac{1}{4}$ " thick	3 $\frac{1}{2}$	"
Ashes	27	lbs./c.ft.
Ashes and coke loose	30—46	"
Asphalt	88 (av.)	"
" mastic, per 1" thickness laid	11	lbs./sq.ft.



Babbit metal	456	lbs./c.ft.
Bakelite	80—120	"
Ballast, brick	57	"
„ stone, loose $1\frac{1}{2}$ " (approx)	90	"
„ consolidated	120	"
Basalt	185	"
Beaswax	60	"
Bitumen	64	"
Blocks, hollow, partition		
clay, 1" of thickness as laid	5.25	lbs./sq.ft.
„ concrete „	5.50	"
Brass-cast	500—520	lbs./c.ft.
Bricks-dry	120	"
Brick-ballast	57	"
Brickwork, ordinary	112—120	"
„ sundried	100	"
„ reinforced	115—125	"
Bronze-Gun metal	545 (av.)	"
Cast iron	(430—467)—450 (av.)	"
„ steel	490	"
Cement, loose	75—90	"
„ compacted	110	"
Chalk	117—174	"
Charcoal (wood)	19—40	"
Chromium	428	"
Clay	120—130	"
Clinker (for concrete)	90	"
Coal, loose	50—56	"
„ dust	56	"
„ steam (Bengal)	55	"
„ heavy quality	75—95	"
Coal Tar	65 (av.)	"
Coke	60—100	"
Concrete, cement (stone)	130—150	"
„ „ reinforced	150 (av.)	"
„ „ and cinders	110	"
„ lime (brick)	112	"
„ coke breeze	80—100	"
„ clinker	90	"
„ pumice	50—70	"

Concrete sawdust	70	lbs./c.ft.
„ aerated or cell	16	„
Copper, sheet	548	„
„ wire	555	„
„ wrought	558	„
Cork	15	„
Earth, rammed	100	„
„ loose	90	„
German silver	517—555	lbs./c.ft.
Glass		
rolled plate $\frac{1}{4}$ " thickness	3.5	lbs./sq.ft.
„ $\frac{3}{16}$ " „	2.75	„
sheets $\frac{1}{10}$ " „	21	oz./sq.ft.
„ $\frac{1}{8}$ " „	26	„
„ $\frac{3}{32}$ " „	32	„
„ crown	155	lbs./c.ft.
„ flint	187	„
„ plate	170	„
Gold	1200	„
Grains	35—48	„
Granite	164—172	„
Graphite	137 (av.)	„
Gravel	112	„
Gum resin	69	„
Gun-metal	540	„
Gun-powder	59	„
Gutta Parcha	61	„
Gypsum	144	„
Ice	57 (av.)	„
Iridium	1400	„
Iron, ore	150—325	„
„ pig	450	„
„ wrought (461—486)	—480 (av.)	„
Lead	707	„
„ sheet $\frac{1}{16}$ " thickness as laid	8	lbs./sq..ft.
Leather	53—62	lbs./c.ft.
Lime stone	116—160	„
„ „ boulders	90 (av.)	„
„ „ unslaked	54—66	„

Lime slaked, fresh	36—40	lbs./c. ft.
„ „ after 10 days	50	„
„ unslaked (kankar)	74	„
„ slaked kankar	64	„
„ mortar	110	„
Linseed oil	58	„
„ 1 gall.	9½	lbs.
„ 1 pint	1.17	„
Loam	100	lbs./c.ft.
Macadam (binder premix)	140	„
„ (rolled)	160	„
Marble	170	„
„ girt	100	„
Masonry, dry (stone)	130	„
„ granite	160	„
„ sandstone	140	„
Mercury	849	„
Mica	177.8	„
Nickel	548	„
Oil, fuel and lubricating	56	„
„ linseed	58	„
„ turpentine	54	„
Paint-ready mixed, white zinc	18	lbs./gall
„ chocolate	25	„
„ white lead	28	„
„ aluminium or bituminous	11	„
Paper	44—67	lbs./c.ft.
Paraffin wax	50—60	„
Peat	40	„
Petrol	42.5	„
Petroleum	54	„
Phosphor-bronze	549	„
Pitch and tar	75—80	„
Plaster, cement	130	„
„ „ ½" thick	6	lbs./sq.ft.
„ lime	110	lbs./c.ft.
„ „ ½" thick	5	lbs./sq.ft.
Plaster of Paris	142	lbs./c.ft.
Platinum	1342	„



Porcelain	147	lbs./c.ft.
Pudlo	42	"
Pumice stone	25—56	"
Quartz	170	"
Red lead and litharge, dry	132	"
Red oxide, dry	64	"
Resin	50—68	"
Rip rap	80—90	"
Roofs—(see Section on "Roofs")		
Rubber	59	"
Salt	60	"
Sand, dry clean	96—100	"
" river	115	"
" wet	110—125	"
Sand-stone	140—175	"
Shale	162	"
Shellac gum	38	"
Shingle	90	"
Silica	130 (av.)	"
Silver	655	"
Slag, broken $\frac{1}{2}$ "	90	"
Slate	175—180	"
" 1" thick	14.75	lbs./sq.ft.
Snow, fresh	5—12	lbs./c.ft.
" compacted	15—50	"
Soapstone	165	"
Soda, caustic	80	"
" silicate	55	"
Steel rolled	(482—493)—490 (av.)	"
Sulphur	112—128	"
Tallow	57	"
Tar	65 (av.)	"
Terra-Cotta	117—148	"
Tin	454	"
Tungsten	1180	"
Turpentine—1 gall	8 $\frac{1}{2}$	lbs.
Uranium	1167	lbs./c.ft.

Water-fresh	62.4	lbs./c.ft.
„ sea	64—67	„
„ 1 gall.	10	lbs.
Wax (av.—60 lbs.)	36—112	lbs./c.ft.
Wheat	48	„
White lead, dry	86	„
Wire-barbed ordinary for fencing —440 yds.		1 cwt.
Wood (see Section on “Timber Structures”)		
Wood fuel	22	lbs./c.ft.
Wrought iron	480	„
Zinc	446	„
„ cast	428	„
„ sheet	448	„
„ „ as laid, 14 zinc gauge	1.59	lbs./sq.ft.
„ „ 0.025 in. thick	0.94	„

*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  $\times$  weight of water.

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## SECTION 2

### MATHEMATICS

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2. Logarithms .. ..	2/7
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4. Trigonometry .. ..	2/8
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### Mathematical Symbols

$+$ plus $-$ minus $=$ is equal to $\neq$ is not equal to $>$ is greater than $<$ is less than $\geq$ is equal to or greater than $\leq$ is equal to or less than $\times$ multiplied by $\div$ divided by $\angle$ any angle $\perp$ right angle $\perp$ is perpendicular to $\therefore$ since, because $\therefore$ therefore, hence $()$ parentheses $[]$ bracket	$n^{\circ}$ n degrees $n'$ n minutes $n''$ n seconds $\triangle$ triangle or delta $\infty$ infinity $\parallel$ is parallel to $\sqrt{\quad}$ square root $\sqrt[3]{\quad}$ cube root $\Sigma$ sum of finite quantities $\Phi$ Phi } any $\theta$ Theta } angle $\pi$ $\pi = \frac{22}{7} = 3.14159$ $\propto$ varies as is proportional to $a_1$ a sub one $a_2$ a sub two
---	---



# MENSURATION

## Square

Diagonal of a square = side  $\sqrt{2} = 1.414$  side

## Triangle

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

or  $\sqrt{s(s-a)(s-b)(s-c)}$

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

## Equilateral triangle

$$h = \frac{a\sqrt{3}}{2}$$

h = height  
 a = side

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

## Isosceles triangle

$$\text{Area} = \frac{c\sqrt{4a^2 - c^2}}{4}$$

c = base

## Circle

$$\text{Area} = \frac{\pi \times \text{dia}^2}{4} = \pi \times \text{radius}^2 \text{ or } 0.7854 \times \text{dia}^2.$$

$$\text{Circumference} = \pi d = 3.1416 \sqrt{\text{area of circle}}$$

$$\text{Dia.} = \text{circum.} \times 0.3183 = 1.1283 \sqrt{\text{area of circle}}$$

$$\text{Side of a square equal in area} = \text{dia.} \times 0.8862$$

$$\text{Side of an inscribed square} = \text{dia.} \times 0.6071$$

## Length of an Arc (BCD)

(Also see under "Road Curves".)

$$= \frac{\phi}{360} \times 2\pi r = \frac{8b-2a}{3}$$

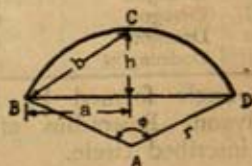
$$\text{or No. of degrees} \times \text{radius} \times 0.01745 = \frac{2 \text{ Area}}{r}$$

$$\text{Area of Sector ABCD} = \frac{\phi}{360} \times \pi r^2$$

$$= \text{length of arc} \times \frac{1}{2} \text{ radius}$$

## Area of Segment BCD

$$= \frac{1}{2} h \sqrt{(a^2 + \frac{2}{3} h^2)} \text{ or } \frac{2}{3} BD \times h \text{ (approx.)}$$



$$r = \frac{h + a^2}{2h}; \quad h = r - \sqrt{r^2 - a^2}$$

$$a = \sqrt{h(2r-h)} \quad b = \sqrt{2rh}$$

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

Distance of centre of area of a semi-circle from diameter  $\left\{ \begin{array}{l} = \frac{4r}{3\pi} \end{array} \right.$

Distance of centre of area of a semi-circle arc from diameter  $\left\{ \begin{array}{l} = \frac{2r}{\pi} \end{array} \right.$

Distance of centre of area of surface of a hemisphere from diameter  $\left\{ \begin{array}{l} = \frac{r}{2} \end{array} \right.$



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

**Ellipse.** Area =  $\frac{\pi}{4} Dd$   $D$ =major axis;  $d$ =minor axis

(Long axis  $\times 0.7854$  = short axis)

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

$$\text{Or Perimeter} = 3.1416 \sqrt{2(a^2 + b^2)}$$

$$= 3.1416(a+b) - \text{approx.}$$

$$b = \frac{d}{2}, a = \frac{D}{2}$$

**Parabola.** Area = base  $\times \frac{2}{3}$  height

**Polygons.** Area of any regular polygon = radius of inscribed circle  $\times \frac{1}{2}$  number of sides  $\times$  length of one side.

TABLE OF POLYGONS

No. of sides	Name	A degree	Area = $S^2 \times$	$S = R \times$	$S = r \times$
3	Triangle	60	0.433	1.732	3.464
5	Pentagon	108	1.721	1.176	1.454
6	Hexagon	120	2.598	1.000	1.155
8	Octagon	135	4.828	0.765	0.828
10	Decagon	144	7.694	0.618	0.650
12	Dodecagon	150	11.196	0.517	0.543

$A$ =angle formed by intersection of 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.

**Quadrilateral inscribed in circle**

$$A = \sqrt{(s-a)(s-b)(s-c)(s-d)}$$

a b c d are sides;  $s = \frac{a+b+c+d}{2}$

**Circle inscribed in triangle**

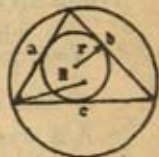
Radius =  $\frac{\text{area of triangle}}{s}$   $s = \frac{a+b+c}{2}$

For equilateral triangle, Radius =  $\frac{\text{side} \sqrt{3}}{6}$

**Circle circumscribed about triangle**

Radius =  $\frac{a b c}{4 \times \text{area of triangle}}$

For equilateral triangle, Radius =  $\frac{\text{side}}{\sqrt{3}}$



**Simpson's Rule**

Divide the area into an equal number of parallel strips of equal width d.

$$A = \frac{d}{3} \left[ \text{first ordinate} + \text{last ordinate} + 2 (\text{total of all odd ordinates}) + 4 (\text{total of all even ordinates}) \right]$$

**Volumes and Surfaces**

Diagonal of a cube = edge of cube  $\times \sqrt{3}$

Diagonal of a rectangular solid =  $\sqrt{a^2 + b^2 + c^2}$

a=length, b=breadth, c=depth.

**Pyramids**

$$V = \frac{1}{3} A h$$

$$S = \frac{1}{2} p s + A$$

A=area of base,

h=ht. of cone,

S=whole surface area,

p=perimeter of base,

s=slant height.



**Circular Cones**

$$V = \frac{1}{3} \pi r^2 h$$

$$s = \text{slant height,} = \sqrt{r^2 + h^2}$$

$$S = \pi r (\sqrt{h^2 + r^2} + r)$$

= Area of base + (circumference of base  $\times \frac{1}{2}$  slant height)

$$p = \text{perimeter of base,}$$

$$r = \text{radius of base,}$$

$$S = \text{area of entire surface.}$$

**Regular Tetrahedron**

$$V = \frac{2a^3 \sqrt{2}}{3}; \quad h = 2a \sqrt{\frac{2}{3}}$$

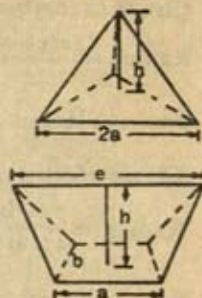
$$S = 4a^2 \sqrt{3}$$

**Wedge on rectangular base**

$$V = \frac{bh}{6} (2a + c) = \frac{A}{3} (2a + c)$$

or Area of base  $\times \frac{1}{3}h$  (approx)

A is area of cross section.

**Spheres**

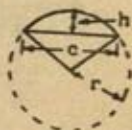
$$V = \frac{\pi d^3}{6} = 0.5236d^3; \quad \text{Area of surface} = \pi d^2$$

Side of an equal cube = diameter  $\times 0.806$

**Spherical Sector**

$$V = \frac{2}{3} \pi r^2 h = 2.094r^2 h$$

$$S = \pi r (2h + \frac{1}{2}c)$$



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

S = total area of conical and spherical surface.

**Spherical Segment**

$$V = \pi h^2 \left( r - \frac{h}{3} \right) = \pi h \left( \frac{c^2}{8} + \frac{h^2}{6} \right)$$

Area of spherical surface

$$= 2\pi rh = \pi \left( \frac{c^2}{4} + h^2 \right)$$

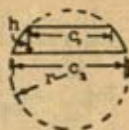
$$c = 2\sqrt{h(2r-h)} \quad r = \frac{c^2 + 4h^2}{8h}$$



# Spherical Zone

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

Area of spherical surface =  $2\pi rh$



# Spherical Wedge

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

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



# Hollow Sphere or Spherical Shell

$$V = \frac{4\pi}{3} (R^3 - r^3)$$

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

$V = \pi D^2 h$  (approx): D is outer dia., h is thickness.



# Cylindrical Ring or Torus

$$V = 2\pi^2 Rr^2 = \frac{\pi^2}{4} Dd^2$$

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



# Prismoids

$$V = \frac{h}{6} (A_1 + A_2 + 4A)$$

$A_1, A_2$  are the areas of ends.  
 $A$  is the area of mid section  
parallel to ends.,

$h$  is the length between ends,  
or height.

End faces are in parallel  
planes.

# Frusta of Pyramids and Cones

$$V = \frac{h}{3} (A_1 + A_2 + \sqrt{A_1 A_2})$$

Area of spherical surface  
=  $\frac{1}{2} s (P + p)$

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

## LOGARITHMS

$$\log 1 = 0$$

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

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

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

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

$$\log 52.23 = 1.7180$$

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

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

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

$$\log .05223 = \bar{2}.7180$$

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

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

## ALGEBRAIC FORMULAE

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

$$.1 = \frac{1}{10} = 10^{-1}; \quad 1 = 10^0; \quad a^0 = 1$$

$$a^{\frac{2}{3}} = \sqrt[3]{a^2}; \quad X^{\frac{1}{m}} = m\sqrt{X};$$

$$(-a)^{2n} = a^{2n};$$

$$(-a)^{2n+1} = -a^{2n+1}$$

$$X^{-m} = \frac{1}{X^m}; \quad X^m = \left(\frac{1}{X}\right)^{-m}$$

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

$$a^m \div a^n = a^{m-n}$$

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

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

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

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

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

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



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

$$(a+b)(a-b) = a^2 - b^2 \quad a^2 + b^2 = (a+b)^2 - 2ab$$

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

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

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

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

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

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

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

### Quadratic Equations

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

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

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

### Cubic Equations

$$x^3 + ax + b = 0$$

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

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

## 4. TRIGONOMETRY

A Radian is the angle subtended at the centre of a circle by an arc whose length is equal to the radius.

$$1 \text{ Radian} = 57^\circ - 17' - 45'' = \frac{180}{\pi} \text{ degrees.}$$

$$180^\circ = \pi \text{ Radians.}$$

$$\sin = \frac{\text{perp.}}{\text{hyp.}} = \frac{1}{\csc}$$

$$\cos = \frac{\text{base}}{\text{hyp.}} = \frac{1}{\sec}$$

$$\tan = \frac{\text{perp.}}{\text{base}} = \frac{1}{\cot}$$

$$\cot = \frac{\text{base}}{\text{perp.}} = \frac{1}{\tan}$$

$$\sec = \frac{\text{hyp.}}{\text{base}} = \frac{1}{\cos}$$

$$\csc = \frac{\text{hyp.}}{\text{perp.}} = \frac{1}{\sin}$$

$$\text{Versin } A = 1 - \cos A$$

$$\text{Conversin } A = 1 - \sin A$$

$$\text{Versed sin} = \frac{\text{hyp} - \text{base}}{\text{hyp}}$$

$$\text{Conversed sin} = \frac{\text{hyp} - \text{perp}}{\text{hyp}}$$

$$\begin{aligned} \text{Versin}(180 - A) \\ = 2 - \text{versin } A \end{aligned}$$

$$\begin{aligned} \text{Conversin}(180 - A) \\ = \text{conversin } A \end{aligned}$$

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

The complement of an angle  $A = 90^\circ - A$

The supplement of an angle  $A = 180^\circ - A$

### Functions of Negative Angles

$$\begin{array}{l|l} \sin(-A) = -\sin A & \cot(-A) = -\cot A \\ \tan(-A) = -\tan A & \sec(-A) = \sec A \\ \cos(-A) = \cos A & \csc(-A) = -\csc A \end{array}$$

### Functions of $0^\circ$ and $90^\circ$

$$\begin{array}{l|l|l|l} \sin 0^\circ = 0 & \cot 0^\circ = \infty & \sin 90^\circ = 1 & \cot 90^\circ = 0 \\ \tan 0^\circ = 0 & \sec 0^\circ = 1 & \tan 90^\circ = \infty & \sec 90^\circ = \infty \\ \cos 0^\circ = 1 & \csc 0^\circ = \infty & \cos 90^\circ = 0 & \csc 90^\circ = 1 \end{array}$$

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

$$\sin^2 + \cos^2 = 1; \quad 1 + \tan^2 = \sec^2; \quad 1 + \cot^2 = \csc^2$$

$$\begin{array}{l|l} \sin(A \pm B) = \sin A \cos B \pm \cos A \sin B & \tan \theta = \frac{\sin \theta}{\cos \theta} \\ \cos(A \pm B) = \cos A \cos B \mp \sin A \sin B & \cot \theta = \frac{\cos \theta}{\sin \theta} \\ \tan(A \pm B) = \frac{\tan A \pm \tan B}{1 \mp \tan A \tan B} & \end{array}$$

### Sums and Differences of Functions

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

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

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

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

$$\tan A + \tan B = \frac{\sin (A+B)}{\cos A \cos B}; \quad \tan A - \tan B = \frac{\sin (A-B)}{\cos A \cos B}$$

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

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

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

### Functions of $2A$ , and $\frac{1}{2}A$

$$\sin 2A = 2 \sin A \cos A$$

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

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

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

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

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

$$\cos \frac{1}{2}A = \sqrt{\frac{1 + \cos A}{2}}$$

$$\tan \frac{1}{2}A = \sqrt{\frac{1 - \cos A}{1 + \cos A}}$$

$$= \frac{1 - \cos A}{\sin A}$$

### Solution of Triangles

#### Any triangle

Sine formulae:

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

where  $R$  = radius of circumscribing circle

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

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

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

Cosine formulae:

$$a^2 = b^2 + c^2 - 2bc \cos A$$

$$b^2 = c^2 + a^2 - 2ca \cos B$$

$$c^2 = a^2 + b^2 - 2ab \cos C$$

#### Right angled triangles

$$a = c \sin A = b \tan A$$

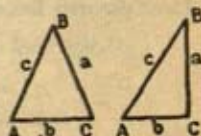
$$b = c \cos A = a \cot A$$

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

$$\cos B = \frac{c^2 + a^2 - b^2}{2ca}$$

$$\cos C = \frac{a^2 + b^2 - c^2}{2ab}$$

$$c = a \csc A = b \sec A$$





**Area of triangles**

$$A = \frac{ab \sin C}{2} = \frac{bc \sin A}{2} = \frac{ac \sin B}{2}$$

$$= \sqrt{s(s-a)(s-b)(s-c)}$$

$$\sin \frac{A}{2} = \sqrt{\frac{(s-b)(s-c)}{bc}}$$

$$\cos \frac{A}{2} = \sqrt{\frac{s(s-a)}{bc}}$$

$$\tan \frac{A}{2} = \sqrt{\frac{(s-b)(s-c)}{s(s-a)}}$$

$$\frac{a+b+c}{2} = s$$

$$\sin A = \frac{2}{bc} \sqrt{s(s-a)(s-b)(s-c)}$$

$$A = \frac{b^2 \sin A \sin c}{2 \sin B}$$

$$= \frac{b^2}{2 (\cot A + \cot C)}$$

**INTEREST****Simple Interest**

$$A = P + Pr = P(1+r)$$

**Compound Interest**

$$\text{at the end of 1 year— } A = P(1+r)$$

$$,, ,, ,, ,, 2 \text{ years— } A = P(1+r)(1+r) = P(1+r)^2$$

$$,, ,, ,, ,, n \text{ years— } A = P(1+r)^n$$

$$\text{and } P = \frac{A}{(1+r)^n} = A(1+r)^{-n} \quad \frac{A}{P} = (1+r)^n$$

If the interest is compound  $q$  times per year, we have

$$\frac{A}{P} = \left(1 + \frac{r}{q}\right)^{qn}$$

$P$  = principal,

$A$  = sum of principal and interest,

$r$  = rate of interest per 100 rupees,

$n$  = number of years

SECTION 3

**APPLIED MECHANICS  
&  
BENDING MOMENTS IN BEAMS**

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## 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.

*Stress* is the load applied per unit area.

*Strain* is the extension per unit length of the material stressed.

*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* is strength under twisting moments of components. This is not true shear strength of the material.

*Allowable unit stress* is the ultimate strength at failure divided by a factor of safety.

*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*.

*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 Section 5).



*Young's 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) or in other words, is the value of the increase of stress divided by the corresponding increase in strain i.e.

$$= \frac{\text{stress per sq. in. of section}}{\text{strain (or deformation) per in. of length}} = E$$

Modulus of elasticity is required in all calculations involving the deformation of structural members. The *co-efficient of extension* is the reciprocal of the modulus of elasticity and is  $\frac{\text{strain}}{\text{stress}}$ . These values are constant

for steel up to the "yield point" but not so with concrete. In other words, the stress is proportional to the strain (Hook's law) and the modulus of elasticity is constant.

Young's modulus is the tensile modulus and is the ratio of stress to strain in tension. The *shear modulus* or *torsion modulus* is the ratio of stress to strain in shear or torsion.

*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."

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

*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  $I = \sum A r^2$

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,

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

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  $1/16$  in. 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 in sq. inches.

$I_{AA}$  = 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 tons per sq. inch in extreme fibres of beams.

R = Radius of curvature.

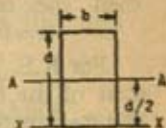
y = Distance of outermost fibre from neutral axis =  $d/2$

B M = Bending Moment or Moment of Resistance.

$$BM = \frac{f I}{y} = f Z; f = \frac{B M y}{I} = \frac{B M}{Z}; Z = \frac{I}{y}$$

$$\frac{B M}{I} = \frac{f}{y} = \frac{E}{R}$$

$$I = A r^2; A = b d$$



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

$$= 0.2887 d = \frac{d}{3.45}$$

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



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

When  $b$  is the least breadth of section, least  $r = \frac{b}{3.45}$

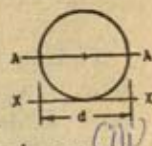
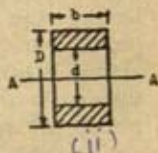
The stiffness of a beam depends on its moment of inertia and inversely on its length.

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

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

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

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



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'  $Z_c$ , and 'tension modulus of section'  $Z_t$ , 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 in inches. *Solid semi-circle or half round (resting on flat bottom).*

Neutral axis from bottom = 0.4244 R

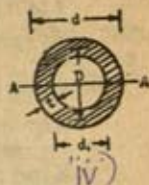
$$I_{AA} = 0.1098R^4; \quad Z_{AA} = 0.1907R^3; \quad r_{AA} = 0.2643R$$

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

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

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

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

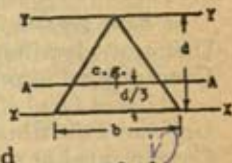


$$v) \quad I_{AA} = \frac{bd^3}{36}; \quad Z_{AA} = \frac{bd^2}{24};$$

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

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

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



### Thin Tubes, Cylinders or Shells of thickness "t"

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

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.	Channel	Unequal	Equal	Equal	Unequal
I	C	L	L	T	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 40 to 60 ft. lengths.

### Relative Strengths of Beams

Kind and position of load	Strength ratio
<i>(a) Beam supported at both ends :</i>	
Uniformly distributed over entire span .. ..	1
Concentrated at middle of span .. ..	$\frac{1}{2}$
<i>(b) Beams Fixed at both ends:</i>	
Uniformly distributed over entire span .. ..	$1\frac{1}{2}$
Concentrated at middle of span .. ..	1
<i>(c) Cantilever beams:</i>	
Uniformly distributed over entire span .. ..	$\frac{1}{4}$
Concentrated at the free end .. ..	$\frac{1}{8}$
<i>(d) Cantilever beam supported at the free end :</i>	
Uniformly distributed over entire span .. ..	1
Concentrated near the middle of span .. ..	0.65

**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 consequence, this limit may be exceeded. Depths of 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 deflection should not be more than  $1/40$  inch per foot of span or depth to span ratio of 18. Depth of R.S. Joists used as roof purlins should not be less than  $1/40$  of the span. In order not to exceed the deflection of  $1/40$  inch per foot of 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. For a greater depth than 12 to 14 inches, a built up girder is usually preferable and more economical under heavy loads. Deflection should be measured at mid-span of beams and at the end of cantilevers.



**Lateral Stability of Beams.** In all beams without continuous lateral support there is a tendency for the compression flange to buckle at right angles to the plane of bending in the mid-span section of the beam. The principal factor governing the elastic stability (against buckling) of a beam or column is the "slenderness ratio"  $l/r$ , where  $l$  is the distance between lateral supports and  $r$  is the least radius of gyration in the case of I beams. To safe-guard against this buckling the permissible stress (in compression) is reduced when the ratio of  $l/r$  exceeds 100. The basic principal compressive stress in bending according to B.S.S. 449 is:

$f = \frac{1000}{l/r} \times k$  tons/sq. in. when the depth to breadth ratio is less than 2.5. The factor  $k$  is assumed to have the following values:—

Depth to breadth ratio—	2.5	2.25	2.00	1.75	1.5 or less
Factor $k$	—1.00	1.125	1.25	1.375	1.50

Increased stress should not be allowed for deeper beams.

Where a beam is laterally supported continuously throughout its length, a deep narrow section is most economical, but where the compression flange is laterally unsupported, a wider, shallower section is preferable. A continuous load over the top flange of 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. Where the I beam is encased in cement concrete and reinforced with stirrups at not more than 6 ins. pitch and the minimum width of solid casing is equal to the width of the steel flange plus 4 inches, and the beam has a concrete cover of not less than 2 inches over the surface of the flanges, the permissible stress may not be reduced, but the stress shall not, however, exceed  $1\frac{1}{2}$  times that permitted for the uncased section. Stiffeners should be provided at all concentrated load points and at ends as a precaution against buckling of the web due to high compressive stresses. (This point is further discussed under "Plate Girders" in Section 19.) The ratio of depth of web to its thickness shall not exceed 75.

**Working Stresses**

For simple iron structures the safe working loads in tons per sq. inch may be taken as follows for dead loads:

	Ten.	Comp.	Shear	Modulus of Elasticity
Mild steel	10	10	6.5	13400 tons/sq. in.
Wrought iron	5	6	4	12500 "
Cast iron	1½	*	2	6500-7500 "

The structural steel manufactured by the Tata Iron and Steel Co. Ltd. complies with BS. 15 of 1948 and working stresses should confirm to the BS. Code of Practice—CP. 113(1948). The yield point is 15.25 tons/sq. in. minimum, and the ultimate strength 28/33 tons/sq. in. The ultimate stress of wrought iron ranges from 10/25 tons/sq. in., and that of cast iron 10/25 tons/sq. in. Ultimate tensile strength of cast steel ranges from 28/40 tons/sq. in.

*Permissible Working Stresses in Mild Steel Complying with BS. 15 of 1948—for Beams:*

Tensile stresses in Bending—10 tons/sq. in. for beams and 9.5 tons/sq. in. for plate girders.

Compressive stresses in Bending for uncased beams—10 tons/sq. in. for beams with adequate lateral support, and  $\frac{1000}{l/r} \times k$  for beams with insufficient lateral support.

Shear stresses in unstiffened webs of beams and girders—6.5 tons/sq. in.

(For properties of other steels and also for working stresses see Section 5).

**Stresses due to temperature changes.** Total expansion of a structure due to temperature change in inches = coefficient of linear expansion (of the structure metal)  $\times$  length of the structure in inches  $\times$  change of temperature in deg. F. If the structure is rigidly fixed so that the expansion due to temperature change cannot take place, the compressive stress developed in tons per

\*Working stresses for cast iron frequently used in compression free from flexure are  $\frac{1}{4}$  to  $\frac{1}{2}$  for dead weights,  $\frac{1}{4}$  for columns free from vibrations, and  $\frac{1}{4}$  for arch work, of the ultimate strength.



sq. in. = total (checked) expansion of the structure  $\times$  modulus of elasticity of the structure metal in tons per sq. in.

Co-efficient of linear expansion of steel is taken 0.0000067 per deg. F., and modulus of elasticity 13,000 tons per sq. in.

**Variable loads or Reversal of stresses** (varying from tension to compression):—

Equivalent dead load = max: load + variation.

Launhardt—Weyrauch 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.

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.

## BENDING MOMENT & DEFLECTION IN BEAMS

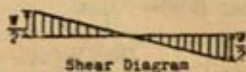
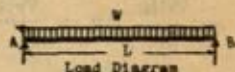
### Under Various Systems of Loading

W = total load in tons; w = load per ft. run in tons;  
L = span in inches; BM = bending moment in in. tons;  
E = modulus of elasticity (assumed at 13,000 tons per sq. in. for steel); RA = reaction at A; RB = reaction at B.  
Deflection is in inches.

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

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

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

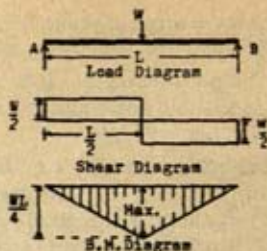




$$R_A = R_B = \frac{W}{2}$$

$$\text{Max: BM} = \frac{WL}{4}$$

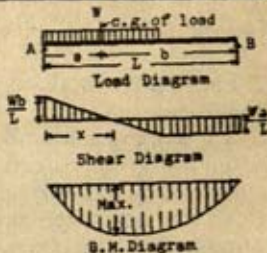
$$\text{Max: Def.} = \frac{WL^3}{48 EI}$$



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

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

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

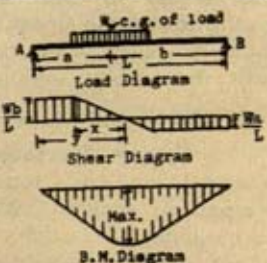


$$R_A = \frac{W.b}{L}, R_B = \frac{W.a}{L}$$

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

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

Def: varies with position of load

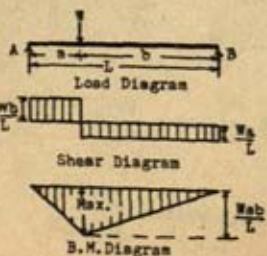


$$R_A = \frac{W.b}{L}, R_B = \frac{W.a}{L}$$

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

$$\text{Max: Def.} = \frac{W.a.b(2L-b)}{9EI L} \times$$

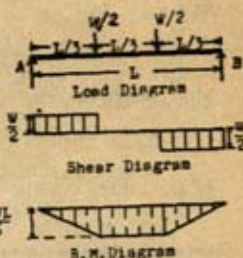
$$\sqrt{\frac{b}{3}(2L-b)} \quad b \geq a$$



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

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

$$\text{Max: Def.} = \frac{6.82 WL^3}{384 EI}$$



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

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

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

(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:—

$$\text{BM under } W_1 = R_A \times a$$

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

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

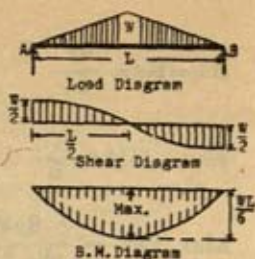
$$\text{or} = R_B \times (L-c)$$

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.

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

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

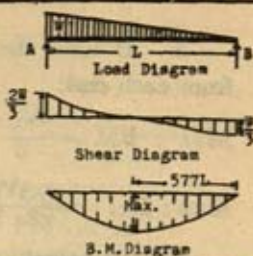
$$\text{Max : Def :} = \frac{W L^3}{60 E I}$$



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

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

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

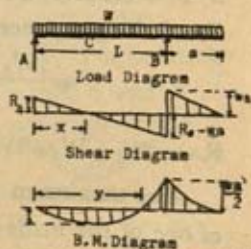


$$R_A = W \left[ \left( \frac{L+a}{2} \right) - a \right] \times \frac{1}{L} ; \quad R_B = W \left( \frac{L+a}{2} \right) \times \frac{1}{L}$$

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

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

$$\text{B M at B} = - \frac{W \cdot a^2}{2}$$

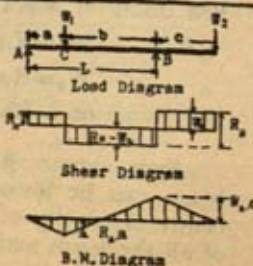


$$R_A = \frac{W_1 \cdot b - W_2 \cdot c}{L}$$

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

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

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

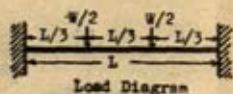






If  $a = \frac{1}{3}L$ ,  $b = \frac{1}{3}L$ 

$\frac{5W}{32}$	$\frac{27W}{32}$	$\frac{WL}{7.1}$	$\frac{WL}{21.3}$	$\frac{WL}{7.1}$	$\frac{WL}{14.2}$	$\frac{1.08WL^3}{384EI}$	0.30 L from A 0.17 L from B
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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  $L/3$  :—

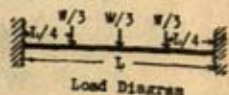
$\frac{W}{2}$	$-\frac{W.a(L-a)}{2L}$	$+\frac{W.a^2}{2L}$	$\frac{8W.a^2(3L-4a)}{384 EI}$	$\frac{(L-a)a}{L}$
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(c) Loads placed at  $L/4$  from ends instead of  $L/3$  :—

$\frac{W}{2}$	$-\frac{3WL}{32}$	$+\frac{WL}{32}$	$\frac{WL^3}{384 EI}$	0.19 L
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2. Loads  $W/3$  placed at  $L/4$  :—

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



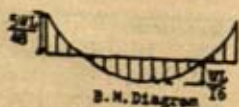
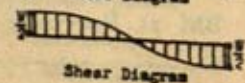
$$R_A = R_B = \frac{W}{2}$$

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

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

$$\text{Max : Def} = \frac{1.4WL^3}{384EI}$$

Point of contraflexure = 0.22L from A or B.



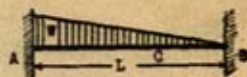
$$R_A = \frac{7}{10} W, \quad R_B = \frac{3}{10} W$$

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

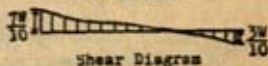
$$\text{BM at B} = -\frac{WL}{15}$$

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

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



Load Diagram



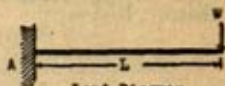
Shear Diagram



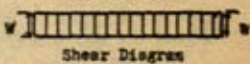
B.M. Diagram

$$\text{BM at A} = WL$$

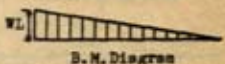
$$\text{Max : Def : } = \frac{WL^3}{3 EI}$$



Load Diagram



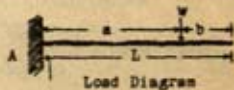
Shear Diagram



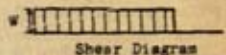
B.M. Diagram

$$\text{BM at A} = W \cdot a$$

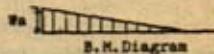
$$\text{Max : Def : } = \frac{W}{6EI} \times (3a^2L - a^3)$$



Load Diagram



Shear Diagram



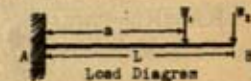
B.M. Diagram

$$R_A = W_1 + W_2$$

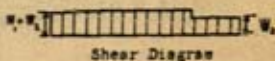
$$\text{Max : BM at A} = W_1 \cdot a + W_2 \cdot L$$

$$\text{Max : Def : }$$

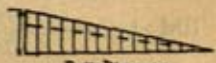
$$= \frac{W_1 \cdot a^2(3L - a) + 2W_2 \cdot L^3}{6 EI}$$



Load Diagram



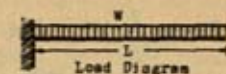
Shear Diagram



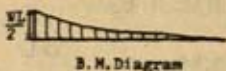
B.M. Diagram



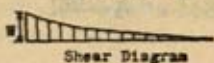
$$\text{Max : BM} = \frac{WL}{2}$$



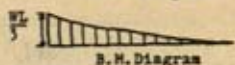
$$\text{Max : Def} := \frac{WL^3}{8EI}$$



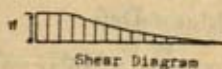
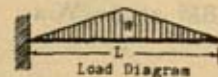
$$\text{Max : BM} = \frac{WL}{3}$$



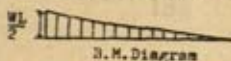
$$\text{Max : Def} := \frac{WL^3}{15EI}$$



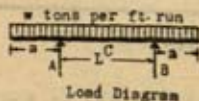
$$\text{Max : BM} = \frac{WL}{2}$$



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



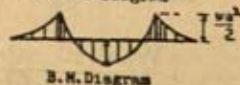
$$R_A = R_B = w.a + \frac{w.L}{2}$$



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



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



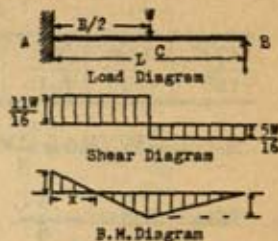
$$R_A = \frac{11}{16}W, \quad R_B = \frac{5}{16}W$$

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

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

$$B.M. \text{ at } C = +\frac{5}{32}WL$$

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



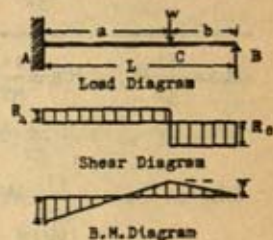
$$R_A = \frac{W.b(3L^2 - b^2)}{2L^3}$$

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

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

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

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



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

Reactions		B M			Max: Def:	Point of Contraflexure from A
$R_A$	$R_B$	Max:	at A	at C		
$\frac{13W}{27}$	$\frac{4W}{27}$	$\frac{WL}{5.78}$	$\frac{WL}{6.75}$	$\frac{WL}{5.78}$	$\frac{3.58 WL^3}{384 EI}$	$0.31 L$

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

$\frac{23W}{27}$	$\frac{4W}{27}$	$\frac{WL}{5.405}$	$\frac{WL}{5.405}$	$\frac{WL}{10.12}$	$\frac{2.37 WL^3}{384 EI}$	$0.21 L$
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If  $a = \frac{3}{4}L$ ,  $b = \frac{1}{4}L$

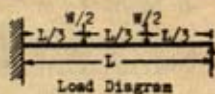
$\frac{47W}{128}$	$\frac{81W}{128}$	$\frac{WL}{6.305}$	$\frac{WL}{8.5}$	$\frac{WL}{6.305}$	$\frac{3.06 WL^3}{384 EI}$	$0.32 L$
-------------------	-------------------	--------------------	------------------	--------------------	----------------------------	----------

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

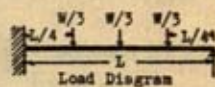
$\frac{117W}{128}$	$\frac{11W}{128}$	$\frac{WL}{6.10}$	$\frac{WL}{6.10}$	$\frac{WL}{15.5}$	$\frac{1.57 WL^3}{384 EI}$	0.186 L
--------------------	-------------------	-------------------	-------------------	-------------------	----------------------------	---------

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

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



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

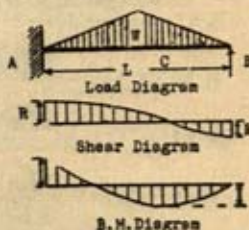


$$R_A = \frac{21}{32} W, R_B = \frac{11}{32} W$$

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

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

$$\text{Max: Def} = \frac{WL^3}{1395 EI}$$

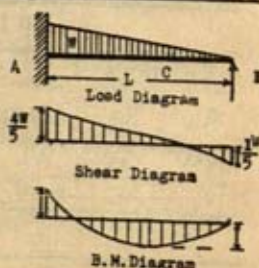


$$R_A = \frac{4}{5} W, R_B = \frac{1}{5} W$$

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

$$\text{BM at C} = +\frac{1}{167} WL$$

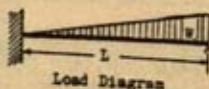
$$\text{Max: Def} = \frac{WL^3}{210 EI}$$



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

If there is no support at the end:

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





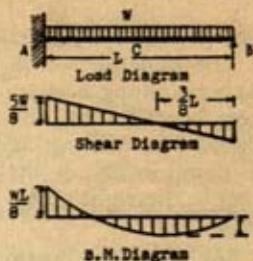
$$R_A = \frac{5}{8} W, \quad R_B = \frac{3}{8} W$$

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

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

Point of contraflexure is at  $0.25L$  from A.

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



(a) Uniformly distributed load over two equal spans:

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

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

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

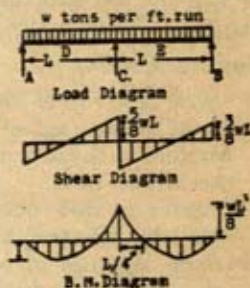
$$\text{BM at D or E} = +\frac{9}{128} \times w.L^2$$

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

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

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

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



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

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

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

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

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

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

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

$$\text{BM at C} = -0.094 WL$$

*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:

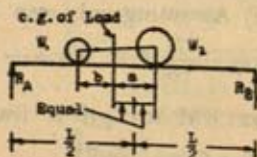
$$R_A = W_1 + \frac{W_2[L - (a + b)]}{L}$$

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.

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## SECTION 4

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## 1. ROUND AND SQUARE STEEL BARS

## Areas and Weights

Dia. or Side  in.	Round Bars			Square Bars	
	Sectional Area sq. in.	Circum- ference in.	Wt. per ft. lb.	Wt. per ft. lb.	Sectional Area sq. in.
$\frac{3}{16}$	.028	.590	.094	.119	.035
$\frac{1}{4}$	.049	.785	.167	.213	.063
$\frac{5}{16}$	.077	.982	.261	.332	.098
$\frac{3}{8}$	.110	1.178	.376	.478	.141
$\frac{7}{16}$	.150	1.375	.511	.651	.191
$\frac{1}{2}$	.196	1.571	.668	.849	.250
$\frac{9}{16}$	.248	1.767	.845	1.076	.316
$\frac{5}{8}$	.307	1.964	1.043	1.328	.391
$\frac{11}{16}$	.371	2.160	1.262	1.607	.472
$\frac{3}{4}$	.442	2.356	1.502	1.912	.563
$\frac{13}{16}$	.519	2.553	1.763	2.245	.660
$\frac{7}{8}$	.601	2.749	2.044	2.603	.766
$\frac{15}{16}$	.690	2.945	2.347	2.988	.879
1	.785	3.142	2.670	3.400	1.000
$1\frac{1}{8}$	.994	3.534	3.321	4.303	1.266
$1\frac{1}{4}$	1.227	3.927	4.172	5.312	1.565
$1\frac{3}{8}$	1.485	4.320	5.049	6.428	
$1\frac{1}{2}$	1.767	4.712	6.008	7.650	
$1\frac{3}{4}$	2.073	5.105	7.051	8.978	
$1\frac{7}{8}$	2.405	5.498	8.178	10.41	
$1\frac{7}{8}$	2.761	5.890	9.388	11.95	
2	3.141	6.283	10.68	13.60	
$2\frac{1}{8}$	3.546	6.676	12.06	15.35	
$2\frac{1}{4}$	3.976	7.069	13.52	17.21	
$2\frac{3}{8}$	4.908	7.854	16.69	21.25	

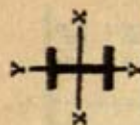
The weight of steel bars and wire is calculated on the basis of 3.40 lbs. /sq. in. of cross-sectional area per ft. run



## 2. OLD BRITISH STANDARD BEAMS (R.S. Joists)

*(Not generally rolled in India)*

Size ins.	Wt. per ft. lbs.	Area sq. ins.	Moment of Inertia		Radii of Gyration		Section Moduli	
			About x-x	About y-y	About x-x	About y-y	About x-x	About y-y
24 x 7½	90	26.50	2443	—	9.61	1.51	204	—
20 x 7½	89	26.19	1673	62.54	7.99	1.55	167.3	16.68
18 x 7	75	22.09	1151	46.56	7.22	1.45	127.9	13.30
16 x 6	62	18.21	725.1	27.14	6.31	1.22	90.63	9.50
14 x 6	57	16.78	533.3	27.95	5.64	1.29	76.19	9.31
14 x 6	46	13.59	442.6	21.45	5.71	1.26	63.22	7.15
10 x 8	70	20.60	344.9	71.67	4.09	1.86	68.98	17.91
10 x 6	42	12.35	211.5	22.95	4.13	1.36	42.30	7.65
9 x 7	58	17.06	229.5	46.3	3.66	1.64	51.00	13.22
6 x 4½	20	5.89	34.71	5.40	2.43	0.96	11.57	2.40
5 x 3½	9	2.65	10.91	0.79	2.03	0.55	4.36	0.63
4½ x 2½	7½	2.28	6.93	—	1.74	0.47	2.92	—
4½ x 2	7	2.06	6.65	0.38	1.80	0.43	2.90	0.38
4 x 3	9½	2.79	7.52	1.28	1.64	0.68	3.76	0.85
4 x 2½	7½	2.21	5.72	—	1.61	0.62	2.86	—
4 x 2	5½	1.69	3.88	—	1.52	0.41	1.94	—
3 x 3	8½	2.52	3.81	1.25	1.23	0.70	2.54	0.83



## 3. R.S. JOISTS

## Dimensions and Properties

Size ins.	Wt. per ft. lbs.	Area sq. ins.	Moment of Inertia		Radii of Gyration		Section Moduli	
			About x-x	About y-y	x-x	y-y	About x-x	About y-y
24 × 7½	100	29.40	2654	66.92	9.50	1.50	221.1	17.8
• 24 × 7½	95	27.94	2533	62.54	9.52	1.50	211.1	16.7
• 22 × 7	75	22.06	1677	41.07	8.72	1.36	152.4	11.7
20 × 6½	65	19.12	1226	32.56	8.01	1.31	122.6	10.0
• 18 × 8	80	23.53	1292	69.43	7.41	1.72	143.6	17.4
18 × 6	55	16.18	841.8	23.64	7.21	1.21	93.5	7.9
• 16 × 8	75	22.06	973.9	68.30	6.64	1.76	121.7	17.1
16 × 6	50	14.71	618.1	22.47	6.48	1.24	77.3	7.5
• 15 × 6	59	17.35	628.9	28.22	6.02	1.27	83.8	9.4
15 × 6	45	13.24	491.9	19.87	6.10	1.23	65.6	6.6
15 × 5	42	12.36	428.5	11.81	5.89	0.98	57.1	4.7
• 14 × 8	70	20.59	705.6	66.67	5.85	1.80	100.8	16.7
• 14 × 5½	40	11.77	377.1	14.79	5.66	1.12	53.9	5.4
• 13 × 5	35	10.30	283.5	10.82	5.25	1.03	43.6	4.3
• 12 × 8	65	19.21	487.8	65.18	5.05	1.85	81.3	16.3
12 × 6	44	13.00	316.8	22.12	4.94	1.30	52.8	7.4
• 12 × 6	54	15.89	375.8	28.28	4.86	1.33	62.6	9.4
• 12 × 5	32	9.45	221.1	9.69	4.84	1.01	36.8	3.9

12x5	30	8.83	206.9	8.77	4.84	1.00	34.5	3.5
*10x8	35	16.18	288.7	54.74	4.22	1.84	57.7	13.7
10x6	40	11.77	204.8	21.76	4.17	1.36	40.96	7.25
10x5	30	8.85	146.2	9.73	4.06	1.05	29.3	3.9
10x4½	25	7.35	122.3	6.49	4.08	0.94	24.5	2.9
*9x7	50	14.17	208.13	40.17	3.76	1.65	46.25	11.48
9x4	21	6.18	81.1	4.15	3.62	0.82	18.0	2.1
8x6	35	10.30	115.1	19.54	3.34	1.38	28.8	6.5
*8x5	28	8.28	89.7	10.19	3.29	1.11	22.4	4.1
8x4	18	5.30	55.6	3.51	3.24	0.81	13.9	1.7
7x4	16	4.75	39.5	3.37	2.89	0.84	11.3	1.7
*7x3½	15	4.42	35.9	2.41	2.85	0.74	10.3	1.4
*6x5	25	7.37	43.7	9.10	2.44	1.11	14.6	3.6
6x3	12	3.53	21.0	1.46	2.44	0.64	7.0	0.97
5x4½	18	5.29	22.7	5.66	2.07	1.03	9.1	2.5
*5x4½	20	5.88	25.0	6.59	2.06	0.06	10.0	2.9
5x3	11	3.26	13.7	1.45	2.05	0.67	5.5	0.97
4½x1½	6½	1.91	6.73	0.26	1.88	0.37	2.8	0.30
*4x3	10	2.94	7.80	1.33	1.63	0.67	3.9	0.88
4x1½	5	1.47	3.66	0.19	1.58	0.36	1.8	0.21
*3x1½	4	1.18	1.66	0.13	1.19	0.33	1.1	0.17

\*Marked sections are not generally rolled in India.



## 4. SAFE DISTRIBUTED LOADS

*Based on working stress*

Size ins.	Weight per ft. in lbs.	Spans in						
		10	12	14	16	18	20	22
24×7½	100		98.3	84.2	73.7	65.5	59.0	53.6
24×7½	95		93.8	80.4	70.3	62.5	56.1	51.1
22×7	75		67.7	58.0	50.8	45.1	40.6	36.9
20×7½	89	89.2	74.3	63.7	55.7	49.5	44.6	40.5
20×6½	65		54.4	46.7	40.8	36.3	32.6	29.7
18×8	80		63.8	54.6	47.8	42.5	38.2	34.8
18×7	75	68.2	56.8	48.7	42.6	37.8	34.1	31.0
18×6	55	49.8	41.5	35.6	31.1	27.7	24.9	22.6
16×8	75	64.9	54.1	46.3	40.5	36.0	32.4	29.5
16×6	62	48.3	40.2	34.5	30.2	26.8	24.1	21.9
16×6	50	41.2	34.3	29.4	25.7	22.8	20.6	18.7
15×6	59	44.7	37.3	31.9	28.0	24.8	22.4	20.3
15×6	45	34.9	29.1	24.9	21.8	19.4	17.4	15.9
15×5	42	30.4	25.3	21.7	19.0	16.9	15.2	13.8
14×8	70	53.7	44.7	38.3	33.5	29.8	26.8	24.4
14×6	57	40.6	33.8	29.0	25.3	22.5	20.3	18.4
14×6	46	33.7	28.0	24.0	21.0	18.7	16.8	15.3
14×5½	40	28.7	23.9	20.5	18.0	16.0	14.4	13.1
13×5	35	23.2	19.3	16.6	14.5	12.9	11.6	10.5
12×8	65	43.3	36.1	30.9	27.0	24.0	21.6	19.7
12×6	54	33.4	27.8	23.3	20.8	18.5	16.7	15.1
12×6	44	18.1	23.4	20.1	17.5	15.6	14.0	12.7
12×5	39	23.2	19.3	16.5	14.5	12.8	11.6	10.5
12×5	32	29.6	16.3	14.0	12.2	10.9	9.8	8.9
12×5	30	18.4	15.3	13.1	11.5	10.2	9.2	8.3

## ON R.S. JOISTS AS BEAMS IN TONS

of 8 tons per sq. in.

feet

24	26	28	30	32	36	40	45	50
49.1	45.4	42.1	39.3	36.9	32.8	29.5	26.2	23.6
46.9	43.2	40.2	37.5	35.1	31.2	28.1	25.0	22.5
33.8	31.2	29.0	27.0	25.4	22.5	20.3	18.1	..
37.1	34.3	31.8	29.7	27.8	24.7	22.3	20.0	..
27.2	25.1	23.3	21.7	20.4	18.1	16.3	14.5	..
31.9	29.4	27.3	25.5	23.9	21.2	17.2		
28.4	26.2	24.3	22.7	21.3	18.9	15.3		
20.7	19.1	17.8	16.6	15.5	13.8	11.2		
27.0	24.9	23.1	21.6	20.2	16.0	12.9		
20.1	18.5	17.2	16.1	15.1	11.9	9.6		
17.1	15.8	14.7	13.7	12.8	10.1	8.2		
18.6	17.2	16.0	14.9	13.1	10.3			
14.5	13.4	12.4	11.6	10.2	8.0			
12.6	11.7	10.8	10.1	8.9	7.0			
22.3	20.6	19.1	16.7	14.7				
16.9	15.6	14.5	12.6	11.1				
14.0	12.9	12.0	10.4	9.2				
12.0	11.0	10.3	8.9	7.8				
9.6	8.9	7.7	6.7					
18.0	15.3	13.2						
13.9	11.8	10.2						
11.7	9.9	8.6						
9.6	8.2	7.1						
8.1	6.9	6.0						
7.6	6.5	5.6						

## SAFE DISTRIBUTED LOADS

Size ins.	Weight per ft. in lbs.	Spans in						
		3	4	5	6	7	8	9
10×8	70	..	..	..	..	52.5	46.0	40.8
10×8	55	..	..	..	..	..	..	34.2
10×6	42	..	..	..	37.6	32.2	28.2	25.1
10×6	40	..	..	..	..	31.2	27.3	24.2
10×5	30	..	..	31.2	26.0	22.2	19.5	17.3
10×4½	25	..	..	26.1	21.7	18.6	16.3	14.4
9×7	58	..	..	..	44.6	38.8	34.0	30.2
9×7	50	..	..	..	..	..	30.8	27.4
9×4	21	..	24.0	19.2	16.0	13.7	12.0	10.6
8×6	35	..	..	..	25.5	21.9	19.1	17.0
8×5	28	..	..	23.9	19.9	17.0	14.9	13.2
8×4	18	..	18.5	14.8	12.3	10.5	9.2	8.2
7×4	16	..	15.0	12.0	10.0	8.6	7.5	6.6
7×3½	15	..	13.7	10.9	9.12	7.8	6.8	6.1
6×5	25	..	19.4	15.5	12.9	11.0	9.7	8.6
6×4½	20	..	15.4	12.3	10.2	8.8	7.7	6.8
6×3	12	..	9.3	7.4	6.2	5.3	4.6	4.1
5×4½	20	..	13.3	10.6	8.8	7.6	6.6	5.9
5×4½	18	..	12.1	9.7	8.1	6.9	6.1	5.4
5×3	11	9.7	7.2	5.8	4.8	4.1	3.6	3.2
5×2½	9	..	5.8	4.7	3.9	3.3	2.9	2.6
4½×1½	6.5	5.0	3.7	3.0	2.5	2.1	1.8	1.6
4×3	10	6.9	5.1	4.1	3.4	2.9	2.5	2.0
4×3	9½	6.7	5.0	4.0	3.3	2.9	2.5	2.0
4×1½	5	3.2	2.4	1.9	1.6	1.3	1.2	.96
3×3	8½	4.3	3.3	2.7	2.2	1.6	1.2	..
3×1½	4	1.9	1.4	1.1	.98	.72	.55	..

The tabulated loads are calculated as uniformly distributed loads. The deflection of the joists has been limited to  $1/325$  of the span, for the full zig-zag line. Tabular loads to the right of this line have been span. The loads are calculated on the assumption that the beams apart not exceeding 20 times the width of the compression flange, per sq. in.



## ON R.S. JOISTS AS BEAMS IN TONS (contd.)

feet

10	11	12	14	16	18	20	22	24
36.8	33.4	30.7	26.3	23.0	20.4	18.4	17.1	15.6
30.7	27.9	25.6	21.9	19.2	17.1	15.3	14.0	12.8
22.6	20.5	18.8	16.1	14.1	12.5	11.3		
21.8	19.8	18.2	15.6	13.6	12.1	10.9	10.0	6.3
15.6	14.1	12.9	11.1	9.7	8.6	7.8	7.1	6.5
							6.0	5.4
13.0	11.8	10.8	9.3	8.1	7.2	6.5		
22.6	20.5	18.8	16.1	14.1	12.5	11.3	..	..
24.6	22.4	20.5	17.6	15.4	13.7	11.1		
9.6	8.7	8.0	6.8	6.0	5.3	4.3		
15.3	13.9	12.7	10.9	9.5	7.5	6.1		
11.9	10.8	9.9	8.5	7.4	5.9	4.7		
7.4	6.7	6.1	5.2	4.6	3.6	2.9		
6.0	5.4	5.0	4.3	3.2	2.6			
5.4	4.9	4.6	3.9	3.0	2.3			
7.7	7.0	6.4	4.7	3.6				
6.1	5.6	5.1	3.7	2.8				
3.7	3.3	3.1	2.2	1.7				
5.3	4.4	3.7						
4.8	4.0	3.4						
2.9	2.4	2.0						
2.3	1.9	1.6						
1.4	1.1	—						
1.6								
1.6								
.78								

on a simply supported beam and include the weight of the joists, responding to a ratio of span to depth of 24. This limit is shown by reduced in value so that the deflection does not exceed  $\frac{1}{250}$ th of the have adequate lateral supports to the compression flange at distances The loads have been worked out for an extreme fibre stress of 8 tons

## 5. R.S. JOISTS—

*Based on Working Stress*

Size in.	Wt. per ft. in lbs.	Safe Concentric Loads in Tons								
		3	4	5	6	7	8	9	10	11
18×8	80				154	149	142	135	127	118
18×6	55				95.6	88.0	79.5	70.6	62.2	54.6
16×8	75				145	140	135	128	121	113
16×6	50				87.8	81.2	73.8	66.0	58.4	51.4
15×6	59				105	97.2	88.8	79.8	71.0	62.8
15×6	45				78.8	72.7	66	58.9	52.0	45.8
15×5	42				64.6	56.4	48.3	41.1	35.0	30.0
14×8	70				136	132	127	121	114	107
14×5½	40				67.0	60.6	53.7	47.0	40.8	35.5
13×5	35				55.7	49.3	42.8	36.7	31.5	27.1
12×8	65				127	123	119	113	108	102
12×6	54				97.5	91.2	84.1	76.4	68.6	61.1
12×6	44				79.1	73.8	67.7	61.2	54.6	48.5
12×5	32				50.4	44.4	38.3	32.8	28.1	24.1
12×5	30				46.7	41.0	35.3	30.1	25.7	22.1
10×8	55	116	113	110	107	104	100	96.2	91.3	85.8
10×6	40	82.5	79.7	76.5	72.8	68.3	63.3	57.7	52.1	46.6
10×5	30	60.2	57.0	53.2	48.5	43.1	37.6	32.4	27.9	..
10×4½	25	49.1	45.9	42.0	37.2	32.0	27.2	23.0	19.4	..
9×7	50	104	102	99.2	95.9	92.1	87.7	82.8	77.5	..
9×4	21	40.2	36.8	32.4	27.5	22.8	18.7	15.5	13.0	..
8×6	35	72.3	69.9	67.2	64.0	60.2	55.9	51.1	46.2	..
8×5	28	56.7	54.1	50.8	46.8	42.3	37.4	32.6	28.3	..
8×4	18	34.4	31.3	27.5	23.2	19.2	15.8	13.1	10.9	..
7×4	16	31.0	28.5	25.3	21.6	18.0	14.9	12.4	10.4	..
7×3½	15	28.0	25.0	21.2	17.4	14.0	11.4	9.3	..	..
6×5	25	50.5	48.1	45.2	41.7	37.6	33.3	29.0	25.2	..
6×3	12	21.3	18.1	14.5	11.4	8.9	7.1	..	..	..
5×4½	20	40.0	38.0	35.4	32.4	28.9	25.2	21.8	18.8	..
5×3	11	20.0	17.3	14.1	11.2	8.9	7.1	..	..	..
4½×1½	6.5	7.5	4.9	..	..	..	..	..	..	..
4×3	10	18.0	15.6	12.7	10.1	8.0	6.4	..	..	..
4×1½	5	5.6	3.6	..	..	..	..	..	..	..
3×1½	4	3.9	2.5	..	..	..	..	..	..	..

# COLUMNS—Safe Loads on of 8 tons per sq. in.

for Effective Heights in Feet.

12	13	14	15	16	18	20	22
109	101	92.3	84.2	76.9	64.1	53.9	
48.0	42.2	37.3	33.2				
105	97.2	89.0	81.5	74.5	62.3	52.5	44.6
45.3	39.9	35.3	31.4	..			
55.4	49.0	43.4	38.7	..			
40.2	35.5	31.4	27.9	..			
25.9							
100	92.8	85.5	78.4	71.8	60.2	50.9	43.3
30.9	27.1	23.8	..				
23.5							
95.5	88.7	81.9	75.4	69.2	58.3	49.4	42.1
54.2	48.1	42.9	38.3	34.3			
42.9	38.0	33.8	30.1	27.0	..		
20.8							
19.1							
80.4	74.6	68.9	63.5	58.2	49	41.4	35.3
41.4	37.1	32.9	29.5	26.4	..		
20.8							
..	..	..					
65.8	60.2	54.7	49.8	45.1	37.4	31.3	
..	..	..		..	..	..	
37.0	33.2	29.5	26.6	23.7			
21.4							
19.0							
14.0							

Note: The loads tabulated are safe concentric loads calculated on the least radius of gyration in accordance with the B.S.S. formula and safe working stresses as detailed in Section 10—(8 ton/sq. in.) for mild steel. Gross areas of the sections have been taken without any deduction being made for rivet holes.



## 6. SAFE LOADS FOR ANGLES

## For the Design of Steel

Based on Working Stress

Size ins.	Wt. in lbs. per ft. length	Area in sq. ins.	Ties in Tons		Safe load for L struts in Tons on a length of—							
			L	┐	2'	3'	4'	5'	6'	7'	8'	
$1\frac{1}{2} \times 1\frac{1}{2} \times \frac{1}{2}$	1.91	.56	2.5	6.5	1.1	.63	.54					
$1\frac{1}{2} \times 1\frac{1}{2} \times \frac{3}{8}$	2.34	.69	3.1	8.4	1.7	1.1	.72	.49				
$1\frac{1}{2} \times 1\frac{1}{2} \times \frac{1}{4}$	2.76	.81	3.5	9.8	2.2	1.6	1.1	.74				
$2 \times 1\frac{1}{2} \times \frac{1}{2}$	2.76	.81	3.5	9.3	2.4	1.6	1.1	.68				
$2 \times 2 \times \frac{1}{4}$	3.19	.94	4.0	11.2	2.7	2.1	1.5	1.1	.75			
$2\frac{1}{2} \times 1\frac{1}{2} \times \frac{1}{2}$	3.19	.94	4.2	11.2	3.1	2.1	1.4	.94				
$1\frac{1}{2} \times 1\frac{1}{2} \times \frac{5}{16}$	3.39	1.00	4.4	12.0	2.8	2.0	1.4	.93				
$2 \times 1\frac{1}{2} \times \frac{5}{16}$	3.39	1.00	4.4	11.4	3.0	2.0	1.3	.85				
$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{2}$	3.61	1.06	4.7	13.0	3.2	2.6	2.0	1.5	1.1	.85		
$2\frac{1}{2} \times 2 \times \frac{1}{2}$	3.61	1.06	4.7	12.6	3.5	2.8	2.0	1.5	1.1	.85		
$2 \times 2 \times \frac{5}{16}$	3.92	1.15	4.9	13.8	3.4	2.6	1.8	1.3	.93			
$2\frac{1}{2} \times 1\frac{1}{2} \times \frac{5}{16}$	3.92	1.15	5.2	13.4	3.8	2.6	1.7	1.2				
$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}$	4.04	1.19	5.1	14.5	3.6	3.1	2.4	1.8	1.4	1.1		
$3 \times 2 \times \frac{1}{2}$	4.04	1.19	5.1	14.0	4.3	3.5	2.5	1.9	1.4	1.0		
$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{5}{16}$	4.45	1.31	5.8	16.1	4.0	3.2	2.4	1.8	1.4	1.1		
$2\frac{1}{2} \times 2 \times \frac{5}{16}$	4.45	1.31	5.8	15.7	4.3	3.5	2.5	1.9	1.4	1.1		
$3 \times 2\frac{1}{2} \times \frac{1}{2}$	4.46	1.31	5.6	15.9	4.5	3.8	3.1	2.5	1.9	1.5		
$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{2}$	4.46	1.31	5.6	16.3	4.1	3.6	2.9	2.2	1.8	1.4		
$2 \times 2 \times \frac{3}{4}$	4.62	1.36	5.6	16.2	4.1	3.1	2.2	1.6	1.1			
$3 \times 3 \times \frac{1}{2}$	4.89	1.44	6.1	17.8	4.6	4.1	3.4	2.8	2.3	1.8	1.5	
$3\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{2}$	4.89	1.44	6.5	17.8	5.2	4.5	3.7	3.0	2.3	1.9	1.5	
$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{5}{16}$	4.98	1.46	6.4	18.0	4.5	3.7	3.0	2.3	1.7	1.4	..	
$3 \times 2 \times \frac{5}{16}$	4.98	1.46	6.4	17.4	5.3	4.3	3.2	2.3	1.8	1.3	..	
$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{3}{4}$	5.26	1.55	6.7	19.0	4.7	3.9	2.9	2.2	1.6	1.3		
$2\frac{1}{2} \times 2 \times \frac{3}{4}$	5.26	1.55	6.3	19.0	5.2	4.2	3.1	2.3	1.7	1.3		
$3\frac{1}{2} \times 3 \times \frac{1}{2}$	5.32	1.56	7.1	19.6	5.4	4.9	4.2	3.6	2.9	2.3	1.9	
$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{5}{16}$	5.60	1.62	6.8	20.2	5.1	4.5	3.7	2.8	2.2	1.7	1.4	
$3 \times 2\frac{1}{2} \times \frac{5}{16}$	5.51	1.62	6.8	19.6	5.6	4.8	3.9	3.1	2.4	1.9	1.4	
$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{3}{4}$	5.90	1.73	7.9	21.0	5.5	4.6	3.6	2.8	2.1	1.6	..	

## AS STRUTS AND TIES

## Trusses Generally

of 8 tons per sq. in.

Safe load for  $\square$  struts in Tons on a length of —


2'	3'	4'	5'	6'	7'	8'	9'	10'	11'	12'		
	4.3 6.3 8.3	2.9 4.8 6.8	2.2 3.6 5.3	1.5 2.7 4.2		2.1 3.2 4.9	2.5 3.1 3.9	2.1 2.6				
10.8	9.7	8.1	6.4	4.9	3.9	3.1	2.5	2.1				
12.3	10.9	8.9	6.9	5.3	4.1	3.2	2.6					
12.7	11.5	9.9	8.1	6.4	5.0	4.0	3.3	2.7				
	10.2	6.3	6.5	5.1	4.0	3.2						
	11.0	9.6	8.1	6.4	4.7	4.3	3.5	3.0				
14.3	13.0	11.3	9.2	7.3	5.8	4.6	3.7	3.1	2.6			
14.6	13.6	12.3	10.6	8.8	7.2	5.8	4.8	4.0	2.9			
15.2	13.4	11.0	8.5	6.5	5.0	4.0	3.2					
	12.8	11.0	9.4	7.5	5.4	5.0	4.1	3.5				
16.3	15.1	13.5	11.5	9.5	7.7	6.2	5.1	4.2	3.6	3.0		
16.6	15.7	14.5	13.0	11.3	9.5	8.0	6.7	5.6	4.9	4.1		
17.7	16.1	13.9	11.3	9.0	7.1	5.7	4.6	3.8	3.2			
18.0	16.7	15.0	13.0	10.7	8.7	7.1	5.8	4.9	4.2	3.5		
18.4	17.5	16.3	14.9	13.1	11.3	9.6	8.1	6.8	5.9	5.0		
	15.8	14.5	13.0	11.4	9.8	8.5	7.3	6.2	5.3	4.6		
	14.8	12.7	10.4	8.4	6.7	5.5	4.5					
1.3	20.1	19.1	17.8	16.1	14.1	12.0	10.1	8.5	7.2	6.1	5.3	4.0
1.2	20.5	19.6	18.6	17.4	15.9	14.2	12.5	10.8	9.3	8.1	7.0	5.4
..	20.1	18.6	16.7	14.2	11.7	9.7	7.7	6.3	5.2	4.4	3.8	..
..	20.5	19.4	18.0	16.2	14.2	12.0	10.1	8.4	7.1	6.2	5.2	3.9
..	20.9	19.0	16.4	13.3	10.5	8.3	6.7	5.4	4.5	3.7	..	..
..	21.2	19.7	17.7	15.2	12.6	10.2	8.3	6.8	5.7	4.1	..	..
1.6	20.8	19.5	18.6	17.6	16.4	14.9	13.6	12.0	10.8	9.6	8.4	6.7
..	20.6	19.5	18.0	16.1	14.1	12.1	10.5	9.0	7.6	6.6	5.6	4.0
..	22.7	21.6	20.1	18.3	16.2	13.9	11.7	9.9	8.3	7.2	6.1	4.6
..	23.7	22.0	19.6	16.7	13.6	11.0	8.9	7.3	6.1	5.1	4.4	..

## SAFE LOADS FOR ANGLES



Size ins.	Wt. in. lbs. per ft. length	Area in sq. ins.	Ties in Ton		Safe load for L struts in Tons on a length of—							
			L	U	2'	3'	4'	5'	6'	7'	8'	
3×2× $\frac{3}{16}$	5.90	1.73	7.8	20.4	6.3	5.1	3.8	2.8	2.1	1.5	..	
3 $\frac{1}{2}$ ×2 $\frac{1}{2}$ × $\frac{5}{16}$	6.04	1.78	8.1	22.0	6.5	5.7	4.7	3.8	2.9	2.3	1.9	
3×3× $\frac{5}{16}$	6.04	1.78	7.5	22.0	5.7	5.1	4.3	3.5	2.8	2.3	1.9	
3×2 $\frac{1}{2}$ × $\frac{5}{16}$	6.54	1.92	8.6	23.1	6.7	5.7	4.7	3.7	2.8	2.2	1.7	
2 $\frac{1}{2}$ ×2 $\frac{1}{2}$ × $\frac{5}{16}$	6.53	1.92	8.6	23.8	6.1	5.3	4.4	3.4	2.7	2.1	1.7	
3 $\frac{1}{2}$ ×3× $\frac{5}{16}$	6.58	1.93	8.6	24.4	6.7	6.1	5.2	4.4	3.6	2.9	2.3	
3 $\frac{1}{2}$ ×3 $\frac{1}{2}$ × $\frac{5}{16}$	7.11	2.09	9.0	26.6	6.8	6.2	5.6	4.8	4.0	3.3	2.7	
4×3× $\frac{5}{16}$	7.11	2.09	9.6	26.6	7.6	6.9	5.9	5.0	4.1	3.2	2.7	
3×3× $\frac{3}{8}$	7.17	2.11	9.3	26.0	6.8	6.2	5.2	4.2	3.4	2.7	2.3	
3 $\frac{1}{2}$ ×2 $\frac{1}{2}$ × $\frac{3}{8}$	7.17	2.11	10.0	26.0	7.8	6.8	5.6	4.5	3.4	2.8	2.3	
4×3 $\frac{1}{2}$ × $\frac{5}{16}$	7.64	2.25	10.8	29.1	7.8	7.2	6.6	5.7	4.8	4.1	3.3	
2 $\frac{1}{2}$ ×2 $\frac{1}{2}$ × $\frac{3}{8}$	7.65	2.25	9.8	27.1	7.2	6.1	4.8	3.7	2.8	2.2	..	
3×2× $\frac{3}{8}$	7.65	2.25	10.3	26.2	8.5	6.9	5.0	3.7	2.8	2.1	..	
3 $\frac{1}{2}$ ×3× $\frac{3}{8}$	7.81	2.30	10.2	28.7	8.0	7.3	6.3	5.3	4.3	3.4	2.8	
5×3× $\frac{5}{16}$	8.17	2.40	12.1	31.4	9.6	8.6	7.6	6.3	5.1	4.1	3.3	
4×4× $\frac{5}{16}$	8.17	2.40	10.8	31.4	7.8	7.3	6.8	6.0	5.1	4.4	3.7	
4×3× $\frac{3}{8}$	8.45	2.49	11.5	31.5	9.2	8.3	7.3	6.1	4.9	3.9	3.2	
3 $\frac{1}{2}$ ×3 $\frac{1}{2}$ × $\frac{3}{8}$	8.45	2.49	10.9	31.5	8.1	7.5	6.7	5.7	4.8	4.0	3.3	
3×2 $\frac{1}{2}$ × $\frac{3}{8}$	8.50	2.50	11.2	30.0	8.9	7.6	6.2	4.9	3.7	3.0	2.3	
2 $\frac{1}{2}$ ×2 $\frac{1}{2}$ × $\frac{3}{8}$	8.50	2.50	11.2	30.9	8.2	7.1	5.8	4.5	3.6	2.8	2.3	
4×3 $\frac{1}{2}$ × $\frac{3}{8}$	9.09	2.67	12.8	34.5	9.4	8.7	7.9	6.9	5.8	4.9	4.0	
3×3× $\frac{1}{2}$	9.35	2.75	12.2	33.7	9.1	8.2	6.9	5.6	4.5	3.6	3.1	
3 $\frac{1}{2}$ ×2 $\frac{1}{2}$ × $\frac{1}{2}$	9.35	2.75	13.1	33.6	10.4	9.0	7.4	6.0	4.5	3.7	3.0	
4×4× $\frac{1}{2}$	9.73	2.86	13.0	37.1	9.4	8.8	8.2	7.2	6.2	5.3	4.5	
5×3× $\frac{1}{2}$	9.73	2.86	14.4	37.1	11.5	10.4	9.0	7.6	6.1	4.9	4.0	
4×3× $\frac{7}{16}$	9.76	2.88	13.5	36.6	10.7	9.7	8.4	7.1	5.7	4.6	3.7	
3 $\frac{1}{2}$ ×3 $\frac{1}{2}$ × $\frac{7}{16}$	9.76	2.88	12.8	36.6	9.5	8.8	7.8	6.7	5.6	4.6	3.8	
3 $\frac{1}{2}$ ×3× $\frac{7}{16}$	10.20	3.00	14.0	33.1	10.7	9.7	8.4	7.1	5.7	4.6	3.7	
5×3 $\frac{1}{2}$ × $\frac{7}{16}$	10.37	3.05	15.1	40.0	11.7	11.0	9.9	8.8	7.6	6.3	5.3	



## AS STRUTS &amp; TIES (Contd.)

Safe load for  struts in Tons on a length of													
9'	2'	3'	4'	5'	6'	7'	8'	9'	10'	11'	12'	14'	
..	24.3	23.1	21.5	19.4	17.0	14.5	12.2	10.3	8.7	7.6	6.4	4.8	
1.5	25.3	24.3	23.1	21.6	19.9	17.8	15.7	13.6	11.7	10.3	8.8	6.8	
1.6	24.9	23.5	21.9	19.8	17.3	14.7	12.3	10.4	8.7	7.4	6.4	4.8	
..	27.0	25.6	23.8	21.6	19.1	16.3	13.8	11.6	9.8	8.5	7.2	5.4	
..	23.5	23.0	21.2	19.1	16.7	14.3	12.5	10.6	9.0	7.8	6.6	..	
1.9	..	..	21.5	23.5	21.6	19.3	17.0	14.8	12.7	11.2	9.6	7.3	
2.3	..	..	27.0	25.2	23.0	20.5	17.9	15.5	13.3	11.7	10.0	7.6	
2.2	..	..	28.0	26.7	25.1	23.3	21.2	19.0	16.9	15.0	13.1	10.3	
1.9	29.5	28.0	26.0	23.5	20.5	17.5	14.7	12.3	10.4	8.8	7.6	5.7	
1.8	30.1	28.9	27.5	25.7	23.7	21.3	18.7	16.3	14.1	12.4	10.6	8.2	
2.8	..	..	30.0	28.5	26.8	24.8	22.5	20.2	17.8	15.8	13.8	10.8	
..	..	26.5	23.6	21.0	18.2	15.1	12.7	10.6	9.1	7.8	..	..	
..	..	27.2	26.0	23.6	21.5	19.1	16.8	14.6	12.7	11.2	9.5	7.5	
2.4	29.8	27.9	25.6	22.9	20.1	17.4	15.0	13.2	11.3	8.7	6.8	..	
2.8	31.9	30.2	28.3	25.9	23.4	20.7	18.2	16.1	14.0	10.9	8.6	..	
3.2	29.1	27.6	26.1	24.4	22.4	20.4	18.3	16.6	14.8	12.0	..	..	
2.7	33.2	31.7	29.8	27.6	25.1	22.5	20.0	17.8	15.5	12.1	9.7	..	
2.8	32.1	30.0	27.4	24.4	21.3	18.4	15.9	13.9	11.9	9.1	7.1	..	
..	28.8	26.3	23.8	21.2	18.6	16.2	14.2	12.4	10.6	8.4	..	..	
..	27.6	25.0	21.8	18.6	16.2	13.8	11.8	10.2	8.7	..	..	..	
3.4	35.6	33.9	31.8	29.4	26.7	23.9	21.1	18.7	16.3	12.8	10.1	..	
2.5	33.6	30.2	26.2	22.2	18.5	15.5	13.0	11.1	9.5	7.2	..	..	
2.4	32.6	31.0	29.0	26.4	24.0	21.0	19.0	17.0	14.8	11.7	..	..	
3.8	38.0	36.1	33.8	31.1	28.1	24.9	22.0	19.5	16.9	13.2	10.4	..	
3.4	38.0	36.1	33.8	31.1	28.1	25.0	22.0	19.5	17.0	13.1	10.5	..	
3.1	35.3	33.8	31.9	29.6	27.4	25.8	23.2	21.4	18.6	15.0	..	..	
3.2	34.1	31.9	29.6	26.8	24.3	21.4	18.7	16.8	15.0	12.1	..	..	
3.1	36.8	36.2	33.1	29.6	25.9	22.4	19.3	16.9	14.4	11.1	8.7	..	
4.4	41.8	40.4	38.8	36.0	34.7	32.2	29.5	26.9	24.2	19.6	15.9	..	

## SAFE LOADS FOR ANGLES

Size ins.	Wt. in lbs. per ft. length	Area in sq. ins.	Ties in Tons		Safe load for L struts in Tons on a length of—							
					2'	3'	4'	5'	6'	7'	8'	
$4 \times 3\frac{1}{2} \times \frac{7}{16}$	10.60	3.10	15.0	39.6	10.9	10.1	9.2	8.0	6.7	5.7	4.7	
$5 \times 4 \times \frac{1}{2}$	11.00	3.24	15.9	43.0	11.8	11.2	10.4	9.5	8.4	7.2	6.3	
$6 \times 3 \times \frac{1}{2}$	11.00	3.24	17.2	43.0	13.8	12.5	10.8	9.1	7.3	5.9	4.8	
$4 \times 3 \times \frac{1}{2}$	11.05	3.25	15.9	41.1	12.2	11.1	9.6	8.1	6.5	5.2	4.2	
$3\frac{1}{2} \times 3\frac{1}{2} \times \frac{1}{2}$	11.05	3.25	15.0	41.1	10.8	10.0	8.9	7.6	6.4	5.3	4.4	
$4 \times 4 \times \frac{7}{16}$	11.05	3.25	15.1	43.0	10.8	10.3	9.6	8.4	7.3	6.2	5.3	
$5 \times 3 \times \frac{7}{16}$	11.25	3.32	16.9	43.3	13.4	12.1	10.4	8.9	7.1	5.7	4.6	
$4 \times 3\frac{1}{2} \times \frac{1}{2}$	11.91	3.51	16.8	45.0	12.4	11.6	10.6	9.1	7.7	6.5	5.4	

Notes on above Table

**Struts:** All struts calculated according to British Standard Specifications for half fixed ends such as in trusses, with factor of safety of 3.3, by Claxton Fidler's formula. In the case of single Ls as struts, area of the attached leg only is considered and in the double angle struts, gross area effective. In the case of double unequal Ls, legs are considered connected together at intervals. Longer legs connected in all cases.

The centre of gravity of an angle is approximately at  $0.28 \times \text{leg}$  from the back of the other leg.

**Ties:** Single Ties: Effective area = net area of attached leg +  $\frac{1}{2}$  area of other leg. Double ties: Net area effective. Rivet deducted from Ties.

## 7. WEIGHT OF HOOP IRON

Breadth ins.	Gauge B. W.	Wt. per 100 ft. lbs.	Breadth ins.	Gauge B. W.	Wt. per 100 ft. lbs.
$\frac{1}{2}$	21	6.66	$1\frac{1}{2}$	15	33.0
$\frac{3}{4}$	20	6.90	$1\frac{1}{4}$	15	36.0
$\frac{1}{2}$	20	8.75	$1\frac{1}{2}$	14	48.4
1	20	13.00	$1\frac{1}{4}$	14	40.4
$\frac{3}{4}$	19	12.00	2	13	63.4
1	18	16.36	$2\frac{1}{4}$	13	71.4
$1\frac{1}{4}$	18	25.00	2	12	73.4
$1\frac{1}{2}$	17	21.0	$2\frac{1}{4}$	12	91.0
$1\frac{3}{4}$	16	27.00	3	11	125

## AS STRUTS &amp; TIES (Contd.)

Safe load for  struts in Tons on a length of—

9'	4'	5	6	7	8'	9'	10'	11'	12'	14'	16'
3.9	38.1	36.4	34.5	32.0	29.7	27.8	25.1	23.2	20.2	16.2	..
5.3	44.7	43.3	41.7	39.9	37.8	35.4	32.8	30.1	27.4	22.5	18.4
4.0	42.9	40.6	37.9	34.8	31.3	27.7	24.4	21.6	18.7	14.5	11.5
3.6	43.4	41.3	38.8	35.9	32.6	29.1	25.8	22.9	20.0	15.6	12.4
3.7	41.8	38.9	35.4	31.4	27.3	23.5	20.2	17.5	15.0	11.5	9.0
4.4	40.2	38.2	36.1	33.4	30.9	28.2	25.4	22.9	20.4	16.5	
3.8	40.7	38.8	36.6	34.2	31.8	29.6	26.8	24.7	21.6	17.3	
4.4	46.6	44.3	41.5	38.3	34.7	30.9	27.3	24.2	21.1	16.4	13.0

## 8. SAFE LOADS FOR TEES AS STRUTS

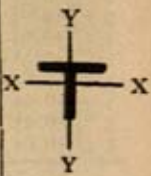
## For Design of Steel Trusses Generally

Size ins.	Wt. per ft. lbs.	Safe Load for Struts in Tons on a length of—						
		3'	4'	5'	6'	7'	8'	10'
$1\frac{1}{2} \times 1\frac{1}{2} \times \frac{1}{2}$	2.35	3.7	2.7	2.0	1.4			
$1\frac{1}{2} \times 1\frac{1}{2} \times \frac{1}{2}$	2.79	4.5	3.5	2.7	2.0			
$1\frac{1}{2} \times 2 \times \frac{1}{2}$	2.79	4.5	3.3	2.4	1.8			
$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{2}$	4.07	5.5	4.3	3.4	2.7	2.1		
$2 \times 2 \times \frac{1}{2}$	4.64	5.3	3.9	2.8	2.1			
$3 \times 3 \times \frac{1}{2}$	4.93	7.2	6.2	5.1	4.2	3.4	2.8	
$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{5}{16}$	5.00	6.7	5.3	4.2	3.3	2.5		
$2\frac{1}{2} \times 2 \times \frac{3}{8}$	5.28	7.2	5.9	4.7	3.6	2.8		
$2 \times 3 \times \frac{1}{2}$	5.92	6.3	4.6	3.3				
$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{3}{8}$	5.92	8.0	6.3	5.0	3.9	3.0		
$3 \times 3 \times \frac{5}{16}$	6.07	8.9	7.7	6.3	5.2	4.2	3.4	
$3 \times 3 \times \frac{3}{8}$	7.20	10.6	9.1	7.5	6.2	5.0	4.0	
$2 \times 3 \times \frac{3}{8}$	7.70	8.2	5.9	4.2				
$3 \times 3 \times \frac{7}{16}$	8.30	12.1	10.5	8.6	7.1	5.7	4.7	
$4 \times 3 \times \frac{3}{8}$	8.48	13.5	12.6	11.4	10.3	8.8	7.7	5.6
$3\frac{1}{2} \times 3\frac{1}{2} \times \frac{3}{8}$	8.49	13.0	11.6	10.0	8.6	7.1	5.8	4.2
$3 \times 3 \times \frac{1}{2}$	9.38	13.7	11.9	9.8	8.1	6.5	5.3	
$3\frac{1}{2} \times 3\frac{1}{2} \times \frac{1}{2}$	11.08	17.0	15.2	13.1	11.2	9.3	7.6	5.6

NOTE:—All struts calculated for half fixed ends by Claxton Fidler's formula. As tees are connected by one gusset 90 per cent. of area has been taken as effective. Factor of safety is 3.3. Working stress in steel is 8 tons/sq. in.



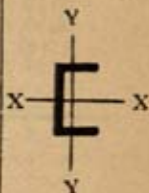
## 9. TEES: Dimensions and Properties

† Size ins.	Wt. per ft. lbs.	Area sq. ins.	Moment of Inertia X-X Max.	Section Moduli X-X. Max.	Radii of Gyration Min.	
4×3× $\frac{3}{8}$	8.49	2.50	1.86	0.83	0.87	
3½×3½× $\frac{3}{8}$	11.08	3.26	3.54	1.44	0.73	
3½×3½× $\frac{1}{2}$	8.49	2.50	2.77	1.10	0.72	
3×3× $\frac{1}{2}$	9.38	2.76	2.17	1.04	0.64	
3×3× $\frac{3}{8}$	7.20	2.12	1.71	0.80	0.62	
2½×2½× $\frac{3}{8}$	5.92	1.74	0.96	0.55	0.52	
2½×2½× $\frac{1}{2}$	4.07	1.20	0.68	0.38	0.50	Sections not generally rolled in India.
2×2× $\frac{1}{2}$	3.21	0.94	0.34	0.24	0.41	
1½×1½× $\frac{1}{2}$	2.79	0.82	0.22	0.18	0.36	
1½×1½× $\frac{3}{8}$	2.36	0.69	0.14	0.13	0.31	
6×3× $\frac{1}{2}$	14.52	4.27	2.63	1.14	0.78	
5×4× $\frac{1}{2}$	14.50	4.27	5.77	1.96	1.08	
5×4× $\frac{3}{8}$	11.06	3.25	4.47	1.49	1.07	
5×3× $\frac{1}{2}$	12.79	3.76	2.52	1.11	0.82	
5×3× $\frac{3}{8}$	9.79	2.88	1.97	0.85	0.83	
4×4× $\frac{1}{2}$	12.78	3.76	5.40	1.90	0.83	
4×4× $\frac{3}{8}$	9.77	2.87	4.19	1.45	0.81	
4×3× $\frac{1}{2}$	11.08	3.26	2.36	1.08	0.85	
3×2½× $\frac{3}{8}$	6.6	1.93	1.01	0.56	0.65	
3×2½× $\frac{1}{2}$	8.52	2.51	1.27	0.73	0.66	
2½×2½× $\frac{1}{2}$	4.07	1.18	0.68	0.38	0.50	
2½×2½× $\frac{3}{16}$	5.01	1.47	0.82	0.46	0.51	
2½×2½× $\frac{1}{4}$	5.28	1.55	0.68	0.44	0.47	
2×2× $\frac{1}{2}$	4.64	1.37	0.47	0.34	0.42	
2×1½× $\frac{1}{2}$	2.79	0.82	0.15	0.14	0.43	
2×1½× $\frac{3}{8}$	4.01	1.18	0.20	0.19	0.41	
1½×2× $\frac{1}{2}$	2.79	0.82	0.31	0.23	0.29	
1½×2× $\frac{3}{16}$	3.41	1.00	0.37	0.28	0.30	
1½×1½× $\frac{3}{16}$	3.40	1.00	0.26	0.22	0.37	
1½×1½× $\frac{1}{4}$	1.81	0.53	0.11	0.03	0.30	
1×1× $\frac{3}{16}$	1.17	0.34	0.03	0.04	0.21	
1×1× $\frac{1}{2}$	0.82	0.24	0.02	0.10	0.19	

† Dimensions are : width of top×total depth×thickness.

## 10. CHANNELS: Dimensions &amp; Properties

Size ins.	Wt. per ft. lbs.	Area sq. ins.	Moment of Inertia X-X Max.	Section Moduli X-X. Max.	Radii of Gyration Y-Y Min.	
*17×4	44.34	13.04	520.2	61.2	1.08	
15×4	36.37	10.70	349.1	45.6	1.12	
*13×4	33.18	9.76	246.9	38.0	1.14	
*12×4	31.33	9.21	200.1	33.3	1.15	
12×3½	26.37	7.76	159.7	26.6	0.96	
*11×3½	26.78	7.88	141.9	25.8	1.00	
*10×4	30.16	8.87	130.7	26.1	1.16	
10×3½	24.46	7.19	109.5	21.9	1.02	
10×3	19.28	5.67	82.7	16.5	0.84	
*9×3½	25.63	7.54	89.3	19.8	1.02	
*9×3½	22.27	6.55	82.6	18.4	1.03	
*9×3	19.91	5.85	67.4	15.0	0.85	
*9×3	17.46	5.14	62.5	13.9	0.86	
*8×3½	20.21	5.94	60.6	15.1	1.04	
8×3	15.96	4.69	46.7	11.7	0.87	
*7×3½	18.28	5.38	42.8	12.2	1.04	
7×3	14.22	4.18	32.8	9.4	0.88	
*6×3½	16.48	4.85	28.9	9.6	1.05	
*6×3	16.51	4.86	26.3	8.8	0.87	
6×3	12.41	3.65	21.3	7.1	0.88	
5×2½	10.22	3.01	11.9	4.8	0.74	
4×2	7.91	2.33	5.4	2.7	0.58	
4×2	7.69	2.09	5.1	2.5	0.58	
*3×1½	5.11	1.50	1.9	1.3	0.44	
3×1½	5.27	1.55	2.0	1.3	0.44	
*3×1½	4.60	1.35	1.8	1.2	0.44	



\*Marked are  
not generally  
rolled in  
India.

## 11. Weight of one sq. ft. of Sheets of Different Metals in lbs. for 1/16 in. (16 B. Gauge) thickness.

C. Iron	W. Iron	Steel	Brass	Copper	Lead	Zinc
2.35	2.50	2.55	2.74	2.68	3.71	2.34

The weight of structural steel plates is calculated on the basis of 40.80 lbs./sq. ft. per in. of thickness.

See page 4/38 .

## 12. FLAT STEEL TIES FOR TENSION MEMBERS

Tensile Strength in Tons at 7.5 tons/sq. in.

Size ins.	1 Rivet deducted of dia.			Size ins.	1 Rivet deducted of dia.		
	$\frac{3}{8}$ "	$\frac{1}{2}$ "	$\frac{7}{8}$ "		$\frac{3}{8}$ "	$\frac{1}{2}$ "	$\frac{7}{8}$ "
$2 \times \frac{3}{16}$	1.76			$3 \times \frac{1}{8}$	4.22	3.98	3.75
$\frac{5}{16}$	2.34			$\frac{5}{16}$	5.27	4.98	4.68
$\frac{3}{4}$	2.93			$\frac{3}{4}$	6.32	5.97	5.63
$\frac{7}{8}$	3.52			$\frac{7}{8}$	8.44	7.97	7.50
$2\frac{1}{2} \times \frac{3}{8}$	3.28	3.05	2.81	$3\frac{1}{2} \times \frac{1}{2}$		4.92	4.69
$\frac{5}{16}$	4.10	3.81	3.52	$\frac{5}{8}$		6.15	7.86
$\frac{3}{4}$	4.92	4.57	4.22	$\frac{3}{4}$		7.39	7.04
				$\frac{7}{8}$		9.85	9.38

## 13. RIVET HOLES IN TENSION MEMBERS

Area in sq. inches to be deducted for each rivet hole

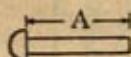
Dia. of Rivet in in.	Thickness of plate in inches							
	$\frac{1}{8}$	$\frac{3}{16}$	$\frac{1}{2}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{7}{8}$
$\frac{1}{8}$	.070	.100	.118					
$\frac{3}{16}$	.109	.137	.164					
$\frac{1}{2}$	.141	.176	.211	.246	.281			
$\frac{3}{4}$	.172	.215	.258	.301	.344			
$\frac{7}{8}$	.203	.254	.305	.355	.406	.508		
1	.234	.293	.352	.410	.469	.586	.703	
	.265	.332	.398	.465	.531	.664	.797	.930

Area to be deducted from any bar for one hole  $\frac{1}{16}$ " larger in dia. than the bolt or rivet.



### 14. WEIGHT OF RIVETS

Weight in Lbs. of 100 Steel Cup Headed Rivets  
included Head (approx.)



Diameter of Rivet in Inches.

Length A in in.	$\frac{1}{8}$	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	1	$1\frac{1}{2}$	$1\frac{3}{4}$
1	4.58	9.0						
$1\frac{1}{8}$	4.98	9.7						
$1\frac{1}{4}$	5.37	10.4						
$1\frac{3}{8}$	5.76	11.1						
$1\frac{1}{2}$	6.15	11.8	19.7	30.4				
$1\frac{3}{4}$	6.54	12.4	20.8	31.9				
$1\frac{7}{8}$	6.93	13.1	21.9	33.5				
$1\frac{5}{8}$	7.33	13.8	23.0	35.1				
2	7.72	14.5	24.1	36.6	52.5	72.0		
3	10.85	20.1	32.8	49.2	69.5	94.3	124	158
Approx. weight in lbs. of 100 Heads	1.45	3.4	6.7	11.6	18.4	27.5	39.2	53.8
Approx. weight in lbs. of 100 Shanks per inch of length	3.13	5.56	8.69	12.5	17.0	22.3	28.2	34.8

### 15. WEIGHTS OF GALVANIZED ROOF FITTINGS

Cup Headed Rivets	$\left\{ \begin{array}{l} \frac{1}{8}'' \times \frac{1}{4}'' \\ \frac{1}{4}'' \times \frac{1}{2}'' \\ \frac{3}{8}'' \times \frac{3}{4}'' \end{array} \right.$	$\left\{ \begin{array}{l} 2.0 \\ 2.1 \\ 2.3 \end{array} \right.$	Roofing Screws	$\left\{ \begin{array}{l} 2\frac{1}{2}'' \\ 2\frac{3}{4}'' \\ 3'' \end{array} \right.$	$\left\{ \begin{array}{l} 4.7 \\ 5.3 \\ 7.0 \end{array} \right.$
Bolts and Nuts	$\left\{ \begin{array}{l} 1\frac{1}{8}'' \times \frac{1}{2}'' \\ 1\frac{1}{4}'' \times \frac{3}{4}'' \\ 1'' \times \frac{1}{2}'' \\ 1\frac{1}{2}'' \times \frac{3}{4}'' \\ \frac{3}{4}'' \times \frac{1}{2}'' \end{array} \right.$	$\left\{ \begin{array}{l} 5.6 \\ 5.1 \\ 4.7 \\ 4.2 \\ 4.0 \end{array} \right.$	Hook Bolts	$\left\{ \begin{array}{l} 4'' \times \frac{3}{16}'' \\ 4'' \times \frac{1}{2}'' \\ 4\frac{1}{2}'' \times \frac{3}{16}'' \\ 4\frac{1}{2}'' \times \frac{1}{2}'' \\ 5'' \times \frac{3}{16}'' \\ 5'' \times \frac{1}{2}'' \end{array} \right.$	$\left\{ \begin{array}{l} 20.4 \\ 28.0 \\ 22.4 \\ 32.0 \\ 24.9 \\ 37.3 \end{array} \right.$
Roofing Pins or Nails	$\left\{ \begin{array}{l} 2\frac{1}{2}'' \\ \frac{1}{2}'' \end{array} \right.$	$\left\{ \begin{array}{l} 5.1 \\ 6.0 \end{array} \right.$	Washers Round and Flat for 1" Rivets and Nails		2.0
			Washers Curved (Limpet)		6.2



## 17. WHITWORTH STANDARD BOLTS &amp; NUTS

## Safe Loads in Tension Members

Tensile Strengths at 7.5 tons/sq. in. for steel and 5 tons/sq. in. for wrought iron

Diameter in inches	$\frac{1}{8}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1	$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{1}{2}$
Area in sq. inches of full bars	.049	.007	.110	.150	.196	.307	.442	.601	.785	.994	1.227	1.485
Area in sq. inches minus threads	.027	.046	.068	.094	.121	.204	.304	.422	.554	.697	.894	1.098
Weight in lbs. { Steel per r. ft. of bar { W.I.	.168 .164	.261 .256	.376 .369	.511 .502	.668 .656	1.04 1.025	1.50 1.48	2.04 2.01	2.67 2.62	3.38 3.32	4.17 4.09	5.05 4.96
Safe loads on Full { Steel or bars { W.I.	.37 .25	.58 .38	.83 .55	1.13 .75	1.47 .99	2.30 1.54	3.31 2.21	4.51 3.01	5.90 3.92	7.40 4.96	9.20 6.16	11.1 7.34
tensile strength Minus { Steel in tons { threads { W.I.	.20 .14	.35 .23	.51 .34	.71 .47	.91 .61	1.53 1.02	2.28 1.52	3.17 2.11	4.16 2.77	5.23 3.49	6.71 4.55	7.94 5.15

The safe load is arrived at by multiplying the area without threads by the safe load allowed per sq. in.



## 18. WORKING STRENGTHS OF STEEL RIVETS &amp; BOLTS

Shearing value @ 5 tons/sq. in.;      Bearing value @ 10 tons/sq. in.

Dia. of Bolt or Rivet in in.	Area in sq. in.	Strength in Single Shear	Strength in Double Shear	Bearing Strength										
				Thickness in inches of plate passed through										
				1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	7/8	
1/2	0.0491	0.25	0.49	.63	.78	.94	1.64	1.87	2.11	2.34	2.59	2.81		
3/8	0.1104	0.55	1.10	0.94	1.17	1.41	2.19	2.50	2.81	3.12	3.43	3.75		
1/4	0.1963	0.98	1.96	1.25	1.56	1.88	2.73	3.13	3.52	3.90	4.30	4.68		
3/16	0.3068	1.53	3.07	1.56	1.95	2.34	3.28	3.75	4.22	4.69	5.16	5.63		
1/8	0.4418	2.21	4.42	1.88	2.34	2.81	3.83	4.38	4.92	5.47	6.02	6.56		
3/32	0.6013	3.01	6.01	2.19	2.73	3.28	4.38	5.00	5.63	6.25	6.88	7.50		8.75
1	0.7854	3.93	7.85	2.50	3.13	3.75	4.38	5.00	5.63	6.25	6.88	7.50		8.75

## 19. WIRE GAUGES—COMPARISON OF

Size on Wire Gauge or No.	Diameter in inches			Size on Wire Gauge or No.	Diameter in inches		
	Imperial Standard Wire Gauge (S.W.G.)	Birmingham Wire Gauge (B.W.G.)	American Brown and Sharp (B and S)		Imperial Standard Wire Gauge (S.W.G.)	Birmingham Wire Gauge (B.W.G.)	American Brown and Sharp (B and S)
7/0	.500	..		23	.024	.025	.0226
6/0	.464	—	.5800	24	.022	.022	.0201
5/0	.432	.500	.5165	25	.020	.020	.0179
4/0	.400	.454	.4600	26	.018	.018	.0159
3/0	.372	.425	.4096	27	.0164	.016	.0142
2/0	.348	.380	.3648	28	.0148	.014	.0126
0	.324	.340	.3249	29	.0136	.013	.0113
1	.300	.300	.2893	30	.0124	.012	.0100
2	.276	.284	.2576	31	.0116	.010	.0089
3	.252	.259	.2294	32	.0108	.009	.0080
4	.232	.238	.2043	33	.0100	.008	.0071
5	.212	.220	.1819	34	.0092	.007	.0063
6	.192	.203	.1620	35	.0084	.005	.0055
7	.176	.180	.1443	36	.0076	.004	.0050
8	.160	.165	.1285	37	.0068	—	.0045
9	.144	.148	.1144	38	.0060		.0040
10	.128	.134	.1019	39	.0052		.0035
11	.116	.120	.0907	40	.0048		—
12	.104	.109	.0808	41	.0044		
13	.092	.095	.0720	4	.0040		
14	.080	.083	.0641	43	.0036		
15	.072	.072	.0571	44	.0032		
16	.064	.065	.0508	45	.0028		
17	.056	.058	.0453	46	.0024		
18	.048	.049	.0403	47	.0020		
19	.040	.042	.0359	48	.0016		
20	.36	.035	.0320	49	.0012		
21	.032	.032	.0285	50	.0010		
22	.028	.028	.0253				

The B.W.G. differs from 'B.G.' (Birmingham Gauge) which is generally used for sheets and hoop iron.

S.S.W.G. is Stubb's Steel Wire Gauge and is about the same as B.W.G.

W.S.W.G. is Whitworth Standard Wire Gauge and is issued mainly for threads of bolts, nuts and screws.

Zinc gauge is different and is used for zinc sheets.

In India gauges generally used are S.W.G. for wires and B.G. for sheets.

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

20. (a) FENCING WIRES  
Weight of Galvanized Fencing Strand

Gauge of full wire S.W.G.	Three Ply		Four Ply		Five Ply		Seven Ply	
	Size of single wire S.W.G.	Yds. per cwt.	Size of single wire S.W.G.	Yds. per cwt.	Size of single wire S.W.G.	Yds. per cwt.	Size of single wire S.W.G.	Yds. per cwt.
0	8	179	9½	185	10½	184	11½	162
1	8½	198	10½	231	11½	226	12½	205
2	9½	246	11	255	12	253	13	232
3	10	280	12	318	13	324	13½	266
4	11	340	12½	356	13½	372	14½	323
5	12	423	13½	434	14	428	15	379
6	12½	476	14	537	15	530	16	481
7	13½	580	14½	593	16	670	16½	560
8	14	717	15½	743	16½	788	17½	675
9	15	884	16½	896	17	876	18	853
10	16	1120	17	1095	18	1194	18½	1011
11	..	..	..	..	..	..	19	1230
12	..	..	..	..	..	..	20	1520

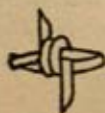
Fencing wires are hard and cannot be used as ropes. These wires are sometimes denoted as 3/15, 4/12 etc., which means 3 plies of 15 gauge each and 4 plies of 12 gauge each respectively.

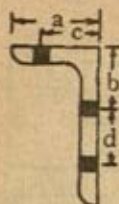


## 20. (b) FENCING WIRES

Galvanized Steel Barb Wire		No. 8 S.W.G. Galvanized Wire Stay Strand			
Approx. Weight of 100 yds. in lbs.		Yds. per cwt.			
	lbs.	3 Ply	4 Ply	5 Ply	8 Ply
12 S.W.G. Line Wire					
2 Point Ordinary					
Barbs round one wire only,		179	134	107	77
5 ins. apart.	19				
—Do— 6 ins. apart	18½				
2 Point Thickest					
Barbs round one wire only,					
2½ ins. apart.	21				
4 Point Ordinary					
Barbs round one wire only,					
6 ins. apart.	20				
4 point Thickest					
Barbs round one wire only,					
3 ins. apart.	25				
4 Point ordinary					
Barbs round both wires,					
6 ins. apart.	20				
4 Point Thickest					
Barbs round both wires,					
3 ins. apart	25				
14 S.W.G. Line Wire					
4 Point, Barb round one					
wire only					
3 ins. apart	16				
6 ins. apart	13				
		Galvanized Wire 7 Ply For Guys, Stays, Signals and Fencing			
		Dia. in.	Size of each wire S.W.G.	Length per cwt., yds.	Approx. Wt. per mile, lbs.
		1/2	7½	69	2840
		7/16	9	95	2085
		3/8	10	120	1650
		5/16	12	182	1085
		9/32	13	232	850
		1/4	13½	266	741
		7/32	15	379	520
		3/16	16	481	410
		1/8	18½	1011	195

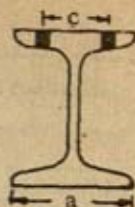
The barbed wire is formed by twisting together two galvanized wires called "line wires" which are generally of 12 or 14 S.W.G. The barbs or pricks are either of 2 points or 4 points which are twined round either one or both the line wires 2½ ins. to 6 ins. apart. 4 point barbs are more common.





# 21. SPACING OF HOLES & SIZE OF RIVETS IN ROLLED SECTIONS

(Dimensions in inches)



Leg a	c	Rivet
1 1/2	3/4	3/8
1 3/4	7/8	3/8
1 7/8	1	3/8
2	1 1/8	3/8
2 1/4	1 1/4	3/8
2 1/2	1 1/2	3/8
2 3/4	1 3/4	3/8
3	1 7/8	7/8
3 1/4	2	7/8
4	2 1/4	7/8
4 1/2	2 1/2	7/8
5	3	7/8
5 1/2	3 1/4	7/8
6	3 1/2	7/8
6 1/2	3 3/4	7/8
7	4	7/8
8	4 1/2	7/8
9	5	7/8
10	5 1/2	7/8
12	..	7/8

Leg a	c	Rivet
1 1/2	3/4	3/8
1 3/4	7/8	3/8
2 1/2	1 1/8	3/8
3	1 1/2	3/8
3 1/4	2	3/8
4	2 1/2	3/8
4 1/2	2 3/4	3/8
5	2 7/8	3/8
5 1/2	3 1/4	3/8
6	3 1/2	3/8
6 1/2	3 3/4	3/8
7	4	7/8
7 1/2	4 1/2	7/8
8	4 3/4	7/8

Sizes given for Ls  
apply to Channels and  
sizes for R.S. Js. apply  
to Ts.

Where 3 holes are  
made in Angle legs :

Taking distances as  
b, d, c from the edge,  
the dimensions are:—

Leg	b	d	c
8	2 1/2	2	2
9	3	2	2
10	3	2 1/2	2 1/2
12	4	3	3

## 22. (a) Approximate Number of Galvanized Plain and Corrugated Sheets per Ton

Thickness	Corrugation	Length in feet							Lengths generally manufactured in India for plain sheets
		6'	7'	8'	9'	10'	11'	12'	
16 B.G.	8/3	58	50	44	39	35			6' to 10'
	10/3	49	42	37	33	29			6' to 10'
18 B.G.	8/3	74	64	56	49	44			6'
	10/3	62	53	46	41	37			6' to 10'
20 B.G.	8/3	95	81	71	63	57			6" al o 6' x 2'
	10/3	79	68	59	53	47			6' to 10'
22 B.G.	8/3	116	99	87	77	69			6' also 6' x 2'
	10/3	97	83	73	65	58	76	70	6' to 12'
24 B.G.	8/3	140	120	105	93	84	64	58	6' also 6' x 2'
	10/3	117	100	88	78	70			6' to 12'
26 B.G.	8/3	186	159	139	124	111	101	93	6', 7', 8'
	10/3	155	133	116	103	93	84	77	also 6' x 2'
27 B.G.	8/3	190	163	143	127	114			6', 7', 8'
	10/3	158	136	119	105	95			6', 9', 10'
28 B.G.	8/3	200	172	150	133	120			6', 9', 10'
	10/3	167	143	125	111	100			6', 9', 10'
29 B.G.	8/3	231	212	184	154	130			6', 9', 10'
	10/3	193	188	164	129	116			6', 9', 10'
30 B.G.	8/3	240	206	180	160	144			6', 9', 10'
	10/3	213	183	160	142	128			

8/3 sheets are 26" wide (edge to edge)—corrugated.

10/3 sheets are 32" wide (edge to edge)—corrugated.

Flat sheets 30" wide count the same as 8/3 corrugations, and 36" wide the same as 10/3 corrugations.



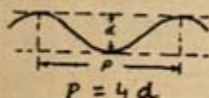
## 22 (b). CORRUGATED GALVANIZED IRON SHEETS

The following table gives the Spans the Sheets will carry for a load of 22 lbs./sq. ft.

Gauge (B.G.*)	16	18	20	22	24	26	Cantilevers will be only $\frac{1}{4}$ of these lengths.
Length in ft. (without laps)	7.4	6.8	5.9	5.2	4.6	4.1	
With end lap of 9" and riveted.	9.0	7.8	6.5	6.0	5.3	4.7	

## Weight of C.G.I. Sheets (laid) in lbs. per 100 sq. ft.

Gauge (B.G.*)	18	20	22	24	26
Without laps .. .. .	241	185	155	129	95
With 6" end lap, 1 corrugation side lap	273	209	175	146	108
With 6" end lap, 2 corrugations side lap .. .. .	303	233	195	162	119



$$P = 4d$$

For 3" corrugations  $P=3''$  and  $d=\frac{3}{4}''$

For 5" corrugations  $P=5''$  and  $d=1\frac{1}{4}''$

[B.G. is sheet gauge—for sizes see Tables at pages 4/25, 4/32.]

8/3 and 10/3 are 3" corrugation sheets. (C.G.I. sheets usually manufactured in India are 3" corrugations). 8/3 means 8 corrugations of 3" in the full width of a sheet and 10/3 means 10 corrugation of 3" in the full width.

The number of corrugations are 8, 9 or 10 per sheet, 8 and 10 are most common. 30 ins. wide flat sheet is given 8 corrugations, 33 ins. wide 9 corrugations and 36 ins. wide 10 corrugations of 3 ins. pitch, which reduce the width of the sheets to 26 ins., 29 ins., and 32 ins. respectively.

The weights given at page 4/29 do not apply to Galvanized sheets tested to specification based on IS: 277/51, Class 1, Class 2 or Class 3.

Mild steel black sheets should be galvanized with a coating of spelter (alloy of zinc), which are specified into

three classes according to the thickness of the zinc coating.\*

Class 1. Extra heavy coating of zinc, average 2.50 oz./sq. ft.

Class 2. Heavy coating of zinc, average 2.00 oz./sq. ft.

Class 3. Medium coating of zinc, average 1.50 oz./sq. ft.

The weights represent the total weight of zinc, both sides inclusive. On any random sample selected, the coating shall be not less than 80 per cent of the average specified for the class to which the sample belongs. The following table gives the exact weight of galvanized sheets (plan flat surface) based on the thickness of zinc coating. B.G. is thickness of ungalvanized sheets.

**Nominal Weight of Galvanized Steel Sheets**  
Lbs. per sq. ft. of plain flat surface

B.G.	16	18	20	22	24	26	28	30
Class 1—Coating of zinc 2.50 oz. per sq. ft.								
	2.643	2.126	1.716	1.400	1.144	0.937	0.777	0.650
Class 2—Coating of zinc 2.00 oz. per sq. ft.								
	2.611	2.094	1.684	1.368	1.112	0.905	0.745	0.618
Class 3—Coating of zinc 1.50 oz. per sq. ft.								
	2.580	2.063	1.653	1.337	1.081	0.874	0.714	0.587

Black sheets are rolled from low carbon mild steel. The sheets should be free from cracks, pittings, blisters, laminations, twists, scales and other surface defects. Black sheets required for galvanizing are annealed; sheets are corrugated before galvanizing.

The above is based on the recommendations in IS: 277 of 1951.

Testing of sheets and methods of galvanizing are given in Section 5.

## 23. MILD STEEL BLACK SHEETS

Gauge B.G.	Equivalent thickness		Weight of sheet/sq. ft. lbs.	Widths and lengths manufactured
	Inch			
15/0	1"	1.000	Not generally manufactured in India	24"—30"—10 ft. lengths all sizes, 31"—36"—12 ft. lengths all sizes, 37"—42"—12 ft. lengths up to 18 B.G. and 10 ft. 19 to 22 B.G. 43"—48"—12 ft. lengths up to 16 B. G. and 10 ft. 17, 18 B. G. 49" to 56" size are not generally available.
10/0		.792		
3/0	$\frac{1}{2}$ "	.500		
2/0		.445		
1/0		.396		
1		.353		
2		.315		
3	$\frac{1}{4}$ "	.280		
4		.250		
5		.222		
6		.198	6.41	
7		.176		
8		.157		
9		.140		
10	$\frac{1}{8}$ "	.1250		
11		.1113		
12		.0991		
13		.0882		
14		.0785		
15		.0669		
16	$1/16$ "	.0625	5.70	
17		.0556		
18		.0495		
19		.0440		
20		.0392		
21		.0349		
22	$1/32$ "	.0312		
23		.0278		
24		.0247		
25		.0220		
26		.0196	1.42	When sheets are galvanized coating of zinc is added at the rate of 2.50 oz, 2.00 oz., or 1.50 oz. per sq. ft., both sides inclusive.
27		.0174		
28	$1/64$ "	.0156		
29		.0139		
30		.0123		
31		.011		
32		.010		

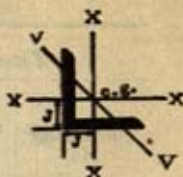


## 24. WEIGHT OF SHEETS OF DIFFERENT METAL

Zinc Sheets			Copper and Brass Sheets		
Zinc Gauge No.	Thickness in.	Weight lb./sq.ft.	Gauge S.W.G.	Weight lbs./sq. ft.	
				copper	Brass
1	0.004	0.149	1	13.68	13.11
2	0.006	0.225	2	12.59	12.06
3	0.007	0.262	3	11.49	11.01
4	0.008	0.300	4	10.60	10.14
5	0.010	0.375	5	9.67	9.26
6	0.011	0.412	6	8.76	8.39
7	0.013	0.487	7	8.03	7.69
8	0.015	0.562	8	7.30	6.99
9	0.017	0.637	9	6.57	6.29
10	0.019	0.712	10	5.84	5.59
11	0.022	0.824	11	5.29	5.07
12	0.025	0.936	12	4.74	4.54
13	0.028	1.049	13	4.20	4.02
14	0.031	1.161	14	3.65	3.50
15	0.036	1.348	15	3.28	3.15
16	0.041	1.536	16	2.92	2.80
17	0.046	1.723	17	2.55	2.45
18	0.051	1.910	18	2.19	2.10
19	0.057	2.135	19	1.82	1.75
20	0.063	2.360	20	1.64	1.57
21	0.070	2.622	21	1.46	1.40
S.W.G.	..	..	22	1.28	1.22
14	0.080	2.996	23	1.09	1.05
13	0.092	3.446	24	1.00	0.95
12	0.104	3.895	25	0.91	0.87
11	0.116	4.345	26	0.82	0.79
10	0.128	4.794	27	0.748	0.717
9	0.144	5.394	28	0.675	0.647
8	0.160	5.993	29	0.620	0.594
7	0.176	6.592	30	0.565	0.542
6	0.192	7.192			
5	0.212	7.941			
4	0.232	8.690			
3	0.252	9.439			
2	0.276	10.338			
1	0.300	11.237			

## 25. EQUAL ANGLES

## Dimensions &amp; Properties



Size ins.	Wt. per ft. lbs.	Area sq. ins	Dimension J	Moment of Inertia	Min. Radii of Gyration	Min : Section Modulus
				about X—X	about V—V	about X—X
$1 \times 1 \times \frac{1}{8}$	0.58	0.172	0.22	0.008	0.139	0.015
$1 \times 1 \times \frac{1}{4}$	1.12	0.313	0.27	0.013	0.145	0.028
$1 \times 1 \times \frac{3}{16}$	0.80	0.234	0.29	0.020	0.189	0.028
$1 \times 1 \times \frac{1}{2}$	1.16	0.34	0.31	0.03	0.19	0.04
$1 \times 1 \times \frac{3}{4}$	1.49	0.437	0.33	0.035	0.190	0.053
$1 \frac{1}{2} \times 1 \frac{1}{2} \times \frac{1}{8}$	1.01	0.30	0.34	0.04	0.24	0.05
$1 \frac{1}{2} \times 1 \frac{1}{2} \times \frac{3}{16}$	1.47	0.43	0.37	0.06	0.24	0.07
$1 \frac{1}{2} \times 1 \frac{1}{2} \times \frac{1}{4}$	1.91	0.56	0.40	0.07	0.24	0.09
$1 \frac{1}{2} \times 1 \frac{1}{2} \times \frac{3}{8}$	1.23	0.37	0.40	0.07	0.29	0.07
$1 \frac{1}{2} \times 1 \frac{1}{2} \times \frac{1}{2}$	1.79	0.53	0.43	0.10	0.29	0.10
$1 \frac{1}{2} \times 1 \frac{1}{2} \times \frac{3}{4}$	2.34	0.69	0.46	0.13	0.29	0.13
$1 \frac{1}{2} \times 1 \frac{1}{2} \times \frac{1}{2}$	2.85	0.84	0.48	0.16	0.29	0.16
$1 \frac{1}{2} \times 1 \frac{1}{2} \times \frac{3}{4}$	2.11	0.62	0.49	0.17	0.34	0.14
$1 \frac{1}{2} \times 1 \frac{1}{2} \times \frac{1}{2}$	2.76	0.81	0.52	0.22	0.34	0.18
$1 \frac{1}{2} \times 1 \frac{1}{2} \times \frac{3}{8}$	3.39	1.00	0.54	0.26	0.34	0.22
$2 \times 2 \times \frac{3}{16}$	2.43	0.71	0.56	0.26	0.39	0.18
$2 \times 2 \times \frac{1}{4}$	3.19	0.94	0.58	0.34	0.39	0.24
$2 \times 2 \times \frac{3}{8}$	3.92	1.15	0.61	0.40	0.38	0.29
$2 \times 2 \times \frac{1}{2}$	4.62	1.36	0.63	0.47	0.38	0.34
$2 \frac{1}{2} \times 2 \frac{1}{2} \times \frac{3}{16}$	2.75	0.81	0.62	0.38	0.44	0.23
$2 \frac{1}{2} \times 2 \frac{1}{2} \times \frac{1}{4}$	3.61	1.06	0.64	0.49	0.44	0.30
$2 \frac{1}{2} \times 2 \frac{1}{2} \times \frac{3}{8}$	4.45	1.31	0.67	0.59	0.43	0.37
$2 \frac{1}{2} \times 2 \frac{1}{2} \times \frac{1}{2}$	5.26	1.55	0.69	0.69	0.43	0.44
$2 \frac{1}{2} \times 2 \frac{1}{2} \times \frac{3}{4}$	4.04	1.19	0.70	0.68	0.49	0.38
$2 \frac{1}{2} \times 2 \frac{1}{2} \times \frac{1}{2}$	4.98	1.46	0.73	0.83	0.48	0.47
$2 \frac{1}{2} \times 2 \frac{1}{2} \times \frac{3}{8}$	5.90	1.73	0.75	0.96	0.48	0.55
$2 \frac{1}{2} \times 2 \frac{1}{2} \times \frac{1}{4}$	6.79	2.00	0.78	1.09	0.48	0.63
$2 \frac{1}{2} \times 2 \frac{1}{2} \times \frac{3}{16}$	7.65	2.25	0.80	1.21	0.48	0.71
$2 \frac{1}{2} \times 2 \frac{1}{2} \times \frac{1}{2}$	4.64	1.31	0.76	0.92	0.54	0.46
$2 \frac{1}{2} \times 2 \frac{1}{2} \times \frac{3}{8}$	6.54	1.92	0.81	1.30	0.53	0.67
$2 \frac{1}{2} \times 2 \frac{1}{2} \times \frac{1}{4}$	8.50	2.50	0.86	1.64	0.53	0.87
$3 \times 3 \times \frac{1}{2}$	4.89	1.44	0.83	1.20	0.59	0.55

## EQUAL ANGLES (Concluded)

Size ins.	Wt. per ft. lbs.	Area sq. ins.	Dimension J	Moment of Inertia	Min : Radii of Gyration	Min : Section Modulus
				about X—X	about V—V	about X—X
$3 \times 3 \times \frac{5}{16}$	6.04	1.78	0.85	1.47	0.58	0.68
$3 \times 3 \times \frac{3}{8}$	7.17	2.11	0.88	1.72	0.58	0.81
$3 \times 3 \times \frac{7}{16}$	8.28	2.43	0.90	1.96	0.58	0.93
$3 \times 3 \times \frac{1}{2}$	9.35	2.75	0.92	2.18	0.58	1.05
$3\frac{1}{2} \times 3\frac{1}{2} \times \frac{5}{16}$	7.11	2.09	0.97	2.38	0.68	0.94
$3\frac{1}{2} \times 3\frac{1}{2} \times \frac{3}{8}$	8.45	2.49	1.00	2.80	0.68	1.12
$3\frac{1}{2} \times 3\frac{1}{2} \times \frac{7}{16}$	11.05	3.25	1.05	3.57	0.68	1.46
$3\frac{1}{2} \times 3\frac{1}{2} \times \frac{1}{2}$	13.55	3.99	1.09	4.27	0.68	1.77
$4 \times 4 \times \frac{5}{16}$	8.17	2.40	1.10	3.61	0.78	1.24
$4 \times 4 \times \frac{3}{8}$	9.73	2.86	1.12	4.26	0.78	1.48
$4 \times 4 \times \frac{7}{16}$	11.25	3.31	1.15	4.87	0.78	1.71
$4 \times 4 \times \frac{1}{2}$	12.75	3.75	1.17	5.46	0.78	1.93
$4 \times 4 \times \frac{5}{8}$	14.23	4.18	1.20	6.02	0.77	2.15
$4 \times 4 \times \frac{3}{4}$	15.68	4.61	1.22	6.56	0.77	2.36
$5 \times 5 \times \frac{5}{16}$	12.28	3.61	1.37	8.53	0.98	2.35
$5 \times 5 \times \frac{7}{16}$	14.23	4.19	1.39	9.81	0.98	2.72
$5 \times 5 \times \frac{1}{2}$	16.16	4.75	1.42	11.04	0.90	3.08
$5 \times 5 \times \frac{3}{4}$	18.06	5.31	1.44	12.22	0.97	3.44
$5 \times 5 \times \frac{1}{2}$	19.93	5.86	1.47	13.37	0.97	3.78
$*6 \times 6 \times \frac{5}{16}$	14.82	4.36	1.61	14.95	1.18	3.40
$*6 \times 6 \times \frac{7}{16}$	17.20	5.06	1.64	17.25	1.18	3.95
$*6 \times 6 \times \frac{1}{2}$	19.55	5.75	1.66	19.48	1.18	4.49
$*6 \times 6 \times \frac{3}{4}$	21.87	6.43	1.69	21.64	1.17	5.02
$*6 \times 6 \times \frac{1}{2}$	24.17	7.11	1.71	23.73	1.17	5.54
$*6 \times 6 \times \frac{11}{16}$	26.44	7.78	1.74	25.77	1.17	6.04
$6 \times 6 \times \frac{3}{4}$	28.69	8.44	1.76	27.74	1.17	6.54
$6 \times 6 \times \frac{7}{8}$	33.10	9.73	1.81	31.51	1.16	7.51
$8 \times 8 \times \frac{1}{2}$	28.90	8.50	2.17	51.83	1.58	8.89
$*8 \times 8 \times \frac{3}{4}$	32.68	9.61	2.20	58.26	1.57	10.05
$8 \times 8 \times \frac{1}{2}$	38.89	11.44	2.25	68.58	1.57	11.94
$*8 \times 8 \times \frac{11}{16}$	1.96	12.34	2.28	73.57	1.56	12.86
$*8 \times 8 \times \frac{3}{4}$	45.02	13.24	2.30	78.44	1.56	13.77
$*8 \times 8 \times \frac{13}{16}$	48.02	14.12	2.33	83.20	1.56	14.66
$*8 \times 8 \times 1$	51.01	15.00	2.35	87.85	1.56	15.55

\*Marked sections are not generally rolled in India.





## 26. UNEQUAL ANGLES

## Dimensions &amp; Properties

Size ins.	Wt. per ft. lbs.	Area sq. ins.	Dimensions		Moment of Inertia		Min. Radii of Gyration	Min. Section Moduli	
			J	P	X-X	Y-Y		X-X	Y-Y
$1\frac{1}{2} \times 1\frac{1}{2} \times \frac{3}{16}$	1.80	0.53	0.57	0.32	0.15	0.06	0.26	0.13	0.07
$1\frac{1}{2} \times 1\frac{1}{2} \times \frac{1}{4}$	2.34	0.69	0.59	0.35	0.20	0.08	0.26	0.17	0.09
$2 \times 1\frac{1}{2} \times \frac{1}{4}$	2.76	0.81	0.65	0.41	0.31	0.15	0.31	0.23	0.13
$2\frac{1}{2} \times 1\frac{1}{2} \times \frac{1}{4}$	3.19	0.94	0.86	0.37	0.58	0.15	0.32	0.35	0.14
$2\frac{1}{2} \times 1\frac{1}{2} \times \frac{5}{16}$	3.92	1.15	0.89	0.39	0.70	0.18	0.32	0.43	0.17
$2\frac{1}{2} \times 2 \times \frac{1}{4}$	3.61	1.06	0.77	0.53	0.63	0.36	0.42	0.37	0.24
$2\frac{1}{2} \times 2 \times \frac{5}{16}$	4.45	1.31	0.80	0.55	0.77	0.43	0.42	0.45	0.30
$2\frac{1}{2} \times 2 \times \frac{3}{8}$	5.26	1.55	0.82	0.57	0.80	0.50	0.41	0.53	0.35
$3 \times 2 \times \frac{1}{4}$	4.04	1.19	0.98	0.48	1.06	0.38	0.43	0.52	0.25
$3 \times 2 \times \frac{5}{16}$	4.98	1.46	1.00	0.51	1.29	0.45	0.43	0.65	0.30
$3 \times 2 \times \frac{3}{8}$	5.90	1.73	1.03	0.53	1.50	0.53	0.42	0.76	0.36
$3 \times 2\frac{1}{2} \times \frac{1}{4}$	4.47	1.31	0.89	0.65	1.14	0.72	0.52	0.54	0.39
$3 \times 2\frac{1}{2} \times \frac{5}{16}$	5.51	1.62	0.92	0.67	1.39	0.87	0.52	0.67	0.48
$3 \times 2\frac{1}{2} \times \frac{3}{8}$	6.54	1.92	0.94	0.70	1.62	0.92	1.02	0.79	0.56
$3 \times 2\frac{1}{2} \times \frac{7}{16}$	7.53	2.22	0.97	0.72	1.84	1.15	0.52	0.91	0.65
$3 \times 2\frac{1}{2} \times \frac{1}{2}$	8.50	2.50	0.99	0.74	2.05	1.28	0.52	1.02	0.73

$3 \times 3 \times \frac{1}{16}$	4.89	1.44	1.09	0.60	1.75	0.74	0.54	0.73	0.39
$3 \times 3 \times \frac{1}{8}$	6.04	1.78	1.12	0.63	2.14	0.91	0.53	0.90	0.48
$3 \times 3 \times \frac{1}{4}$	7.17	2.11	1.15	0.65	2.15	1.06	0.53	1.07	0.57
$3 \times 3 \times \frac{1}{2}$	9.35	2.75	1.19	0.70	3.19	1.33	0.53	1.38	0.74
$3 \times 3 \times \frac{3}{4}$	5.32	1.56	1.01	0.77	1.86	1.26	0.62	0.75	0.56
$3 \times 3 \times \frac{1}{16}$	6.58	1.93	1.04	0.89	2.27	1.54	0.62	0.92	0.70
$3 \times 3 \times \frac{1}{8}$	7.81	2.30	1.07	0.82	2.67	1.80	0.62	1.10	0.83
$4 \times 3 \times \frac{1}{16}$	7.11	2.09	1.24	0.75	3.30	1.59	0.64	1.20	0.71
$4 \times 3 \times \frac{1}{8}$	8.45	2.49	1.27	0.77	3.89	1.87	0.64	1.42	0.84
$4 \times 3 \times \frac{1}{4}$	9.76	2.87	1.29	0.80	4.44	2.13	0.63	1.64	0.96
$4 \times 3 \times \frac{1}{2}$	11.05	3.25	1.32	0.82	4.97	2.37	0.63	1.85	1.09
$5 \times 3 \times \frac{1}{16}$	8.17	2.40	1.66	0.67	6.14	1.68	0.65	1.84	0.72
$5 \times 3 \times \frac{1}{8}$	9.73	2.86	1.68	0.69	7.25	1.97	0.65	2.18	0.85
$5 \times 3 \times \frac{1}{4}$	11.25	3.31	1.71	0.72	8.31	2.25	0.64	2.53	0.99
$5 \times 3 \times \frac{1}{2}$	12.75	3.75	1.73	0.74	9.33	2.51	0.64	2.86	1.11
$5 \times 3 \times \frac{3}{4}$	14.23	4.18	1.76	0.77	10.31	2.76	0.64	3.18	1.24
$5 \times 3 \times \frac{1}{16}$	10.37	3.05	1.59	0.85	7.63	3.09	0.75	2.24	1.16
$5 \times 3 \times \frac{1}{8}$	13.61	4.00	1.64	0.90	9.84	3.96	0.75	2.93	1.52
$5 \times 3 \times \frac{1}{4}$	9.24	2.72	2.09	0.61	10.13	1.74	0.64	2.59	0.73
$6 \times 3 \times \frac{1}{16}$	11.00	3.24	2.12	0.63	11.99	2.05	0.64	3.19	0.87
$6 \times 3 \times \frac{1}{8}$	12.74	3.75	2.15	0.66	13.78	2.34	0.64	3.58	1.00
$6 \times 3 \times \frac{1}{4}$	14.45	4.25	2.17	0.68	15.51	2.62	0.63	4.05	1.13
$6 \times 3 \times \frac{1}{2}$	16.14	4.75	2.20	0.71	17.18	2.88	0.63	4.52	1.26
$6 \times 3 \times \frac{3}{4}$	17.80	5.24	2.22	0.73	18.80	3.14	0.63	4.98	1.38
$6 \times 3 \times \frac{1}{16}$	11.63	3.42	2.01	0.77	12.62	3.21	0.76	3.16	1.18
$6 \times 3 \times \frac{1}{8}$	13.48	3.96	2.04	0.80	14.53	3.68	0.75	3.66	1.36
$6 \times 3 \times \frac{1}{4}$	15.30	4.50	2.06	0.82	16.36	4.13	0.75	4.15	1.54
$6 \times 3 \times \frac{1}{2}$	18.86	5.55	2.11	0.87	19.85	4.96	0.75	5.11	1.89
$6 \times 4 \times \frac{1}{16}$	12.28	3.61	1.91	0.92	13.21	4.74	0.87	3.23	1.54
$6 \times 4 \times \frac{1}{8}$	14.23	4.19	1.95	0.95	15.21	5.44	0.86	3.75	1.78
$6 \times 4 \times \frac{1}{4}$	16.16	4.75	1.97	0.97	17.14	6.11	0.86	4.25	2.02

## UNEQUAL ANGLES (Concl'd.)

Size ins.	Wt. per ft. lbs.	Area sq. ins.	Dimensions		Moment of Inertia		Min. Radii of Gyration	Min. Section Moduli	
			J	P	About X—X	Y—Y		X—X	about Y—Y
6×4× <sup>9</sup> / <sub>16</sub>	18.06	5.31	1.99	1.00	19.01	6.76	0.86	4.74	2.25
6×4× <sup>1</sup> / <sub>2</sub>	19.93	5.86	2.02	1.02	20.82	7.37	0.86	5.23	2.48
6×4× <sup>11</sup> / <sub>16</sub>	21.77	6.40	2.04	1.05	22.57	7.96	0.85	5.70	2.70
6×4× <sup>3</sup> / <sub>4</sub>	23.59	6.94	2.06	1.07	24.26	8.53	0.85	6.16	2.91

## 27. WEIGHT OF FLAT STEEL BARS IN LBS. PER LINEAL FOOT

Width ins.	Thickness in inches									
	$\frac{3}{16}$	$\frac{1}{8}$	$\frac{9}{16}$	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	1
1	.213	.425	.638	.850	1.06	1.28	1.49	1.70	1.91	2.34
1 $\frac{1}{4}$	.266	.531	.797	1.06	1.33	1.59	1.86	2.13	2.39	2.92
1 $\frac{1}{2}$	.319	.638	.956	1.28	1.59	1.91	2.23	2.55	2.87	3.51
1 $\frac{3}{4}$	.372	.744	1.12	1.49	1.86	2.23	2.60	2.98	3.35	4.09
2	.425	.850	1.28	1.70	2.13	2.55	2.98	3.40	3.83	4.68
										5.10
										5.95
										6.80

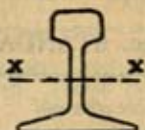
## 28. STEEL SHEETS WEIGHT PER SQ. FT.

Wt. lbs. per sq. ft.	2.55	5.10	7.65	10.20	12.70	15.30	17.80	20.40	22.95	25.50	28.08	30.60	35.70	40.80
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## 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 FF	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 DH	2.60	8.63	22.45	1.73	5.187
4.000	74 FF	2.52	8.16	20.56	1.67	4.750
4.500	70 FF	2.44	7.46	18.21	1.65	4.562
2.437	68 DH	2.50	7.86	19.66	1.71	5.000
2.125	68 BH	2.91	7.63	22.19	1.81	5.343
2.250	64 DH	2.50	7.32	18.30	1.73	5.000
4.000	62 FF	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 FF	2.13	6.57	13.99	1.54	4.250
3.875	58½ FF	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 FF	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 FF	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  $\frac{1}{2}$  to  $\frac{3}{4}$ .

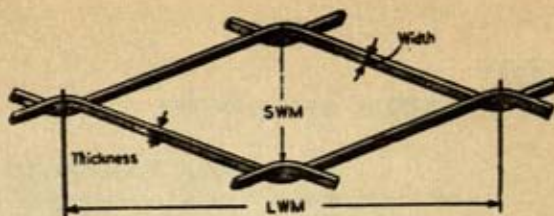
## 30. STANDARD SIZES OF EXPANDED METAL

Size of Mesh	Dimensions of Strands		Weight per sq. yd.	Cross sectional area per ft. SWM
SWM $\times$ LWM	Width	Thickness		
3 $\times$ 7 $\frac{1}{2}$	$\frac{1}{8}$	$\frac{1}{8}$	30.60	1.000
	$\frac{7}{16}$	$\frac{1}{8}$	26.78	0.875
	$\frac{1}{4}$	$\frac{3}{16}$	22.95	0.750
	$\frac{5}{16}$	$\frac{1}{4}$	19.25	0.625
	$\frac{3}{8}$	$\frac{3}{16}$	17.21	0.563
	$\frac{1}{2}$	$\frac{1}{2}$	15.30	0.500
	$\frac{9}{32}$	$\frac{3}{16}$	12.91	0.422
	$\frac{5}{16}$	$\frac{3}{16}$	11.48	0.375
	$\frac{3}{8}$	$\frac{1}{2}$	9.56	0.313
	$\frac{3}{16}$	$\frac{3}{16}$	8.61	0.281
	$\frac{1}{4}$	$\frac{1}{2}$	7.65	0.250
	$\frac{3}{16}$	$\frac{1}{2}$	5.74	0.188
$1\frac{1}{2} \times 4\frac{1}{2}$	$\frac{1}{8}$	$\frac{3}{16}$	34.43	1.125
	$\frac{5}{16}$	$\frac{3}{16}$	28.69	0.938
	$\frac{1}{4}$	$\frac{3}{16}$	22.95	0.750
	$\frac{3}{8}$	$\frac{1}{2}$	15.30	0.500
	$\frac{3}{16}$	$\frac{1}{2}$	11.48	0.375
	$\frac{1}{2}$	$\frac{1}{2}$	7.65	0.250
	$\frac{3}{16}$	16 BG	7.65	0.250
	$\frac{1}{4}$	16 BG	5.74	0.188
	$\frac{3}{32}$	16 BG	3.83	0.125
$1\frac{1}{2} \times 3$	$\frac{3}{32}$	18 BG	2.27	0.074
	$\frac{3}{16}$	$\frac{1}{2}$ in.	11.48	0.375
	$\frac{1}{4}$	13 BG	5.75	0.187
	$\frac{3}{8}$	$\frac{1}{2}$ in.	7.65	0.250
	$\frac{1}{2}$	16 BG	3.83	0.125
1 $\times$ 3	$\frac{3}{16}$	$\frac{1}{8}$	15.06	0.492
	$\frac{3}{16}$	11 BG	13.40	0.438
	$\frac{1}{4}$	$\frac{1}{8}$	10.04	0.328
	$\frac{1}{4}$	11 BG	8.94	0.292
	$\frac{1}{4}$	13 BG	7.53	0.246
	$\frac{3}{16}$	16 BG	7.53	0.246
	$\frac{1}{2}$	16 BG	5.02	0.164
	$\frac{1}{2}$	18 BG	4.00	0.130
	$\frac{3}{32}$	16 BG	3.76	0.123
	$\frac{3}{32}$	18 BG	2.97	0.097
	$\frac{3}{32}$	20 BG	2.36	0.077

Size of Mesh	Dimensions of Strands		Weight per sq. yd.	Cross sectional area per ft. SWM
SWM $\times$ LWM	Width	Thickness		
$\frac{3}{4} \times 2^{39/64}$	$\frac{1}{4}$	$\frac{3}{16}$	33.00	1.078
	$\frac{3}{16}$	$\frac{1}{4}$	19.25	0.629
	$\frac{1}{2}$	$\frac{1}{2}$	13.25	0.433
	$\frac{1}{2}$	13 BG	9.18	0.318
	$\frac{1}{2}$	16 BG	11.25	0.368
	$\frac{3}{16}$	16 BG	8.75	0.286
	$\frac{1}{2}$	16 BG	6.50	0.212
	$\frac{3}{32}$	16 BG	5.00	0.163
	$\frac{3}{32}$	18 BG	3.75	0.123
	$\frac{3}{32}$	20 BG	3.00	0.098
	$\frac{3}{32}$	24 BG	1.75	0.057
$\frac{1}{2} \times 2$	$\frac{1}{4}$	$\frac{1}{2}$ in.	13.00	0.450
	$\frac{1}{2}$	13 BG	9.18	0.318
	$\frac{1}{2}$	16 BG	6.45	0.230
	$\frac{3}{32}$	18 BG	4.25	0.138
$\frac{1}{2} \times 2$	$\frac{1}{4}$	16 BG	8.10	0.310
	$\frac{3}{32}$	16 BG	6.32	0.234
	$\frac{3}{32}$	18 BG	5.78	0.210
	$\frac{3}{32}$	20 BG	4.64	0.170
$\frac{1}{2} \times 1\frac{1}{2}$	$\frac{1}{4}$	16 BG	9.00	0.294
	$\frac{1}{2}$	18 BG	7.00	0.229
	$\frac{3}{32}$	16 BG	7.00	0.229
	$\frac{1}{2}$	20 BG	6.00	0.196
	$\frac{3}{32}$	18 BG	5.50	0.180
	$\frac{3}{32}$	20 BG	4.50	0.147
	$\frac{3}{32}$	22 BG	3.50	0.114
$\frac{1}{2} \times 1\frac{11}{16}$	$\frac{1}{4}$	16 BG	17.00	0.556
	$\frac{3}{16}$	16 BG	14.25	0.466
	$\frac{1}{2}$	18 BG	13.50	0.441
	$\frac{5}{32}$	16 BG	12.50	0.408
	$\frac{3}{16}$	18 BG	11.50	0.376
	$\frac{1}{2}$	16 BG	10.50	0.343
	$\frac{1}{2}$	20 BG	10.50	0.343
	$\frac{5}{32}$	18 BG	10.00	0.327
	$\frac{3}{16}$	20 BG	9.00	0.294
	$\frac{3}{32}$	16 BG	8.25	0.270
	$\frac{1}{2}$	18 BG	8.25	0.270
	$\frac{5}{32}$	20 BG	7.75	0.253
	$\frac{3}{32}$	18 BG	6.50	0.212
	$\frac{1}{2}$	20 BG	6.50	0.212
	$\frac{3}{32}$	20 BG	5.00	0.163
	$\frac{3}{32}$	22 BG	4.00	0.131



Size of Mesh	Dimensions of Strands		Weight per sq. yd.	Cross sectional area per ft. SWM
SWM × LWM	Width	Thickness		
$\frac{1}{2} \times 1\frac{1}{2}$	$\frac{1}{4}$	16 BG	22.50	0.735
	$\frac{1}{4}$	18 BG	18.20	0.588
	$\frac{3}{16}$	16 BG	18.00	0.588
	$\frac{5}{32}$	16 BG	15.25	0.498
	$\frac{3}{16}$	18 BG	14.25	0.466
	$\frac{1}{4}$	20 BG	14.00	0.458
	$\frac{1}{4}$	16 BG	13.50	0.441
	$\frac{5}{32}$	18 BG	12.50	0.400
	$\frac{3}{32}$	16 BG	11.25	0.368
	$\frac{3}{16}$	20 BG	11.00	0.359
	$\frac{1}{4}$	18 BG	10.75	0.351
	$\frac{5}{32}$	20 BG	9.50	0.310
	$\frac{3}{32}$	18 BG	9.00	0.294
	$\frac{1}{4}$	20 BG	8.50	0.278
	$\frac{3}{32}$	20 BG	7.00	0.229
	$\frac{3}{32}$	22 BG	5.50	0.180
$\frac{3}{16} \times \frac{27}{32}$	$\frac{3}{32}$	20 BG	8.75	0.286
	$\frac{3}{32}$	22 BG	7.00	0.229
	$\frac{3}{32}$	24 BG	5.50	0.180
$\frac{1}{2} \times \frac{7}{16}$	$\frac{1}{16}$	20 BG	8.33	0.272
	$\frac{3}{64}$	18 BG	8.25	0.270
	$\frac{1}{16}$	22 BG	6.67	0.218
	$\frac{3}{64}$	20 BG	6.25	0.204
	$\frac{1}{16}$	24 BG	5.25	0.172
	$\frac{3}{64}$	22 BG	5.00	0.163
	$\frac{3}{64}$	24 BG	4.00	0.131
	$\frac{1}{16}$	27 BG	3.75	0.123
	$\frac{3}{64}$	27 BG	2.75	0.090
	$\frac{1}{32}$	24 BG	2.67	0.087
	$\frac{1}{32}$	27 BG	1.80	0.059



SWM is shortway of mesh, LWM is longway of mesh. When ordering expanded metal it is necessary to specify which dimension refers to LWM or SWM. Size of mesh is the short length of diamond. If only one mesh is mentioned, it means the shortway mesh. The governing factor of the expanded sheets is the weight per sq. yd. The dimensions of the meshes, varying as they do with the thickness and width of the strands, are to be taken as nominal only.

To allow for the inclination of strands the safe tensile strength per ft. width of fabric shall be taken at 85 per cent of the permissible tensile stress of the strands and the cross sectional area listed in the table. Working stress in tension is taken not more than 27,000 lbs./sq. in.

The weight of expanded metal sheets is calculated on the basis that steel weighs 3.40 lbs. per sq. in. of nominal cross sectional area per ft. run. The cross sectional areas and weights given in the table vary slightly with different manufacturers. Sizes of sheets normally stocked are:  $8 \times 12$ ,  $8 \times 9$ ,  $8 \times 4$ ,  $4 \times 12$  (LWM  $\times$  SWM), but sizes upto  $12 \times 24$  may be available.

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## SECTION 5

### PROPERTIES & USES OF METALS

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## 1. DEFINITIONS OF TERMS

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

*Critical temperature.* The temperature at which the elements of steel go into solution.

*Drawing* is the pulling of round sections or wires through holes of successively smaller diameters (dies) which stretches the length and reduces the diameter.

*Ductility* is the property of being permanently extended or stretched (i.e., drawn into wires) by a tensile force to a smaller section (before fracture).

*Elasticity* is the property of a solid material whereby it returns to its original dimensions upon the removal of the applied load, provided the load has not exceeded a certain limit called the *elastic limit*. 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 a load causing stress less than the 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. (Also defined in Section 3.)

*Forging.* Moulding a hot piece of iron to a different shape by hammering.

*Ingots.* Castings of uniform sizes and shapes for subsequent rolling, forging or processing.

*Malleability* is the property of being permanently expanded or flattened into sheets without fracture when rolled or hammered.

*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.

*Pickling.* Removing scale from steel by immersion in a diluted acid bath.

*Quenching.* Rapid cooling by immersion, which may be in liquids (water, oil), gases or solids (molten), or 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 rollers to shape them into different steel sections such as I, L, T, rounds, flats and sheets.

*Tenacity* is that property by which a material resists a tensile force without giving way.

*Toughness* is the property to resist tendency to fracture or break under bending or impact, and which enables materials to undergo relatively large deformation at high stress.

## 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 generally in the forms of oxides and carbonates, etc. Iron occurs in various forms mixed with stone and other impurities which are called *Iron Ores*, 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 which is red in colour and an oxide of iron containing 70 per cent of iron (with impurities) and is considered to be the most valuable iron-producing ore. Brown Haematite is brown in colour and yields about 60 per cent of metallic iron. Magnetite is a black oxide and when pure yields about 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, Mysore, Central India, Saurashtra, Rajputana and Utter Pradesh and are of different qualities, that from Assam is considered to be the best.

Iron ores are heated in a special type of furnace called Blast Furnace, under intense heat, which is called *smelting*. Limestone, clay and sometimes sand, is added with the ores as a flux, and charcoal and coke are used as fuel. 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 then 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 refining processes so that only the requisite quantity of carbon and other elements remain. The pig iron from blast furnace contains about 90 per cent of iron. Chemically pure iron is not an article of commerce.

Carbon plays a very important part on iron. 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 more suitable for foundry casting works.

### **Simple Field Tests to Distinguished Different Irons**

*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 ring, wrought iron a note of a low pitch while cast iron when struck gives a hollow sound.

*Fracture.* A fracture in steel is granular, in wrought iron fibrous and in cast iron crystalline in appearance.

*Acid test.* Dilute nitric acid applied to a clean fracture of grey iron will produce a black stain and to a white cast iron a brown stain, while it will produce a greenish stain on wrought iron and dark grey stain on steel.

### **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 blast furnace. Lime is usually added as flux and coke is used as fuel for melting the pig iron. Old castings or scrap iron are sometimes added with the pig iron before melting, these improve the quality of 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.

Cast irons are generally classified into three main varieties: Grey cast irons, White cast irons and Malleable cast irons. Grey cast iron is softer and tougher than white cast iron and can be easily machined, while white cast iron is hard and brittle and cannot be easily machined. Grey cast iron is generally used for engineering works and ordinary castings. These irons have ultimate tensile strengths of 9 to 25 tons/sq. in.

*Malleable Cast Iron:* Is annealed white cast iron. It is stronger and more ductile than ordinary cast iron but is inferior to cast steel. It is softer and tougher than ordinary 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.

The heavy percentage of carbon in cast iron makes it brittle, non-malleable and non-ductile metal. It cannot be forged at any temperature, rolled, drawn or welded under a hammer. Cast iron cannot be punched or riveted like steel but it can be easily melted (is fusible at 2000 deg.F.) and cast into intricate shapes and machined. Molten fluid shrinks on cooling from about 1 per cent to 3 per cent according to the shape of the casting. This iron is very strong in compression but weak in tension and cracks when subjected to shocks. Normally offers excellent resistance to corrosion compared with any other ferrous metal. It can be made very hard by rapid cooling.

Cast iron can be made hard and malleable by alloying with nickel and chromium, and non-corrosive with brass, bronze and other such alloys. Very intricate castings can be produced from alloyed cast irons. Its use in industry is second only to steel.



Cast iron should never be used horizontally for either heavy or variable loads, nor where the slightest liability for shock exists for cast iron gives little or no warning of approaching failure under tensile stresses.

**Foundry Work.** Patterns of the articles to be moulded are made either of well seasoned wood, metal or plaster of Paris, and kept about 10 per cent bigger in size to allow for the shrinkage of the molten metal on solidifying. Special foundry sand is used possessing the necessary properties of adhesion. Pure silica sands with very small quantities of alumina and lime give best results. 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. The pattern is usually made in parts to facilitate its removal. After the casting has solidified and cooled it is taken out from the mould, cleaned with a wire brush and irregularities on the surface are removed by either filing or chipping. Cast iron should be painted soon after it leaves the mould to preserve intact the hard skin before it has time to rust; oxide of iron paints should be preferred to lead paints.

**Defects in Castings.** A defective casting gives a dull and deadening sound when gently struck with a small hammer, which indicates blow-holes and air bubbles in the body of the casting. A good casting should not have any cracks or honeycomb surfaces. The casting should be close-grained and sound to be corrosion resistant. Blow holes are due to the formation of steam from the damp moulds; sound-holes due to misplaced sand particles; rough surfaces are due to the 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.

**Uses of Cast Iron.** Building columns, caps and bases of columns, brackets, water and sewage pipes, wheels, spiral staircases, gratings, agricultural implements, etc.

All cast iron 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 chisel or drill.

Hard and brittle material is commonly light grey in colour having little or no lustre. A tough iron is marked by a uniformly diffused dark grey colour having a good lustre; a weak material is mottled in colour and without lustre. A light grey colour with pronounced lustre indicates a hard and tenacious iron, and when a fracture shows up as a much mottled dark colour with an entire absence of lustre, the suitability of the iron should be suspected with a view to rejection. 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.** Castings must be of such a strength that a test bar cast from the same heat of metal, 2 in. deep  $\times$  1 in. wide, placed upon bearings 3 ft. apart, will sustain without fracture a weight of 27 cwts. placed at the centre. Or, bars 1 inch square in section laid upon supports 1 ft. apart, should resist a load of 1 ton placed in the centre of their length, or a central load of 500 lbs. on a clear span of 4 ft. 6 ins.

#### 4. PROPERTIES AND USES OF WROUGHT IRON

Wrought iron is low in carbon content and is made from white pig iron by remelting and purifying it in the Puddling furnace. It is the purest form of iron with carbon not exceeding 0.15 per cent. It possesses the important qualities of toughness, ductility, malleability and weldability at white heat (softens at 1500 to 1600 deg. F.) It is not fusible except at a very high temperature of 3000 deg. F. It becomes soft and plastic at red heat and could be easily forged at about 1650 deg. F., and can be bent or twisted when either hot or cold, but it cannot be cast into moulds. Is not appreciably hardened by quenching (suddenly, cooling) or tempering. Some grades are more rust resisting than steel and are at times used on hydraulic and marine works. It can be worked more easily than steel in threading machines. Mild steel which is stronger than wrought iron is now largely replacing it.

**Uses of Wrought Iron.** Wrought iron is used for making spikes, nails, bolts and nuts, wires, chains, horse-shoe bars, sheets and plates, stay bolts, fish plates, ties, handrails, ornamental gates, straps for timber roof trusses, pipes and tubing, armatures, electromagnets, etc. 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 22 tons/sq. in. Rivet and bolt bars should stand bending double when cold without cracking. Bars over 2 inches 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.

## 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 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. A general classification is as follows:—

Carbon	0.10 to 0.25%	.. Mild steel, low carbon steel.
Carbon	0.26 to 0.60%	.. Medium carbon steel, medium high carbon steel.
Carbon	0.60 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 is hard for drawing



into wires, and a very hard and brittle steel is required for making tools.

Steels with carbon less than 0.10 per cent are soft steels. Carbon in structural steel and high tensile steel is not more than 0.30 per cent and in rivet bars not more than 0.25 per cent. Increase of carbon percentage increases the tenacity and hardness with a corresponding decrease in ductility and toughness. The tensile strength, hardness, yield point and elastic limit of plain 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 which is between 1535 deg. C. for pure iron free from carbon to 1130 deg. C. for 1.8 per cent of carbon steel. Most steels become hard and more or less brittle by hardening but very low carbon steels (and wrought irons) cannot be hardened or tempered (but they can be "case hardened"). Harder varieties of steel are better suited for tempering as carbon renders the steel susceptible to hardening, the degree of hardness obtained by the heat treatment depends upon the carbon content. Steel always contains some or all of the elements—manganese, silicon, sulphur, phosphorus; sulphur and phosphorus are next in importance to carbon.

Most of the steels are highly elastic, malleable, ductile, forgeable, weldable, capable of receiving different degrees of hardness by tempering and are fusible at a lower temperature than wrought iron (2400 deg. F.). Steels have much higher tensile and compressive strengths than wrought iron and will stand wear and tear much better. Smithing of steel is more difficult than that of wrought iron and it is more liable to injury from over-heating. Steel plates sustain greater injury when punched than wrought iron, therefore it is preferable to drill holes in all steel plates.

All finished steel should be sound, free from cracks, surface flaws, laminations, splits, and rough, jagged and imperfect edges or any other surface defects.

**Mild Steel.** Contains 0.2 to 0.5 per cent of carbon. Is an elastic but ductile material and is superior to wrought iron in ductility and strength. It can be easily cut, machined, punched or drilled, welded, forged and rolled,



but cannot be hardened or tempered 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.

### FIELD TESTS FOR STRUCTURAL STEEL

(Based on IS:432, 277, 266, 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). When sections permit, the test pieces should not be less than  $1\frac{1}{2}$  ins. For cold bend tests, the test pieces should not be subjected to any heat treatment. Tests 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 blows from a hammer until the two sides of the test piece are parallel and, in the case of bars above 1 in. in diameter or thickness, the internal radius is not greater than  $1\frac{1}{2}$  times the diameter or thickness of the bar, and, in the case of bars of 1 in. and under in diameter or thickness, the internal radius of the bend is not greater than the diameter or thickness of the bar.

#### **Tests for Rivets**

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  $2\frac{1}{2}$  times the diameter of the shank.



### **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 back 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  $37/42$  tons per sq. in. with minimum elongation of  $7\frac{1}{2}$  per cent.

### **Cold Bend Tests for Mild Steel Sheets**

#### *(a) Black Sheets:*

A strip from sheet 18 B.G. and under cut lengthwise or crosswise, shall withstand bending through 180 deg., flat on itself, without fracture; while a strip of sheet over 16 B.G., shall withstand bending through 180 deg. with an included radius equal to  $1\frac{1}{2}$  times the thickness of the sheet without fracture.

#### *(b) Galvanized Sheets:*

Shall withstand the doubling test as given for black sheets above.

Strips 1 inch 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 in IS:227—1951 for galvanized sheets:—

Test pieces, preferably 'L' shaped, 9 ins. long and 3 to 4 ins. 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 oz. per sq. ft.	Birmingham Gauge for Sheets							
	16	18	20	22	24	26	28	30
2.50	10	10	11	12	12	14	14	14
2.00	8	8	9	10	10	11	11	11
1.50	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—1954.

**Temper Bend Test.** The test piece shall be heated to a blood red colour and then quenched in water at a temperature not exceeding 80 deg. F. 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  $1\frac{1}{2}$  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 should have an ultimate tensile stress of 28 to 33 tons per sq. in.; yield stress (min.) 14.75 to 15.25 tons per sq. in. (according to thickness); elongation (min.) 16 to 20 per cent (according to thickness) on a gauge length of 8 ins. (8 diameters for round or square bars).

For steel which have no well defined yield point the yield point stress shall be taken as the stress at which the permanent set of the bar reaches a value of 0.2 per cent of the original gauge length.

For bars from which rivets are cut a more ductile steel is specified having an ultimate tensile stress of 25 to 30 tons per sq. in. and a minimum elongation of 26 per cent on a gauge length of 8 diameters.

**Working Stresses** are related to the yield stress since for normal design purposes a steel member is assumed to become unserviceable when it commences to "yield". Working stress in direct tension or compression is taken



about 60 per cent of the yield stress in tension which gives a factor of safety of about 1.7 against failure by yielding and of about 3.3 against ultimate failure.

For working stress in bending 65 per cent of the yield stress is taken, and for shear 42 per cent of the yield stress in tension since the yield stress in shear is 65 per cent of the yield stress in tension. Working stresses have been given in Section 3.

### **Mechanical Properties of High Tensile Steel.**

High Tensile Structural Steel has carbon content not exceeding 0.3 per cent (0.25 per cent for rivet bars) with ultimate tensile stress of 37 to 43 tons per sq. in.; yield stress (min.) of 19 to 23 tons per sq. in. (according to thickness). For rivet bars ultimate tensile stress is 30 to 35 tons per sq. in. 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. Used for the manufacture of high grade surgical instruments. *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.

## **6. HEAT TREATMENT OF STEELS**

Hardening, tempering and annealing 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.

**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 temperatures between 950 deg. to 1000 deg. C. according to the thickness of the section and quenched (suddenly cooled). Thin sections such as knife-blades are effectively hardened by cooling in air. 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 or oil; the quicker the cooling the harder the steel becomes. Quenching in water results in a much more rapid cooling than oil quenching. The larger the mass the longer is the time required to make the effect of the quenching reach the core, and the thickness of the material has also a great effect on the uniformity of the hardness developed in it from the surface to the interior. 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 the harder it becomes. But if the temperature exceeds 50 degrees above the upper critical temperature, it becomes too brittle to be of any practical use. The steel should be lightly tempered immediately after hardening, otherwise it may crack. Steel can also be hardened by cold mechanical work.

Suitability and purity of the *quenching liquid* is very important. Grease or acids in water are objectionable, as the grease is liable to cause uneven hardness and acids produce brittleness. Hard water is very satisfactory.

**Case-Hardening or Carburising.** Certain parts of machines which are subjected to vibrations, shocks, wear and tear have to be made of hard and wear resisting surface with tough inside. These are made of low carbon steels or wrought iron which are packed in a box containing a carbon medium and heated to a high temperature (900 deg. C.) for sometime at which the steel absorbs carbon



on the outer surface, it is then cooled suddenly by immersion in water. 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. This method is employed in small parts, such as pins, where the wearing property of steel is desired to be combined with the ductility of wrought iron.

**Liquid and Gaseous Case-Hardening Materials** are also employed. For the liquid case-hardening, the steel article is immersed in a molten bath of sodium and potassium cyanides with sodium carbonate, maintained at 900 to 950 deg. C. which results in the formation of very hard surface. For the gaseous case-hardening, various gases and gas mixtures, containing excess of carbon have been employed. The article is placed in a furnace and gases are driven through and the hot steel article absorbs carbon from them, which is then quenched to give the required hardness.

*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.

**Tempering.** Tempering is reheating of hardened steel to a temperature below its critical temperature range (or lower change-point) and subsequent cooling in air or quenching in oil or water. Slow and uniform heating for tempering is essential. In the hardening process the contraction and the dimensional change in the article is not uniform and the steel becomes very hard and brittle, therefore, another treatment is given with much lower temperature. Tempering imparts ductility, increases toughness and reduces brittleness, and hardness can be modified.

**Annealing.** A heat treatment in which the iron is reheated to slightly above the critical temperature (light red heat) and then allowed to cool slowly. This treatment 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 the ultimate strength but increases the ductility and brings back the steel to the best physical state to resist fracture under sudden stresses.

**Normalizing.** Normalizing is done in place of annealing when the reduction of ultimate strength due to annealing is to be avoided. This is reheating the steel to a temperature slightly above the upper critical range and allowing it to cool freely in air. This removes any inequalities consequent upon previous heat treatment and gives a finer grained tougher steel than does annealing. *Spheroidizing* is same as normalizing.

**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

**Galvanizing.** 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 1 to 2.50 oz. per sq. ft. 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.** Is a process for coating articles with zinc by packing them in zinc dust and heating. 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 a spraying machine under high pressure at the 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.

**Tin-plating** is a process similar to hot galvanizing, tin being used for the coating instead of zinc. The protection afforded to iron and steel (against corrosion) by tin is less effective than is that of zinc.

**Electro-plating.** Metallic coatings of chromium, nickel aluminium, copper or zinc, are given on the principles of electrolysis; the article to be electroplated is made to form a cathode. The process produces very bright surfaces.

In *Nickel-plating* the articles are usually copper-plated first. Before *Chromium-plating* the articles are nickel-plated first otherwise they do not give a good finish and there is a tendency for peeling of the chromium. The colour of chromium is silver-white and which will hold indefinitely, but nickel has a little more of a yellow tinge and will turn black in some atmospheres. A suitable electrolyte for chromium plating is 30-33 oz. of chromic acid and 0.3 oz. sulphuric acid per gallon of water.

## 8. STEEL ALLOYS

An alloy is formed by mixing two or more metals in certain determined proportions while in molten state.



### Effects of Certain Elements in Steel

*Nickel in Steel.* Nickel improves tensile strength and elasticity and reduces the brittleness of steel. It imparts hardness, ductility and resistance to shocks and fatigue. Improves endurance and wear, and gives corrosion resisting properties. The best properties are developed by a heat treatment, of quenching and tempering.

*Chromium in Steel.* Addition of 1 to 1.5 per cent of chromium to steel makes it very hard and tough, increases elastic limit and strength of steel. It is used for making chisels, drills, saw blades, files, bearing balls and rollers, safes, cutlery, etc. Both nickel and chromium added to steel give superior physical properties than when either element is added alone. Chromium imparts greater strength and hardness than nickel.

**Stainless Steels.** There are many varieties of stainless steel having diverse chemical compositions. Stainless steel usually has 18 per cent chromium and 8 per cent of nickel; chromium is an essential constituent of all such steels. It is very strong and tough, acid and corrosion resistant. Stainless steels 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 scale, pits and cracks, otherwise it will rust. Stainless steel is of almost silver-white appearance.

*Manganese in Steel.* Manganese increases tensile strength, toughness, hardness, durability and wear-resisting qualities; it is regarded as the best wear resisting steel (not abrasion-resisting). Manganese in steel has beneficial effects for hot forging, rolling, and appreciably increases the depth and effect of the hardening operation. This steel has a high electric resistance and a low co-efficient of expansion. Used for points and switches in railway track crossings, jaws of stone crushers and rollers, etc.



*Tungsten in Steel.* Tungsten imparts great hardness to steel and best type of high-speed tool steels could be made with it. Because of its high price, it is being replaced by molybdenum. Used for lathe tools, drills, rammers, cutters, etc.

*Vanadium in Steel.* Vanadium increases the tensile strength and hardness of steel. Vanadium steels are tough and will maintain their temper on heating. Hardening range of tool steels is increased and it produces a dense fine-grained steel which will retain its cutting edge under severe service. Vanadium steels are used for high speed tools, castings for locomotives and other engine frames, automobile chassis, crank-shafts, axles and springs, etc.

*Copper.* Improves the properties of cast and malleable iron and steel; increases the strength and hardness of low and medium-carbon steels, decreases the atmospheric corrosion of steel. In locations where structural steel is exposed to excessive atmospheric corrosion, copper bearing steel may be used.

*Molybdenum in Steel.* Increases hardness and strength of steel at high working temperatures and tends to prevent temper-brittleness. Molybdenum increases machinability and leads to no increased difficulties in welding. Is now being used as a substitute for tungsten.

**Tool Steels.** There are two distinct classes; (i) plain carbon steels having small amounts of alloying elements, which are generally used for hard machine tools and (ii) "high speed steels" which contain high percentages of alloying elements and vary widely in composition. Such steels retain their hardness up to relatively higher temperatures and are used for cutting hard materials or for cutting at high speeds. The heat treatment of high-speed steels need special precautions; in forging this steel over heating should be avoided. For hardening, the steel should be heated to a temperature of 760 to 780 deg. C. and quenched in either water or brine. Temperature treatment has a beneficial effect on the life of the tool. steel should not be worked after the temperature has fallen below about 700 deg. C.

## 9. NON-FERROUS METALS

**Aluminium** is found in abundance all over as an oxide and is extracted from the ore called Bauxite. Pure aluminium is very soft and ductile therefore, is alloyed with other metals-copper, magnesium, silicon, manganese, iron, nickel and zinc etc., which increase its tensile strength and hardness while retaining its characteristics of lightness and durability. Aluminium has high resistance to corrosion. Pure aluminium is highly conductive to heat and electricity, being second to copper. Aluminium is largely used in the forms of sheets, plates, bars, wires, structural parts-both cast and forged, suitably alloyed with other metals (since pure aluminium is too soft for practical purposes), and as fine powder for pigment in paints.

**Copper** is an ideal material for many purposes and is next in importance to iron for engineering works. It is light, tough, strong, ductile and malleable metal with good properties of resistance to corrosion in dry air, having neat and pleasing appearance. It can be forged, rolled or otherwise worked hot or cold and drawn into wires, and has a high thermal and electrical conductivity. Rolling, forging, drawing, pressing, hammering or other kind of working hardens copper and raises its tensile strength (although its ductility is decreased); ductility and softness are restored by annealing. Very heavily cold-worked copper, which is generally in the form of wire, may have a tensile strength as high as 28 to 30 tons per sq. in. Offers high resistance to and retains its form under high temperatures. Used for electric wires, cables, light gauge copper 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 a very soft, highly ductile, malleable, plastic, non-corrodible metal with low fusion point and very low strength. The metal is extremely resistant to atmospheric attack and is not affected by soil and sewage effluents or industrial wastes. Lead is used for gutters, flashings, cistern linings, water service pipes, soil and gas pipes,



damp-proof courses of buildings, lead sheets for roofing, cable coverings, type metals for printing, wool, (lead-wool) foil, solders for plumbing. Lead is widely used for making alloys and for paints in the form of oxides (white lead, red lead, litharge). It resists acid actions and is used for lining of tanks and pipes in contact with acids.

**Zinc** is a bluish white metal, very soft, light and highly resistant to corrosion 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 requires no painting. Has very low tensile strength. Zinc oxide is extensively used in high grade paints. It is used for alloying with copper to make *brass*.

**Tin** is a white lustrous metal occurring as an oxide ore. It is a very soft and weak metal but 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, and for the process of tin-plating.

**Monel Metal** is an alloy of copper and nickel which possesses the strength and toughness of steel and is non-corrodible. Resists sea water, alkalies and some dilute acids. It can be rolled, drawn, cast, forged, soldered, brazed and welded and can also be machined readily and maintains high tensile strength at elevated temperatures. Extensively used for valves of pumps, pipes, etc.

**Brass** is an alloy of copper and zinc, properties vary considerably according to the varying proportions of the metals. Brasses 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.

**Chromium** is a silver white metal, harder than steel and does not corrode or discolour even under intense heat. Used for alloying with steel and other metals. Chromium-plating is well-known.



**Molybdenum** is a hard and brittle dense white metal with high melting point. It is alloyed with iron, manganese, nickel or chromium in small quantities for making special tool steels. Molybdenum has higher tensile strength than mild steel and enables steel to maintain good tensile strength at high temperature's

**Tungsten** is alloyed with steel for making tool steels. This metal is three times as hard as platinum but much cheaper; is a very heavy element. Its melting point is twice as high as that of platinum. Is generally available in the form of a hard, brittle greyish powder. It is also used in the filaments of electric lamps.

**Babbitt Metal** is an alloy of tin, antimony and copper. This metal is used on important bearings under heavy loads such as crankshafts and crank pins.

**Tantalum** possesses very high tensile strength and is very ductile, is not affected by acids. It is alloyed with steel for making cutting tools, drills, files, etc., and for making acid resisting alloys.

**Vanadium** is a brilliant white metal of great hardness used as an alloying element in steel. It is a costly metal.

**Bronze** is an alloy of copper, zinc and tin and contains about 80 per cent of copper. "Phosphor-bronze" contains about 1 per cent of phosphorus and is extensively used for anti-friction bearings, pump rods and for such purposes where a metal is exposed to the weather as it has good strength and non-corrosive qualities.

**Gun-metal** is an alloy of copper, tin and zinc and sometimes lead is added, or is bronze to which tin has been added. This metal has high tensile strength and elasticity.

**Bell-metal** is alloy of copper and tin; iron, zinc and lead are frequently added. This metal is used for the manufacture of bells.

**German Silver** is an alloy of copper, zinc and nickel.

**Nickel Silver** is so called because of the silvery white appearance. Alloy of copper, nickel and zinc.

**White or Bearing Metal** is an alloy of tin, lead and antimony with some copper. It presents a smooth surface and accommodates itself for any defects in the alignment of bearings due to insufficient lubrication and production of excessive heat.

**Fusible Alloys** are a group of non-ferrous alloys which melt at very low temperatures. They are usually made of lead, tin and bismuth in varying proportions and iron only as an impurity. Fusible alloys are used as safety measure in controlling the rise of temperature.

Metal			Relative Electric Conductivity	Relative Thermal Conductivity
Silver	..	..	106	108
Copper	..	..	100	100
Gold	..	..	72	76
Aluminium	..	..	62	56
Zinc	..	..	29	29
Nickel	..	..	25	15
Iron	..	..	17	17
Steel	..	..	13-17	13-17
Platinum	..	..	16	18
Tin	..	..	15	17
Lead	..	..	8	9

**Co-efficient of Linear Expansion for 1 deg. F.**

Brass	.0000105	Steel	.0000067
Copper	.0000095	R.C.C.	.0000065
Lead	.0000158	Plain concrete	.0000060
		Brickwork	.0000030

Total expansion of a structure in inches = co-efficient of linear expansion  $\times$  length of the structure in inches  $\times$  change of temperature in deg. F. If expansion is not allowed to occur, stress produced = co-efficient of expansion  $\times$  change of temperature  $\times$  modulus of elasticity of the structure metal.

## 10. GENERAL PROPERTIES OF SOME METALS

Metal	Weight lbs./c. ft.	Ultimate tensile strength tons./sq. in.	Young's Modulus tons/sq. in.	Melting point deg. F.
Aluminium	160-169	5-9	4500	1190
" (cast)	160	6	..	..
" (sheet)	167	12	..	..
Brass (cast)	500-520	8-16	..	1652
" (sheet)	500-520	13	..	..
" (wire)	500-520	19-22	6700	..
Bronze (cast)	553	16-20	6700	..
Cast iron	430-445	9-25	6500-7500	2200
Copper (bolts)	558	16	..	1961
" (cast)	558	8.5	..	..
" (sheets)	558	13.5-15	..	..
" (wire)	558	20-27	8000	..
" (wrought)	558	15	..	..
Gun metal	540	13-19	5800	..
" " (cast)	540	18-22	..	..
Lead (cast)	707	0.81	..	618
" (pipe)	707	1.0	1000	..
" (sheet)	707	0.85	..	..
Nickel	550	38-45	11000-13500	..
" iron	548	38-51	10250	..
" silver	548	22-25	8000	..
Platinum	1342	118	..	3246
Silver	655	18	5100	1795
Steel (structural)	490	28-33	13400	..
Tin	454	2	3450	449
Wrought iron	480-484	20-25	12500	3000
Zinc (cast)	428	2	..	784
" (rolled)	446	8-11	5500	..
" (sheet)	448	8	..	..

*Working Stresses* for Steels and Irons have been given in Section 3.

## 11. 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 types of the various processes : (i) Welding with pressure, which includes Forge welding and Thermit Welding (with pressure), and (ii) Fusion Welding (without pressure) which includes Gas welding, Arc welding and Thermit welding (without pressure). The Metal Arc welding of the Fusion welding processes is the most important and is most extensively used.

*Forge Welding or Plastic Welding.* The metals are heated to a plastic state and the edges to be joined are pressed or hammered (manually) together. 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. The heating of the metals may be done through a blacksmith's forge fire, or by means of an electric current passing through the parts to be welded. This process is confined to the welding of rods and small pieces.

**Metal-Arc Welding.** An arc which is a low voltage, high current discharge, is formed between the work to be welded, which is connected to one pole of the electrical system, and an electrode, which is connected to the other pole. The heat of the arc which is about 5000 deg.C., raises the workpiece 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. 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. Welding of cast iron can be done with a ferrous electrode of soft iron. Smallest gauge electrode suitable for the job should be used and the casting should not be heated more than is necessary. Welding Generators and Transformers are available for D.C. or A.C. currents. Most of the electrodes for welding mild steel operate satisfactorily on D.C. or A.C. but some materials may require A.C. and others D.C.

**Oxy-acetylene Welding.** In this process oxygen and acetylene are fed through a blowpipe, acetylene gas burning in pure oxygen produces a flame which is ignited at its tip. The flame has a temperature of approximately 3000 deg.C., and the edges of the two adjacent pieces of the metal to be welded together are heated to fusion with this high localized temperature; additional metal is added to the joint by melting into it a suitable electrode.

The proportions of oxygen and acetylene forming the oxy-acetylene flame is an important factor and the gases must be mixed in correct proportions otherwise defective welds will be produced. Three flame conditions are developed, viz. (a) Neutral, in which equal quantities of oxygen and acetylene are supplied and which is required for most of the iron works. (b) Oxidizing, in which excess of oxygen is supplied by the blow-pipe. 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. 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.

The oxy-acetylene welding blowpipes are so designed as to bring the two gases together and to 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. Copper requires a larger nozzle than steel.

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  $\frac{1}{4}$  in. 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 coloured glass goggles.

The oxy-acetylene welding of mild steel is generally carried out with a neutral flame. 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.

*Oxy-acetylene Welding Equipment* normally comprises (i) Oxygen cylinders; (ii) acetylene cylinders; (iii) regulators and gauges; (ix) 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.

Oxy-acetylene welding is a slower process than metal-arc welding, especially on heavy sections, and for thicknesses greater than  $\frac{3}{8}$  in. metal-arc welding is preferred. This process is generally used for repair works and for thin metals as it has low operating and equipment cost.

*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. The steel is first heated locally and then a jet of high purity oxygen is introduced on to the steel. This oxygen cutting process can also be used effectively for cutting iron and steel under water.

*Welding Rods or Wires (Electrodes) :*

The choice of the right size of welding rod is important and is based on the thickness of the sheets or plates to be



welded; the melting of both should take place in the molten pool and not in the flame. Ordinary welding wires of mild steel or iron are unsatisfactory but wires containing a small percentage of vanadium or nicked produce highly satisfactory results. Coated electrodes should be preferred to bare wire electrodes.

A simple test to 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 3 or 4 inches 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.

#### *Size of Electrodes*

Average thickness of plate or section	Max. gauge or dia. of electrode to be used
Less than $\frac{3}{16}$ "	10 S. W. G.
$\frac{3}{16}$ " to $\frac{5}{16}$ "	8 S. W. G.
$\frac{5}{16}$ " to $\frac{3}{4}$ "	6 S. W. G.
$\frac{3}{4}$ " to $1\frac{1}{8}$ "	4 S. W. G.
$1\frac{1}{8}$ " to 1"	$\frac{5}{16}$ " dia.
1" and over	$\frac{3}{8}$ " dia.

### **Safety Precautions for the Use of Oxy-acetylene Equipment**

#### *Handling and Storage of Cylinders :*

Do not allow cylinders to drop.

Do not mix gases in a cylinder or fill one gas cylinder from another.

Acetylene and oxygen cylinders should be kept apart, preferably in separate rooms which should be fire-proof and well ventilated top and bottom.

Acetylene cylinders should not be stacked but should always be stored up-right.

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 leads.

Grit, dirt of any sort, oil or water should not be allowed to enter cylinder valves. Dust and moisture should be expelled by opening the cylinder valve before the cylinder is put into use.

Any part of oxygen apparatus, (valves, fittings) or supply line must not be oiled or greased or otherwise repaired with white or red lead.

Repairs to the cylinder valves should not be attempted by the user.

*Use of Equipment :*

Welders and work should, as far as possible, be protected from wind and weather.

A blow-pipe should never be put down or hung from equipments or stands unless the gases are turned off.

Equipment should be tested occasionally for leaks and before attaching any fitting, and it is also very important to see that the correct gas cylinder is being employed for work.

The valves of the cylinders should always be opened slowly and when closing no excessive force should be used and the valves should be closed just sufficient only to shut off the gas. Cylinder valves should be shut when cylinder is empty or when work has stopped for more than a few minutes.

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. If internal ignition occurs frequently the blowpipe should be sent for repairs. By "back-fire" or "flash-back" is meant the momentary return of the flame into the blowpipe tip which may relight immediately upon withdrawing the blowpipe away from the work or necessitate reignition.

*Rubber Hose.* It is very important to inspect the rubber hose periodically for any cracks, cuts or worm out places. All connections should be securely made as leakages are very dangerous.



### Precautions Against Fire:

Sparks from a welding or cutting operation are often thrown a considerable 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 or tanks can be filled with water to within an inch or two of the point where the flame is to be employed, as a precautionary measure.

All welders should wear special goggles fitted with suitable coloured glasses which should not have inflammable lenses or frames, for protection from sparks and to prevent eye-strain. Welders employed on heavy works should be supplied with fire-proof gauntlets and aprons as a protection against radiated heat and where welding is done in the interior of some structure or confined space all the outer clothing should be fire-proof.

**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. A specially prepared mixture of aluminium and red oxide is placed in a crucible and ignited with a special powder. The reaction generates great heat and molten iron at a very high temperature is produced which is poured over the edges to be joined, which are pre-heated, and it fuses with the surfaces of the metal forming a solid joint.

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.

In the ordinary 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. Beveling ensures good penetration, increase in ductility and strength. Materials of thickness less than  $\frac{1}{8}$  in. need not be bevelled. All materials from  $\frac{1}{8}$  in. to  $\frac{3}{8}$  in. or  $\frac{1}{2}$  in.



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 thorough penetration and complete fusion must be obtained. If it becomes necessary to reweld a joint previously welded, it is essential that the previous weld metal is entirely removed.

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.

Welds should be made in the flat position as far as possible.

Freedom of movement of one member should be allowed where possible. The parts to be welded should be thoroughly cleaned and proper flux used.

The strength of a welded joint should be taken only between 60 to 75 per cent, although tests have shown that if the welding is properly done it is possible to develop the full strength of the members joined.

*Flame Cutting.* There are many cutting processes involving the use of an oxygen jet or an electric arc. The most widely used is the oxygen cutting process which has already been described under Oxy-acetylene welding, which is also called Gas Cutting. 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.

## 12. SOLDERING OF METALS

Soldering is the joining of metals by the addition of molten filler metal at temperatures well below the melting point of the metals to be joined. The melting point of the solder should be less than that of the metal to be joined and 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 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. Solders 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 overheating and rapid solidifying of solder are important. Antimony in small proportion is usually added to solder metals. Solder wires are available for different types of works.

*Fluxes for Soldering or Welding.* Fluxes are used to clean the surface of the work and the solder of any existing oxides of the metals and to prevent re-oxidation during the process of heating. The presence of any impurities or dirt on the surfaces to be joined will prevent a good union. There are two main types of fluxes : (i) Strong and corrosive, and (ii) Mild and non-corrosive. Strong fluxes are Zinc Chloride, Hydrochloric Acid, Ammonium Chloride and Ammonium Phosphate. Mild fluxes are Borax, 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. The following fluxes are commonly used :

Iron or steel	Borax, Ammonium chloride.
Stainless steel	Zinc chloride and Hydrochloric acid in equal proportions.
Zinc	Zinc Chloride, Hydrochloric acid, Ammonium phosphate.
Lead	Tallow, Resin.
Copper and brass	Ammonium chloride, Hydrochloric acid.
Tin	Ammonium phosphate.
Tinned iron	Resin, Hydrochloric acid.
Lead and tin pipes	Resin and sweet oil.
Lead and brass pipes	Tallow.



# SECTION 6

## FOUNDATIONS

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## 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.

**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. 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.



## 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 following pages.

**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 12 inches; fine sands and silts have capillary rise up to 4 ft. but dry sands have very little capillarity. Clays may have capillary rise up to 3 or 4 ft. 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 consolidation" 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  $\times 62.4$  (weight or density of water per c. ft.). 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  $\div$  total vol. 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  $\div$  total vol. 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 130 lbs./c. ft. for coarse grained well graded gravels and sands to about 90 lbs./c. ft. 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, or

$$\frac{\text{vol. of voids in the soil}}{\text{vol. of solid particles}} = v$$

*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),

$$\frac{\text{vol. of voids in the soil}}{\text{total vol. of the soil}} = p = \frac{v}{1+v}$$

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 = vol. of water/vol. 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 2.6 and 2.7. The usual soil weight (voidless) is between 162 and 169 lbs.

### 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.

**Hyroscopic 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



**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 10 ft. below the surface. Foundations on such clays should be placed at a depth at least 3 ft. 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 the soil particles. Friable soils are cohesionless.

### **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 *limit 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 PI.

**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  $\frac{1}{8}$  in. 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.

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 :—  
(Burminsters' 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 ..	> 35	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 PI *i.e.*,

$$\frac{\text{Natural moisture content} - \text{PL}}{\text{Liquid limit} - \text{plastic limit}} \times 100 \text{ per cent}$$

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.

**Centrifuge Moisture Equivalent: (CME)** This is the moisture content retained by a soil that has first been saturated with water and then subjected to a force equal to one thousand times the force of gravity for one hour.

**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 two tests for CME and FME are 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 moisture 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 some 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 a natural deposit consisting of the finest aluminous product formed by decomposition of igneous rocks. The term 'clay' in broad sense is applied to a material which is tenacious and plastic when wet, possessing considerable cohesive strength, very pronounced capillarity and practically no internal friction. Clay is smooth and greasy to the touch and 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 or Stiff clay.* A clay which at its natural moisture content requires a pick or pneumatic spade for its excavation or removal and cannot be remoulded with the fingers.

*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.

*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.



A graded sand consists of a mixture of the various grades and is described as : well graded or poorly graded according to whether the proportion of the various grades are such as to give a high or low bulk density when the sand is well compacted. (These terms are described further in detail.)

**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, of which the individual grains are only hardly distinguishable by the naked eye. Silt has a gritty touch (but not very gritty) when squeezed between the fingers or bitten between the teeth. 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 possess 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 particles of size below 0.002 mm., in the form of uncrystalline semi-solid 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 6 ins. wide at the surface and extending to 10 to 12 ft. deep where the soil is thick. The thickness of these soils generally varies from 3 to 12 ft. or even more.

**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'.

**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 together 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 topsoil.

**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 topsoils 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 a 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 clays, 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 sand or sharp fine gravel, the grains of which are more or less angular.

**Pebbles:** Subangular to rounded rock fragments of sizes ranging between that which is retained on a No. 7 B.S. sieve and 3 in. in diameter. Larger pieces are known as "boulders".

**Gravel:** Subangular to rounded, water-worn stones of irregular shape and size occurring 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.

## 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 Casagranade.

**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 No. 7 B.S. sieve. The samples of soil which pass the No. 7 B.S. 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 the No. 200 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.

Grain Size of Soil Particles according to Various Systems

Soil	B. S. I. M. I. T.	A. S. T. M. U. S. B. S. U. S. P. R. A.	I. S. S. S. I. R. C.	D. S. I. R. Most particles lie between— B. S. sieve
Boulders Cobbles Gravels	over 8" dia. 3" to 8" above 2 mm.	— — above 2 mm.	— — above 2 mm.	above No. 7 (1/16")
Sands { Coarse Medium Fine	2 to 0.6 0.6 to 0.2 0.2 to 0.06	2 to 0.25 — 0.25 to 0.05	2 to 0.20 — 0.2 to 0.02	No. 7 and 25 No. 25 and 72 No. 72 and 200
Silts { Coarse Medium Fine	0.06 to 0.02 0.02 to 0.006 0.006 to 0.002	{ 0.05 to 0.005 "	{ 0.02 to 0.002 "	pass No. 200
Clays	under 0.002	under 0.005	under 0.002	under 0.002 mm.

B. S. I. is British Standards Institution; M. I. T. is Massachusetts Inst. of Tech. (U.S.A.); A. S. T. M. is American Society for Testing Materials; U. S. B. S. is U. S. Bureau of Soils; U. S. P. R. A. is U. S. Public Roads Administration; I. S. S. S. is International Society of Soil Science. (The particle size adopted by International Assoc. for Hydraulic Structures Research is different from the I. S. S. S.) D. S. I. R. is Deptt. of Scientific and Industrial Research, England. The classifications adopted by other agencies employ different particle diameters for the division points.



**Grouping of Soils** according to B.S. 1924 : 1953 :—

Fine-grained soils: Soils containing not less than 80 per cent passing the No. 7 B.S. sieve.

Medium-grained soils: Soils containing not less than 80 per cent passing a  $\frac{3}{4}$  in. B.S. sieve.

Coarse-grained soils: Soils containing not less than 80 per cent passing a  $1\frac{1}{2}$  in. B.S. sieve.

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 10 types according to their texture as follows:

Class	% Sand	% Silt	% Clay	Proportions are determined by sieve analysis. The name denotes the constituent predominating in their composition.
Sand ..	80—100	0—20	0—20	
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 Casagranade Classification**

Major divisions	Description and identification	Sub-groups	Casagrande group symbol	Value as a road foundation when subject to frost action	Max. dry density at optimum compaction, lb./cu.ft. and voids ratio, $e$	Equivalent U.S.-PRA classification Group
Gravel and gravelly soils	Boulders, cobbles	Boulder gravels	—	Good to excellent.	—	
		Well-graded gravel-sand mixtures, little or no fines.	GW	Excellent.	$> 125$ $e < 0.35$	A-3
	Soils with an appreciable fraction between the 3 in. and No. 7 sieves. A medium to high dry strength indicates that some clay is present. A negligible dry strength indicates the absence of clay.	Well-graded gravel-sands with small clay content.	GC	Excellent.	$> 130$ $e < 0.30$	A-1
		Uniform gravel with little or no fines.	GU	Good.	$> 110$ $e < 0.50$	
		Poorly-graded gravel-sand mixtures, little or no fines.	GP	Good to excellent	$> 115$ $e < 0.45$	A-3
		Poorly graded gravel-sand mixtures with excess of fines.	GF	Good to excellent	$> 120$ $e < 0.40$	A-2

Coarse-grained Soils

Fine-grained Soils—Containing little or no coarse-grained material.	Sands and sandy soils	Soils with an appreciable fraction between the No. 7 sieve and the No. 200 sieve. Feel gritty when rubbed between the fingers. A medium to high dry strength indicates that some clay is present. A negligible dry strength indicates absence of clay.	Well-graded sands and gravelly sands, little or no fines.	SW	Excellent to good.	$e > 120$ $e < 0.40$	A-3
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.	Well-graded sand with small clay content.	SC	Excellent to good.	$e > 125$ $e < 0.35$	A-1
			Uniform sands, with little or no fines.	SU	Fair.	$e > 100$ $e < 0.70$	
Fine grained soils		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.	Poorly-graded sands, little or no fines.	SP	Fair to good.	$e > 100$ $e < 0.70$	A-3
			Sands with excess of fines.	SF	Fair to good.	$e > 105$ $e < 0.60$	A-2
			Silts (inorganic), rock flour, silty fine sands with slight plasticity.	ML	Fair to poor.	$e > 100$ $e < 0.70$	A-4
			Clayey silts (inorganic).	CL	Fair to poor.	$e > 100$ $e < 0.70$	A-4, 6, 7
Fine grained soils		Soils with liquid limits between 35 and 50 per cent and generally containing between 20 and 40 per cent	Organic silts of low plasticity.	OL	Poor.	$e > 90$ $e < 0.90$	A-4, 7
			Silty clays (inorganic) and sandy clays.	MI	Fair to poor.	$e > 100$ $e < 0.70$	
			Clays (inorganic) of medium plasticity.	CI	Fair to poor.	$e > 95$ $e < 0.80$	

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Fine-grained Soils—Containing little or no coarse-grained material.	having medium plasticity.	clay. Can be readily rolled into threads when moist. Do not exhibit dilatancy. Show some shrinkage on drying.	Organic clays of medium plasticity.	OI	Poor.	$\epsilon > 95$ $\epsilon < 0.80$	
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.	MH	Poor.	$\epsilon > 100$ $\epsilon < 0.70$	A-5
			Clays (inorganic) of high plasticity.	CH	Poor to very poor.	$\epsilon > 90$ $\epsilon < 0.90$	A-6,7
			Organic clays of high plasticity.	OH	Very poor.	$\epsilon > 100$ $\epsilon < 0.70$	A-7,8
Fibrous organic soils.		Usually brown or black in colour. Very compressible.	Peat and other highly organic swamp soils.	Pt	Extremely poor	—	A-8

Legend for group symbols :

G=gravel ; S=sand ; C=Clay ; M=silt, very fine sand ; F=finest ;

O=organic silts and clays ; Pt=peat ; W=well graded ; P=poorly graded ; L=Low to medium compressibility and Low plasticity ; I=medium plasticity ; H=high compressibility.

The density applies only to soils with specific gravities ranging between 2.65 and 2.75.

### 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 No. 8 B.S. sieve (No. 10 U.S. 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 foundations.

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	..	130 (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	..	120-130
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 No. 200 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 120-130 lbs./c. ft.

#### GROUP A-4.

Fine grained soils having low plasticity, consisting predominantly of silt or silt loam soils, containing only moderate to small amounts 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	.. 110-120
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 p.c. 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	.. 80-100		

#### 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	.. 80-110
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	80-110
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	90 (max.)
Shrinkage Limit	.. 30-120		

NOTE—Max. dry weight is in lbs./c.ft.; 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 Group A-5 and A-8 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 in 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 90 lbs./c.ft. 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.'

## 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, and are detailed in B.S. 1377:1948. Brief descriptions and outlines of some of the more important tests are given below.

### Determination of the Density/Moisture Relation of 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 4 in. 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.

**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 extenso-meter. 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 are plotted in the form of a dial reading *vs.* 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 weighing 30 grams is placed in a porcelain evaporating dish about  $4\frac{1}{2}$  in. dia., shaped into a smooth layer approximately  $\frac{3}{8}$  in. 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 10 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 10 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 10 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 L L is the moisture content when the soil sample flows together for  $\frac{1}{4}$  in. along the groove with 25 shakes at 2 drops per sec. Weight of the sample is 100 grams and passes the No. 36 B.S. sieve.

$$LL = \frac{\text{weight of wet soil} - \text{weight of dry soil}}{\text{weight of dry soil}} \times 100$$

**Plastic Limit Test.** A sample weighing about 15 grams is taken from the material passing the No. 36 B.S. 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  $\frac{1}{8}$  in, the soil is kneaded together and rolled out again. This process is continued until crumbling of the thread occurs at a diameter of  $\frac{1}{8}$  in. 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, (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, 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 a 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,

$\phi$  = 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  $26^\circ$ .

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-relationship 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 lbs./sq. ft. and  $\phi$  have been ascertained as given in the table below, from which the vertical load  $n$  in lbs./sq. ft. can be computed:

Soil	$C$	$\phi$ Deg.
Clay liquid .. .. .	100	0
„ very soft .. .. .	200	2
„ soft .. .. .	400	4
„ fairly stiff .. .. .	1000	6
„ stiff .. .. .	1500	8
„ very stiff .. .. .	2000	12
Silt .. .. .	0	20
Sand wet .. .. .	0	10
„ dry or unmoved .. .. .	0	34
„ predominating with some clay .. .. .	400	30
Cemented sand and gravel, wet .. .. .	500	34
Sand—gravel mixture cemented with clay, dry .. .. .	1000	34

(A safety factor of 2 should be applied to the values obtained).

Soil	Density*	$\phi$ Deg.
Compact well-graded sands and gravel-sand mixtures .. .. .	110—120	40—45
Dense well-graded gravel .. .. .	115—125	45—50
Loose well-graded sands and gravel sand mixtures .. .. .	100—110	35—40
Dense sands .. .. .	105—115	35—46
Compact uniform sands .. .. .	100—110	35—40
Loose uniform sands .. .. .	90—100	30—35
Loose fine sands .. .. .	90—100	28—34

\*Dry density in lbs./c.ft.

Immediate shear strength of clays is taken as follows :-

Type of clay	Shear strength
Very stiff boulder clays and hard clays	greater than 3000 lbs./sq.ft.
Stiff clays and sandy clays ..	3000—1500 "
Firm clays and sandy clays ..	750—375 "
Very soft clays and silts ..	less than 375 lbs./sq.ft.

The soils subjected 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.

**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  $1\frac{1}{2}$  in. dia. and 3 ins. 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 governing 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 of the soil mass. Bearing tests are made 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 the 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  $1\frac{1}{2}$  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 gradual 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 comparison 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 considerable depths (say up to about 100 ft.) 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. 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.

**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 practically 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 towards cuttings or deep basements. Reduction in the permissible load should be made if the beds are steeply inclined. Slips or cleavage planes may cause trouble if there are cuttings or deep basements 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 clays 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 consolidation settlement. The possibility of these movements is an important consideration even for light buildings. The behaviour of clayey soils 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 calys 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 appalied (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 "Quick-sand formation." When designing structures on such soils, unit load should be reduced as much as possible and never more than 2 tons/sq. ft., 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 extensive 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 a 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 ground 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 5 to 10 ft. below the proposed formation level are generally sufficient; for footings a depth of approximately  $1\frac{1}{2}$  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 3 to 5 ft., to test them by sinking a few holes to a depth of about 4 to 8 ft., 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 5 to 8 ft. 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  $\frac{3}{4}$  in. to  $1\frac{1}{4}$  ins. 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 hammer and turned after each blow. The relative hardness of the strata penetrated can be judged by the resistance to penetration during 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 solid 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 30 ft. can be made in this manner.

Another better method is by driving in a hollow tube with open end of  $1\frac{1}{2}$  ins. to 2 ins. diameter for about a foot each time and withdrawing it for examining the material caught in the tube. The tube is split at the bottom for about  $1\frac{1}{2}$  ft. with a slit of  $1\frac{1}{8}$  ins. to facilitate removal of the material. A depth of about 12 to 15 ft. 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 10 ft. to 20 ft. beyond which the relative cost 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 auger.** This simple tool is used for putting down holes to a depth of about 20 ft. in soft soils which will stand unsupported but it may also be used with lining tubes if required. Augers of 4 or 5-in. dia. (6-in. max.) have been found most successful; for very dry non-cohesive 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* 1½-in. dia. may be used. These augers are turned by two men with 3 ft. long handles. Samples of the soils brought out are examined. About 50 to 100 ft. of boring can be done in a day. Where auger borings 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 8-in. in diameter and to 80 ft. 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 winch and are turned by hand; casing is driven by means of a 'monkey' suspended from the winch.

**Shell auger** can be manufactured locally from a hollow tube of about 4 ft. length and 4-in. diameter with cutting edge at bottom and split along the length for about 2 ft. The top end is bent so that a square rod of  $1\frac{1}{2}$ -in.  $\times$   $1\frac{1}{2}$ -in. 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 prevent the sides falling 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 12 to 15 ft. long and of varying diameters, from 3-in. 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 areas 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 soil 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 2 ft. square or 30-in. dia. and  $\frac{5}{16}$  in. thick are used. Smaller plates of 18-in. or 12-in. dia. may be used where much accuracy is not desired. Size of the plate should not be less than  $\frac{1}{3}$ th of the width of the 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 9-in. 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  $\frac{1}{4}$  in. 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  $\frac{3}{4}$  in., 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 sizes of plates.

A small area will stand a heavier unit pressure for a short time than a larger area perpetually, 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  $45^\circ$ . 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 must be accompanied by suitable borings.

## 2. LOADS AND WEIGHTS ON FOUNDATIONS

### Safe Loads on Common Soils:

Rocks			lbs./sq. ft.
Hard rock	..	..	..above 20
Ordinary rock	..	..	..above 10
Sandstone ..	..	..	.. 12 to 20

Limestone .. .. .	9 to 20
Soft rock .. .. .	2 to 8
Moorum .. .. .	2 to 4
Clay shales .. .. .	10
Marl and firm shale .. .. .	6
Hard chalk .. .. .	4 to 6
Soft chalk .. .. .	1½

Intensity of pressure on a rock foundation should at no point exceed one-eighth pressure which would crush the rock.

*Cohesive soils :*

Very stiff boulder clays .. .. .	6
Hard or stiff clays and sandy clays .. .. .	3 to 4
Firm clays and sandy clays .. .. .	2
Ordinary clay .. .. .	2
Sand and clay mixed or in layers .. .. .	2
Red earth .. .. .	3
Moist clay .. .. .	1 to 1½
Soft clays and silts .. .. .	1
Very soft clays and silts and peat .. .. .	½ to nil
Black cotton soil .. .. .	½ to 1
Alluvial soil .. .. .	¼ to ¾
Alluvial loams .. .. .	¾ to 1½
Made ground (consolidated) .. .. .	½
Hoggin (compact) .. .. .	6

*Non-cohesive soils :*

Compact gravel or sand well cemented .. .. .	5 to 7
Compact gravel or sand and gravel .. .. .	4 to 5
Loose gravel or sand and gravel .. .. .	3
Compact coarse sand (confined) .. .. .	4
Loose coarse sand .. .. .	2
Compact fine sand (confined) .. .. .	3
Loose fine sand .. .. .	1
Sand with clay .. .. .	2
Kankar .. .. .	3

*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 eccentric 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 ton for each foot 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  $\frac{1}{3}$ th of the bearing capacity under square footings.

### ✓ **Average Weights of Soils and Masonry** in lbs. per c. ft.

Earth, dry to wet	100-150	Brickwork, ordinary	112-120
Sand, dry to wet	90-125	„ sundried	100
Sand and clay	125	Stone masonry	160
Gravel	90	Dry stone masonry	130
Gravel and sand	110	Lime concrete	112
Silt, dry to wet	100-110	Cement concrete	130-150



**Superimposed Loads on Floors**

Type of Building	Lbs./sq.ft. of floor area	
	Slabs	Beams, Columns, Walls, Foundations
Residential buildings, hotel bed-rooms, hospital rooms and wards .. .. .	50	40
Rooms used as offices, without storage ..	70	50
Schools, reading rooms, light workshops ..	80	70
Retail shops, garages for light cars, Banking halls, restaurants, churches .. ..	80	80
*Assembly halls, dance halls, drill halls, public spaces in hotels and hospitals, theatres and and cinemas, staircases and landings, other public places .. .. .	100	100
Ware-houses, book stores, garages for heavy vehicles, heavy workshops .. .. .	200 or actual	200

\*Experiments carried out by Prof. Johnson of Harvard University (U.S.A.), show that it is possible for a crowd of people to exert as high a weight as 181 lbs./sq. ft. of floor area. A weight of 140 lbs./sq. ft. is quite possible where there are throngs of people. A load of 80 lbs./sq.ft. 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 168 lbs./ft. run when they are three deep.

"Superimposed loads" consist of persons occupying the rooms, furniture, equipment, etc.

Minimum load for slabs of less than 64 sq. ft. area should be taken as for 64 sq. ft., and for beams of less than 8 ft. span, load should be taken as for 8 ft. Beams, ribs and joists spaced at not more than 3 ft. 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 100 lbs./sq. ft., the following reduction is made in the superimposed loads for the design of columns and foundations in multi-storey buildings:— The superimposed 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 respectively 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 foundation of about 1 to  $1\frac{1}{2}$  tons per sq. ft.; a 2-storey building, of about  $1\frac{1}{2}$  to  $1\frac{3}{4}$  tons; and a 3-storey building of about 2 to  $2\frac{1}{2}$  tons per sq. ft. A 3-storey building of the heavier type may have a weight of about  $3\frac{1}{2}$  to 4 tons per sq. ft.

### 3. DESIGN OF FOUNDATIONS FOR BUILDINGS

**Depth of Foundations**—Minimum depth of foundation is given by:

$$h = \frac{p}{w} \left( \frac{1 - \sin \phi}{1 + \sin \phi} \right)^2 \quad \text{Rankine's formula applicable to loose soils.}$$

where:  $h$  = min. depth of foundation in ft. below ground level;  $p$  = safe permissible pressure on base in lbs./sq. ft.;  $w$  = weight of the soil in lbs./c. ft.;  $\phi$  = angle of repose of the soil material.

For tall structures such as chimneys and towers,  $\frac{2}{3}$ th of the safe load on the soil should be taken and the depth  $h$  increased by  $\frac{1}{2}$ .

For values of  $\left( \frac{1 - \sin \phi}{1 + \sin \phi} \right)$  see Section 7—"Masonry Structures".

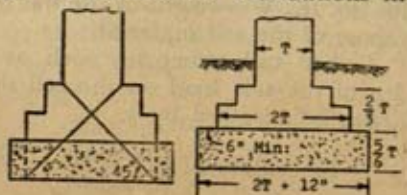
Foundations are generally taken down to about 3 to 4 ft. for main walls,  $1\frac{1}{2}$  to 2 ft. for partition walls and 1 ft. for boundary walls in ordinary good soils 2 to 3-storey building weights. But foundations must be taken down to a firm soil and below weathering effects. When part of a footing is in weaker soil, that part should be taken down deeper and separated, 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  $45^\circ$ . It is considered usually sufficient to fix the depth of the concrete by drawing lines at  $45^\circ$  from the base of the wall to intersect the side of the concrete bed as shown in the illustration. Projection of the concrete beyond the offsets should be not less than six inches 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 6 inches. Minimum size for 1 : 3 : 6 cement concrete (or good lime concrete) wall foundation would be twice the thickness of wall plus one foot for the width. The illustrations 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





d can be calculated. This formula applies to masonry walls and columns which are not considered as rigidly fixed at the bottom.  $W$ =total load due to the column or wall on the footing;  $L$ =total width of the footing;  $T$ =thickness of the column or wall;  $d$ =depth of the footing, and  $f$  is the "modulus of rupture" (explained in Sections 3 and 8) of the material of the footing. (Take all dimensions in lbs. and inches.)

The footing is also checked against "punching shear". This has been further explained under R.C. 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 continuous 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 losing 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 2 to 3 ft. 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 soils to a depth of 10 ft. The fast growing trees are especially dangerous and within 5 or 6 years the roots extend to a distance of 50 or 60 ft. 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 1 or 2 inches 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 3 ft. 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 60 ft. (25 ft. 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 for a depth of about 5 ft. and  $1\frac{1}{2}$  ft. wider, and filled with the hard material to a thickness of about 1 ft. to  $1\frac{1}{2}$  ft. and heavily rammed with water so as to force the hard material in 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 6 to 9 ins. 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 4 to 5 ft. 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 3 to 4 ins.; a layer of concrete 6 to 8 in. in thickness is placed under the lower beams. The maximum spacing of beams should not be more than 18 ins. centre to centre. Overhang ends are designed as cantilevers subject to an upward uniform load equal to the pressure on the foundation. The working stresses of the uncased beams may be increased by 33 $\frac{1}{3}$  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 a few inches below the top surface of the mat, and another layer a few inches 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 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.  
(e) 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 6-in. diameter and 4 ft. long, driven 2 ft. into the ground. These holes are filled with sand. The holes are spaced diagonally so that each hole is 2 ft. apart from those adjacent to it. Work should proceed from the centre of the trench outwards. Sand piling must never be restored 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 12-in. diameter and 10 ft. deep which are filled with sand. The spacing may be about 8 to 10 ft. according to the arrangements of the columns of the structure. The filled-in sand is thoroughly consolidated and a concrete slab laid on top of the piles. The concrete is also let into the pile holes for about 6 ins. to 12 ins. 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 1,000 lbs. per sq. ft. if water finds access to the foundations, otherwise it may be about 1 ton per sq. ft.

(ii) Foundations are taken down to such depths to which the cracks do not extend.

(iii) Trenches are dug on either side of the foundations 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 3 to 4 ft. 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 foundation trench should be about 4 ft. wide and taken down to at least 6 ins. below the depth at which the cracks cease. The bottom of the trench should be well watered and thoroughly rammed with heavy rammers. On the rammed bed a 12-in. layer of good hard moorum or other such soil is spread in 6-in. layers, well watered and rammed. On top of the moorum about 18 ins. 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-brick 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 6 ins. 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 6 ins. for compound walls and unimportant buildings and 1 ft.-6 ins. to 2 ft. for main walls. Another method similar to the above is:

Trenches are excavated to a depth of from 5 to 6 ft. and width greater than the width of the bottom of footings by  $1\frac{1}{2}$  ft. Cement concrete is filled in to a thickness of 9 ins. on both sides of the trench bottom for a width of 9 ins. on



either side, thus leaving a space equal to the width of the bottom of the masonry and 9 ins. high which is filled with sand. On the top of this (for full width of the trench) R.C.C. slab is built 6 ins. thick. Masonry (foundation footings) is built on the R.C.C. slab and the 9 ins. space left on both sides of the foundation masonry is filled with sand. A vertical pipe of 3 ins. diameter is passed through the plinth masonry to the sand under the R.C.C. slab (through the masonry and the slab) which is kept filled with sand. The sand in the tube will fill up the hollows created at the bottom. Such tubes can be built from 4 to 5 ft. 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 4 ft. below the surface, is to put in shallow foundations and use two reinforced concrete courses of bands, each 4 ins. 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 the 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 foot 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 spanning 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 a 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 consist of good granular material. Fills should be rolled in thin layers of 9 ins. to 12 ins.

## 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 types 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 a 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 two feet 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 gradient 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, which is a force pump, is very convenient for foundation work as it requires no staging and can be slung in any position, being attached to a boiler generating steam by a flexible pipe only. Its range of pumping is practically unlimited, depending only on the pressure at which steam can be delivered to the pump.

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 it may tend to remove the fine material from under the foundations or the 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 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 vibrations 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. This 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 foundations. 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 1300 lbs. to 2000 lbs. 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 manufacturers. Many types of anchor bolts are used which are either removable or fixed. Removable bolts are usually provided for large machines and are fastened at the lower end 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 booked 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  $\frac{1}{4}$  in. 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  $\frac{1}{8}$  in. larger than the bolt for bolts up to  $\frac{7}{8}$  in.,  $\frac{1}{4}$  in. for 1 in. to  $2\frac{1}{2}$  ins. bolts, and  $\frac{3}{8}$  in. for larger 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.

Where 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  $120^{\circ}\text{F}$ . 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.

(iv) 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.

*Dolly.* 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



air, or internal combustion, the energy of its blows being derived mainly from the source of motive power and not from gravity.

**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.

**Types of Piles.** There are two types of piles: Bearing piles and Friction piles, and the load taken by each pile varies with the soil characteristics and the arrangement of the soil layers.

**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.

**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 5 ft.

**Friction piles** which are called "floating piles", are driven into hard strata and take load due to friction



of the soil against the surface of the piles. They depend for their stability on the continued supporting power of the material surrounding the piles and any failure of the material will involve a settlement of the pile.

In deep deposits of fairly uniform consistency and which are compressed by piles, the load carrying capacity in the case of single piles depends on the surface area of the pile. For the same superficial area of pile surface a few long piles are more efficient than many short piles.

The following is considered the frictional resistance offered by various soil materials to the pile surface:—

Sand and gravel	.. 1000 to 1800 lbs./sq. ft.
Stiff clay	.. 800 to 1200 lbs./sq. ft.
Clay and sand mixed	.. 400 to 800 lbs./sq. ft.
Dried and compact silt	.. 200 to 300 lbs./sq. ft.
Silt and soft clay	.. 50 to 100 lbs./sq. ft.

Surface area of a pile frictional resistance will give the load the pile will carry.

Friction piles must be long and should have high values for perimeter area ratio. Such piles should be longer than the width of the foundation supported (as explained under "Soil Mechanics".)

**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 premature 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 preferable 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.



**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  $1\frac{1}{2}$  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 0.001 ft. 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 non-cohesive 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:—**

"Engineerings News" Formula—Due to skin friction:

For Timber piles: (not driven to "refusal")

$$R = \frac{2 W h}{S + 1.0} \dots \dots \dots \text{for piles driven with freely falling drop hammer.}$$

$$R = \frac{2 W h}{S + 0.1} \dots \dots \dots \text{ditto. with single-acting steam hammer.}$$

$$R = \frac{2h (W + Ap)}{S + 0.1} \dots \dots \text{ditto. with double-acting steam hammer.}$$

$R$  = safe bearing power of the pile in lbs. with a factor of safety of 6;  $W$  = wt. of hammer in lbs.;  $h$  = height of fall of hammer in feet;  $S$  = average penetration in inches (per blow) in the last six blows;  $A$  = area of piston in sq. ins.;  $p$  = mean effective steam pressure in lbs./sq. in. at the hammer.

For driving heavy piles with light hammers the above formulae were found unsatisfactory and further modifications have been suggested. The modified formula is:

$$R = \frac{2 W h}{S + 0.1 \times P/W} \quad P \text{ is the weight of the pile.}$$

$$\text{Sander's Formula for timber piles: } R = \frac{Wh}{8s}$$

*Dutch Formula for pre-cast concrete piles:*

$$R = \frac{W^2 h}{n(W + w)s}$$

$$R = \frac{4W^2 wh}{n(W + w)^2} \quad (\text{modified formula widely used in Europe})$$

$R$ =safe bearing resistance of piles in tons;  $W$ =weight of hammer in tons;  $h$ =height of drop in feet;  $s$ =set or penetration of piles in ft. (per blow);  $w$ =weight of pile in tons;  $n$ =is a constant, 4 to 6 for concrete piles, 6 to 8 for timber piles.

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 Tons

Penetration of piles in inches	3 cwt. monkey			6 cwt. monkey			1 ton monkey		
	Height of fall of monkey in feet								
	4	6	8	4	6	8	4	6	8
0.25	0.96	1.44	1.92	1.92	2.88	3.84	6.40	9.60	12.80
0.50	0.80	1.20	1.60	1.60	2.40	3.20	5.33	8.66	10.67
0.75	0.69	1.03	1.37	1.37	2.06	3.74	4.57	6.86	9.10
1.00	0.60	0.90	1.20	1.20	1.80	2.40	4.00	6.00	8.00

**Pile Hammers.** These fall into 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 normally steam operated, the ram is raised by steam and drops by gravity when the steam is exhausted. The hammers are usually 2 to 4 tons in weight and have a stroke up to 5 ft. In the double-acting type, the steam raises the ram and also drives it down on to the pile. This type of ram delivers more rapid blows which for the small sizes may be as many as 300 per min. if the hammer weighs less than 1 ton. The double-acting hammer can be used without a frame.

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 1 ft. of pile. With a single-acting or drop hammer, the stroke should be limited to 4 ft. 6 ins. or less for reinforced concrete piles. The weight of the



hammer should also be sufficient to ensure a final penetrations of not less than 1/10 in. 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 5 to 10 cwts. and for driving big and heavy piles it may be about 2 to 3 tons, which gives about 80 blows per minute through a height of about 3 ft. 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 ordinarily 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 some such preservative. 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, Jamba, Kumbia, 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 6 ins. to 16 ins. (or even up to 18 ins.) in diameter or square in section. Round piles are made of tree trunks with bark stripped and square piles are cut from the heartwood of long logs. Length is 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 6 ins. sq. size. The bottom is shaped conically for a length of from  $1\frac{1}{2}$  to 2 times the diameter or about 1 ft. 6 ins. and where the ground is hard it is protected with an iron shoe of V shape. Piles protected by shoes should have a blunt end  $\frac{1}{4}$  to 8 inches in diameter. The top is provided with an iron ring or band of size 3 ins. by  $\frac{1}{2}$  in. to 1 in. 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.

Usual spacing of timber piles is 2 ft.-3 ins. to 2 ft.-6 ins. They should never be driven to "refusal". Piles are considered to be sufficiently driven when five blows fail to drive more than  $\frac{1}{2}$  in. or when the last blow does not sink the head more than  $\frac{1}{2}$  in. 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 6 ins. outside the piles and be suitably reinforced to prevent splitting.

**Testing Existing Timber Piles (in water) for Deterioration—** such as under Bridges:—

If a small bore is made with a carpenter's auger (about  $\frac{1}{2}$  in. 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

Pre-cast concrete piles may be divided into two kinds, tapered and parallel sided. They are usually of square, octagonal or hexagonal section as they are easier to cast than the round section. Square piles are most commonly used as being easy to mould and convenient to drive. Having large superficial area per unit of length they have an advantage for friction piles. Hexagonal piles are favoured for very hard driving, but shape is inconvenient for moulding. The usual size is 6 ins. to 24 ins. but piles have been made up to 36 ins. 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 16 ins.  $\times$  16 ins., an octagonal section is preferable to a square section. Square piles should have chamfered corners.

The maximum lengths of piles are usually 40 ft. for 12 ins. square, 40-50 ft. for 14 ins. square, 50-60 ft. for 16 ins. square, and 60-70 ft. for 18 ins. 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, the maximum load allowed for concrete is 600 lbs. and for steel 9,000 lbs. The 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 2 to 3 ins. 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 4 ft. from the head at right angles to the lifting hole. One in. diameter gas pipe ferrules may be fixed in the holes.

**Reinforcement for Concrete Piles.** The area of the main longitudinal reinforcement may be  $1\frac{1}{2}$  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 R.C. piles". Transverse reinforcement should be provided in the form of hooks or links of not less than  $\frac{3}{8}$  in. 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  $\frac{1}{4}$  in. dia. up to 40 ft. and  $\frac{3}{8}$  in. dia. above 40 ft. and are spaced 2 to 3 inches for lengths up to 3 times the side at each end of the pile, lengthening to 6-8 inches at the centre.

The cover over all reinforcement, including binding wire, should not be less than  $1\frac{1}{2}$  ins. of concrete, but where the piles are exposed to sea water or other corrosive influences, the cover should be nowhere less than 2 ins.

Reinforced concrete piles should be of 1 :  $1\frac{1}{2}$  : 3, or richer mix., with well graded aggregate, of maximum size limited to  $\frac{1}{2}$  in., and a slump of about  $1\frac{1}{2}$  ins.

**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  $\frac{1}{4}$  of the pile diameter and a length equal to  $1\frac{1}{2}$  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  $\frac{1}{4}$  and a length of 3 times the pile diameter. Points should have cast steel shoes where penetrating hard soils.

**Lengthening R.C. 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 3 ft. Ferrules or sleeves 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 3 feet to 5 feet long, and the whole well laced by steel wire. 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 R.C. piles. With a final set of  $\frac{1}{4}$  inch for 10 blows, a 2-ton monkey with a 1 ft.-3 in. fall or a 30-cwt. monkey with a 2-ft. drop might be used where the load on the piles would not exceed 30 tons. For piles designed to carry a load of up to 40 tons with the foregoing final set, a  $1\frac{1}{2}$ -ton monkey with 2 ft.-8 in. drop, or a 2-ton monkey with 1 ft.-8 in. drop would be suitable.

**Cast-in-situ piles** are made by driving hollow tubes or heavy steel pipe casings 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 can be used for lengths up to 70 or 80 ft. They 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 10 ft. radius have been driven.

Under normal conditions no reinforcement is necessary and where required it is placed in the tube before concreting. 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 6-in. centres. The reinforcement should be exposed for a sufficient distance to permit it to be adequately bonded into the pile cap.

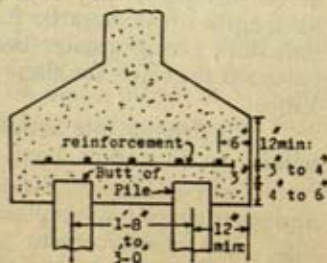
In bored piles care should be taken to prevent the influx of soil into the casings during boring. Before placing the concrete the holes should be inspected by lowering a light for any undersirable materials or water. Placing of concrete should not be started until all the shells in a group have been driven and, in general, until all driving within a radius of 15 ft. 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 3 ins. to 10 ins. and the blade is 1 ft.-6 ins. to 5 ft. in diameter. When the blade is 5 ft. 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 Foundations over Piles. (Capping).** The pile tops should be extended into foundations of the structure for 6 to 9 inches and embedded in concrete, or a space 6 to 12 inches below the top of the piles and 1 foot outside the piles is excavated and concrete placed around and above





the piles. Sometimes concrete is stripped off from the top of the piles for a length of about 2 ft. 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 20 ft. 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:	8 cwts.	5 ft.	$\frac{1}{2}$ inch in 30 blows
or	15 cwts.	15 ft.	$\frac{1}{4}$ inch in 10 blows
or	8 cwts.	30 ft.	$\frac{1}{2}$ inch in 10 blows

### **Pile Foundations for Bridges**

Pile load is limited to 50 tons for R.C. piles and 20 tons for timber piles.

Spacing of Piles—min. 3 ft.-6 ins.

Reinforcement for R. C. piles not less than 1 per cent.  
Driving hammer not less than the weight of pile. Drop of hammer 3 ft. to 4 ft.

Piles must be driven below bed level to a depth not less than twice the max. depth at Observed High Flood, ordinarily subject to a min. of 10 ft. 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 3 ins. 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.

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 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 3 ins. 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 5 to 7 lbs./sq. in. in sand and 30 to 40 lbs./sq. in. 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 2 ins. 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.

**Raking Piles.** Vertical piles have some resistance to lateral loads if driven sufficiently deep into compact soil. Racking piles are driven to take 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 hydraulic 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 2 ins. in diameter and 7 to 8 ft. long, about 1 ft. 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 10 inches diameter and



3 ft. long provided with four handles of  $\frac{3}{4}$  in. iron, screwed by wood screws to the block under 2 ins. iron rings shrunk on at either end. Down the centre of the monkey is a  $2\frac{1}{2}$  ins. 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 generally supported by a waling and the tie rods. For walls of limited height 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 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 the face of the retaining wall.

The spacing of the tie rods is generally 10 to 15 ft. 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 anchorages 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.

Sheet piles are either of steel, concrete or timber. Steel piles are generally standardized or patented and are provided with longitudinal interlocking 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 10 to 12 ft. long, 9 ins. to 12 ins. wide and 3 ins. thick. They are made of 3 planks of 1 in. thickness 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 6 to 10 ft.

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 0.003 in. in sea-water and 0.002 in. in fresh water. A protective coating of tar or some similar anti-corrosive paint should be applied periodically.

### Safe Loads for Pre-cast Reinforced Concrete Piles

Size of pile in ins.	Max. load in tons	Weight of pile shoe in lbs.	Max. length in ft.	Dia. in inches of main bars for length of:					
				20'	30'	40'	50'	60'	70'
10×10	20—25	25	30	$\frac{5}{8}$	$\frac{3}{4}$	..			
12×12	35—40	30	40	$\frac{3}{4}$	$\frac{7}{8}$	1	..		
14×14	50—55	40	55	$\frac{7}{8}$	$\frac{1}{2}$	1	$1\frac{1}{2}$	..	
16×16	65—75	50	65	1	$\frac{1}{2}$	1	$1\frac{1}{2}$	$1\frac{1}{4}$	..
18×18	80—90	60	75	..	..	1	$1\frac{1}{2}$	$1\frac{1}{2}$	$1\frac{1}{2}$

### Safe Loads in Tons for Timber Piles and Struts

Length in ft.	Size				
	4"	6"	9"	12"	15"
6	5.3	13.8	33.5	61.5	97.7
10	4.0	11.5	20.3	57.6	93.2
20	..	7.2	22.0	45.9	79.0
30		..	16.2	36.2	64.9
40			..	28.8	53.8
50			..	23.6	45.2

**Reduction Factors for Piles Acting as Columns**

Ratio of $l/r$	Timber	R.C.	Steel	Cast Iron	$l$ is effective length and $r$ is least radius of gyration. Based on CP 4: 1954.
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	..	

The least width and cross-sectional area of a taper pile should be based on the dimensions at a point two-thirds of its exposed length from the top of the pile, the exposed length being increased as mentioned above if the top stratum consists of very soft clay or mud.

Piles should not generally be loaded above 15 to 20 tons except for bridges.

**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 2 ft.-6 in. centre to centre or twice the least width of the



piles, whichever is greater, and 3 ft. spacing should be aimed at. Friction piles should be not less than 3 ft.-6 in. or the perimeter of the piles, whichever is greater. For heavy piles, the maximum spacing varies from 5 ft. to 7 ft.-6 in. 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 filling.

### **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 8 to 12 ft. by an auger. The most suitable hand auger is the bucket type post-hole auger. Average spacing of the piles is about 8 ft. 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 1 to 2 ft. deep and each lift should be thoroughly compacted before the next is poured. A lightly reinforced concrete beam about 12 ins. wide and 6 ins. 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 4 ft. long and  $\frac{3}{4}$  in. diameter should be set 2 ft. 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 2 ft. greater than the anticipated length of the pile, with a minimum depth of 12 ft.

### Load Bearing Capacity of Bord Piles

Strength Classification	Dia. of pile	Length of pile			
		* 6 ft.	8 ft.	10 ft.	12 ft.
Firm at 2 ft. and stiff at 8 ft.	10 ins.	2 tons	4 tons	5 tons	5 tons
	12 ins.	3 "	5 "	6 "	7 "
	14 ins.	4 "	6 "	7 "	8 "
Stiff at 2 ft. and hard at 8 ft.	10 ins.	4 "	6 "	8 "	..
	12 ins.	5 "	7 "	9 "	..
	14 ins.	6 "	9 "	11 "	..

\* 6 ft. 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.)

## SECTION 7

### GENERAL MASONRY STRUCTURES DESIGN & CONSTRUCTION

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# 1. PERMISSIBLE LOADS ON MASONRY

Average for good class materials.

(Safe permissible loads on Ground or Soil are given in Section 6.)

				tons./sq.ft.
Cement concrete	1 : 1 : 2	..	..	38
—ditto.—	1 : 1½ : 3	..	..	32
—ditto.—	1 : 2 : 4	..	..	28
—ditto.—	1 : 3 : 6	..	..	18
—ditto.—	1 : 2½ : 5	..	..	15
—ditto.—	1 : 4 : 8	..	..	10
Mass cement concrete	1 : 6	..	..	18
—ditto.—	1 : 8	..	..	15
—ditto.—	1 : 10	..	..	10
—ditto.—	1 : 12	..	..	5
Lime concrete		..	..	4
Brickwork in cement	1 : 3	..	..	9
—ditto.—	1 : 4	..	..	8
—ditto.—	1 : 6	..	..	5
—ditto.— in lime	..	..	..	4
—ditto.— in mud	..	..	..	2½
—ditto.— Sundried	..	..	..	1
—ditto.— country, in lime	..	..	..	3
—ditto. ditto. in mud	..	..	..	1½
Stone masonry, ashlar, in cement	1 : 3	..	..	16
—ditto.— —ditto. 1 : 6	..	..	..	8
—ditto.— —ditto.— in lime	..	..	..	7
Coursed rubble masonry in cement	1 : 4	..	..	10
—ditto.— —ditto.— 1 : 6	..	..	..	5
—ditto.— in lime	..	..	..	4½
Random rubble masonry in cement	1 : 4	..	..	8
—ditto.— —ditto.— 1 : 6	..	..	..	4
—ditto.— in lime	..	..	..	3
Block masonry in 1:3 cement mortar, average crushing strength of block not less than:				
500 lbs./sq. in.				2½
1000 "				6
2000 "				9½
Solid cement concrete block masonry				
in cement 1 : 3	..	..	..	15

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  $33\frac{1}{3}$  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 recommended by I.R.C. :—

	tons./sq. ft.
1st class dressed stone or cement concrete	
block masonry in cement mortar 1 : 3	1.30
—ditto.—                                1 : 2	0.65
1st class brick masonry in cement mortar 1 : 3	1.30
—ditto.—                                in lime mortar 1 : 2	0.65
Lime concrete masonry with stone metal and hydraulic lime mortar.	0.65

## 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.

**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 2 ft. 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 3-in. 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 6 to 8 ft. 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 2 ins. 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)  $\frac{1}{2}$ -in. 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)  $1\frac{1}{2}$  to 2 ins. cement concrete 1:2:4. Two coats of Maxphalte 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  $\frac{1}{4}$  in. thick over the concrete. A layer of tough asphalt about  $\frac{3}{8}$  in. 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 6 ins. above the highest level of the ground. In Northern India plinths are usually kept  $1\frac{1}{2}$  to 2 ft. 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 3 ins. to 6 ins. 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 waterproofing 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 due 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 surfaces.

✓ **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  $\frac{1}{4}$  in. 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 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  $\frac{3}{4}$  lb. pulverized alum for each cubic foot of sand. Mix all the three dry and then add the proper quantity of water in which has been dissolved  $\frac{3}{4}$  lb. soft soap per gallon of water, and thoroughly mixed. This mortar is applied as plaster.

**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.

**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 4 oz. shellac dissolved in 1 quart 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 12 ins. 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. (B.S.—C.P. 111)

(Both the terms—Column and Pier, are very loosely used.)

**Pier.** A member, similar to a column except that it is bonded into load bearing walls at the sides and extends to full height of the wall. Piers are usually in the form of thickened sections of a 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.

**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: (B.S.—C.P. 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— $1\frac{1}{2}$  actual height.
- (ii) For walls with lateral support at top— $\frac{3}{4}$  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  $\frac{1}{3}$ rd to  $\frac{1}{4}$ th (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.

The B.S. Code of Practice CP 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
$\frac{H}{24}$	$\frac{H}{18}$	$\frac{H}{16}$	$\frac{H}{12}$	$\frac{H}{9}$	$\frac{H}{8}$

H is height of wall.

For dwellings of not more than two storeys, the thickness of walls may be taken  $H/32$  for works built in cement mortar 1:3.

The above values may be taken for unbraced lengths up to 45 ft. For walls of greater lengths, the thickness should be increased by  $\frac{1}{2}$  brick for cement walls,  $\frac{3}{4}$  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."

## THICKNESS OF WALLS FOR RESIDENTIAL BUILDINGS

Height of wall above plinth	Length of wall	Thickness of wall in		
		cement mortar 1:3 or 1:4	lime mortar 1:2 or cement mortar 1:6	mud mortar
Up to 10'	Any	9" for the whole of its height.	9" for the whole of its height.	13½" for the whole of its height.
10' to 15'	—Ditto.—	—Ditto.—	13½" for the bottom 8' and 9" for the remaining height.	—Ditto.—
15' to 20'	Up to 30'	—Ditto.—	13½" for the bottom 10' and 9" for the remaining height.	18" for the bottom 10' and 13½" for the remaining height.
20' to 25'	Up to 30'	9" for the whole of its height if double storey, 13½" for the bottom 10' and 9" for the remaining height if single storey	13½" for the height of the 1st storey and 9" for the remaining height; 18" for the bottom 12' and 13½" for the remaining height if single storey.	18" for the height of the 1st storey and 13½" for the remaining height.
	Above 30'	13½" for the height of the 1st storey and 9" for the remaining height; ditto.	13½" for the full height if double storey; 18" for a height of 12' and 13½" for the remaining height if single storey.	—Ditto.—
25' to 30'	Up to 25'	9" for the whole of its height if double storey; 13½" at base if single storey.	13½" for the height of 1st and 2nd storeys and 9" for the remaining height.	18" for the height of 1st and 2nd storeys and 13½" for the remaining height.
	25' to 35'	13½" for the height of 1st storey and 9" for the remaining height.	18" for the height of 1st storey and 13½" for the remaining height.	—Ditto.—

Above 35'	13½" for the height of 1st and 2nd storeys and 9" for the remaining height	—Ditto.—	—Ditto.—
30' to 40'	Up to 35'	18" for the height of 1st storey, 13½" for the height of 2nd and 3rd storeys and 9" for the remaining height. 22½" for the height of 1st storey, 18" for the height of 2nd and 3rd storeys and 13½" for the remaining height.	22½" for the height of 1st storey, 18" for the height of 2nd and 3rd storeys and 13½" for the remaining height.
Above 35'	Up to 25'	18" for the height of 1st storey, 13½" for the height of 2nd and 3rd storeys and 9" for the remaining height. 18" for the height of 1st and 2nd storeys, 13½" for the remaining height. 22½" for the height of 1st storey, 18" for the height of 2nd storey and 13½" for the remaining height. 22½" for the height of 1st and 2nd storeys, 18" for the height of 3rd storey and 13½" for the remaining height.	22½" for the height of 1st storey, 18" for the height of 2nd and 3rd storeys and 13½" for the remaining height.
25' to 35'	35' to 45'	18" for the height of 1st and 2nd storeys, 13½" for the rest of its height. 22½" for the height of 1st storey, 18" for the height of 2nd and 3rd storeys and 13½" for the remaining height.	22½" for the height of 1st storey, 18" for the height of 2nd and 3rd storeys and 13½" for the remaining height.
Above 45'	Up to 35'	18" for the height of 1st and 2nd storeys, 13½" for the height of the next storeys below top storey and 9" for the height of the top storey.	22½" for the height of 1st storey, 18" for the height of 2nd and 3rd storeys and 13½" for the remaining height.
50' to 60'			



50' to 60'	Above 35'	22½" for the height of the 1st storey, 18" for the height of the 2nd and 3rd storeys, and 13½" for the remaining height.	22½" for the height of the 1st and 2nd storeys, 18" for the height of the next storeys below top storey, 13½" for the top storey.
60' to 70'	Up to 40'	—Ditto—	—Ditto—
	Above 40'	—Ditto—	27" for the height of the 1st storey, 22½" for the height of the 2nd storey, 18" for the height of the next storeys below top storey, 13½" for the top storey.

### 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 15'	Any	13½"	13½"	(i) Where the construction is proposed to be in mud mortar, the thickness of the wall will be 18" (min:) up to a height of 25' and 22½" up to a height of 40 ft.
15' to 25'	Any	13½"	18"	(ii) Additional thickness of 4½" at the base will usually be sufficient where vibrating machinery is used.
25' to 30'	Up to 45'	13½"	18"	
	Above 45'	18"	22½"	
30' to 40'	Up to 30'	13½"	18"	
	30' to 45'	18"	22½"	
	45' to 60'	22½"	22½"	
	Above 60'	22½"	27"	(iii) The base thickness is reduced towards the top; for 16 ft. from the top of the wall, the thickness should be 13½" but where the wall does not exceed 30 ft. in height, the wall for 11'-6" from the top may be 9" thick if in cement.
40' to 50'	Up to 30'	18"	22½"	
	30' to 60'	22½"	27"	
	Above 60'	27"	31½"	
50' to 60'	Up to 30'	22½"	..	
	30' to 45'	27"	..	

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 35 ft. or made  $13\frac{1}{2}$  ins. 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  $4\frac{1}{2}$  ins. and no wall should be less than  $13\frac{1}{2}$  ins., and work built in cement mortar (at least 1:6) except when the height does not exceed 3 storeys when it may be in line.

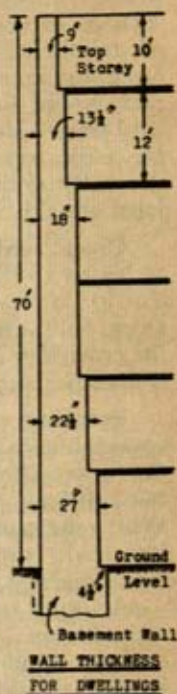
Basement and foundation walls should be  $4\frac{1}{2}$  ins. thicker than the wall thickness at ground level. Check basement walls for earth pressure.

The thickness of walls determined by the above methods should be checked for the maximum permissible loads on the masonry under the superimposed 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 9 ins. thick. The aggregate width of all recesses and openings formed at any one level shall not exceed two-thirds of the length of the wall at that level.

**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. Where, however, the external wall is not more than 20 ft. high and 35 ft. in length, the cross-wall may be built  $4\frac{1}{2}$  ins. 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  $1\frac{1}{2}$  in. by  $\frac{1}{2}$  in. with ends bent up at least 2 in., or with crosspins to form anchorage. Such anchors should be at least 2 ft. long and the maximum spacing should be 4 ft. 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 4 to 6 ft. intervals for tying the wall to the floor to have greater stiffening effect. This is not necessary for small houses.

**Cavity 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 cavities are not continuous. The most common method is of making two leaves of  $4\frac{1}{2}$  ins. thickness each with a 2-in. or  $2\frac{1}{4}$ -in. cavity. The width of the cavity should not be less than 2 ins. and not more than 3 ins. (6 ins. max.). Each leaf of a cavity wall should be not less than 3 ins. thick. The two leaves are securely tied together by cranked galvanized wrought iron wall ties,  $\frac{3}{8}$  in. by  $\frac{3}{16}$  in. with ends split and fish tailed, spaced not more than 3 ft. apart horizontally and 18 ins. apart vertically and staggered, the ties are built into horizontal bed joints during erection. Cavity walls should not be built more than 25 ft. in height and 30 ft. in length, where longer lengths or heights are desired, the walls should be divided into panels.

An 11-in. cavity wall is better than a 9-in. solid wall in heat insulation and is very much better 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  $\frac{2}{3}$  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 1 in. in the case of half-brick thick work and  $\frac{1}{2}$  in. 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 B.S.—C.P. 111.)

**Effective Thickness of a Wall Stiffened by Piers**

Ratio: Thickness of pier Thickness of wall	Pier spacing c/c Ratio: 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 and 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 inches 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 25 ft., 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. Also see 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.

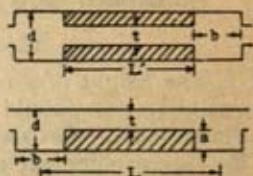
Non-bearing panelled walls are usually made of the following dimensions:

(a)  $bd^3$  should not be less than  $L' \times (d-t)^3$  or  $(L-b) \times a^3$ .

(b)  $L$  should not be more than  $15t$  for work in lime mortar and not more than  $20t$  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:

$$\frac{b \times d \times \frac{1}{2}d + L' \times t \times (d - \frac{1}{2}t)}{b \times d + L' \times t} = \text{say "a" from the projection edge.}$$

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 moduli  $Z$ , one for tension and the other for compression.



$$Z_c = \frac{I}{(d-a)} \text{—for compression on the leeward side,}$$

$$Z_t = \frac{I}{a} \text{—for tension on the windward side.}$$

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. comp.

$$= \left( \frac{W}{A} + \frac{M}{Z_t} \right); \text{ max. ten.} = \left( \frac{M}{Z_c} - \frac{W}{A} \right)$$

All isolated walls should be checked for wind pressure.

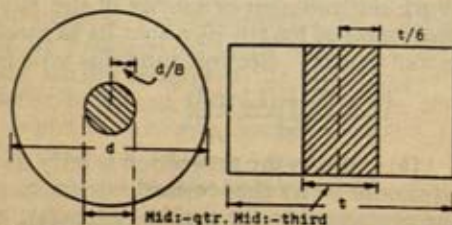
### Masonry Columns

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 before.

## LATERAL STABILITY OF WALLS

### Centre of Pressure:

For the stability of masonry structures there should be no tension produced by 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.

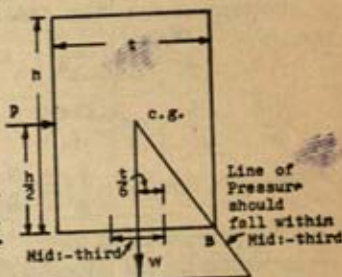


### Design of a Wall Against Wind Pressure (or any Horizontal Force)

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. ft.  
 $w$  = weight of the wall per c.ft.  
 plus any load on it.

Resisting moment of the wall = weight of the wall per unit length  $\times$  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 ft. 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 thro. 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  $\frac{1}{3}$ rd 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 80 lbs./sq. in. 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:  $\left(\frac{W}{A} \pm \frac{M}{Z}\right) = \frac{W}{A} \left(1 \pm \frac{6 \times e}{t}\right)$

$$\frac{ph^2}{2} = \frac{fI}{y} = fZ = \frac{fbt^3}{6}, \quad \text{or} \quad f = \frac{3ph^2}{bt^3}$$

$$\text{Whence, tension} = \left(f - \frac{W}{A}\right) \text{ and compression} = \left(f + \frac{W}{A}\right)$$

This gives tension and compression at the edges/sq.ft.

M=Bending Moment  
=  $W \times e$

Z=Section Modulus  
=  $\frac{bt^3}{6}$

W=total vertical load,  
(including weight of wall)  
A=bt=area of the base,  
b is taken 1 ft. 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:

$$H = T \cos \phi$$

$$V = T \sin \phi$$

H=horizontal component,

V=vertical component,

T=inclined thrust,

$\phi$ =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  $\times \frac{t}{6}$

Moment due to  $V = V \times \frac{2}{3}t$

Moment due to  $H = H \times \text{height of the thrust point above the ground level.}$

"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 coefficients given in the following table:

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** (or projections) are designed as small cantilevers. Each course of a corbel should not project more than  $\frac{1}{4}$  of the length of the brick or stone measured in the same direction as the projection and which should be only  $\frac{1}{8}$  where strength is required. The total projections of corbels in brick should not exceed 6 ins.; 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".)

**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  $\frac{1}{3}$ rd to  $\frac{1}{4}$ th 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 instead 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.



### 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 volume) :—								
	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 of 1955.)

The lime is slaked, sieved and made into a stiff paste and thoroughly mixed with sand and cement. The quick-lime should not contain overburnt 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.

Eminently hydraulic limes should not be gauged with cement. Portland cement or rapid-hardening Portland cement may be used.

The following proportions may not be exceeded :—

- one volume of cement to not less than one nor more than five volumes of lime; and
- 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 by volume			Max. uniform comp. stress in lbs./sq. in. corresponding to units whose crushing strength in lbs./sq. in. is:					
Cement	Lime	Sand	400	1000	1500	3000	5000	10000
1	0- $\frac{1}{2}$	3	40	100	150	210	360	660
1	0- $\frac{1}{2}$	4	40	100	140	190	310	440
1	1	6	40	100	140	190	260	350
1	2	9	40	80	120	170	250	350
1	3	12	30	70	100	130	200	200
	1	3	30	60	80	100	100	100

Values for units whose crushing strengths are intermediate between those given in the table, may be obtained by linear interpolation.

These loads apply on fully hardened masonry. Crushing strengths of burnt Indian bricks vary from 600 lbs./sq. in. to 3000 lbs./sq. in. Values are given in Section 12.

(Based on British Standard Code of Practice : CP 111.)

#### 4. MASONRY ARCH DESIGN

(Also see under "Bridges")

**Choice of Type of Arch.** Where abutments are of ample 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 bigger arches, make the rise of the arch at the crown an inch in height for every foot of span. Rise of an arch between  $\frac{1}{4}$  to  $\frac{1}{2}$  of the span is considered most economical for buildings.

**Fixing Thickness of Arch Rings.** There are many empirical formulae for fixing the thickness of an arch ring. For brick arches:

$$t = \sqrt{0.20 \times R} \text{—for single semi-circular or oval arches,}$$

$$t = \sqrt{0.25 \times R} \text{—for arches in series.}$$

Trautwine's formula : for cut stone work

$$t = \frac{\sqrt{R + \frac{1}{2}S}}{4} + 0.2 \text{ ft.}; \quad R = \frac{(\frac{1}{2}S)^2 + r^2}{2r}$$

Rankine's formula

$$t = 0.4\sqrt{S}$$

A good common rule for light load brick arches is to make half-brick thickness rings for each 5 ft. of span. For heavy loads the following thickness of arch rings may be taken for brickwork in cement mortar 1 : 4.

Span	Thickness of arch ring
Up to 5 ft.	9"
6 ft. to 14 ft.	13"
15 ft. to 25 ft.	18"
26 ft. to 35 ft.	22"
36 ft. to 50 ft.	27"

As a general rule, a 60° arch should be avoided as it has a tendency to crack at haunches and is also more expensive to build. Its only advantage over other types of arches is that it has low rise.

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 20 ft., 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  $\frac{3}{4}$ th.

Data for Segmental and Semi-circular Arches :—

Type of arch	Thickness at crown	Radius R =	Rise r =
60°	$0.43\sqrt{S}$	S	$0.134 S$
75°	$0.40\sqrt{S}$	$0.821 S$	$0.170 S$
90°	$0.38\sqrt{S}$	$0.707 S$	$0.207 S$
120°	$0.36\sqrt{S}$	$0.577 S$	$0.288 S$
180°	$0.36\sqrt{S}$	$0.500 S$	$0.500 S$

t = thickness of the arch ring in ft.,  
R = radius of the arch at the crown in ft.,  
S = span in ft.,  
r = rise in ft.

### 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 25 ft.; 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  $4\frac{1}{2}$  ins. 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  $\frac{1}{4}$ -in. in thickness.

It is preferable to provide all arches of 6 ft. span and above with keys. For spans 6 ft. to 12 ft. there should be one key at the crown, and for spans above 12 ft., additional keys should be provided so that the distance between keys is not more than 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.

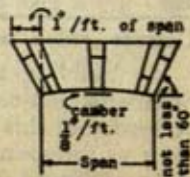
Arch-work should be carried up evenly from either abutment and as soon as the arch is complete the masonry should be built up to the height of the crown so as to load the haunches.

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 completely 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  $\frac{1}{8}$  in. per ft. of span; it should have angle of not less than 60 deg. at the skew-back which gives a projection of about 1 in. per ft. of span at the top (extrados) beyond the span.



**FLAT ARCH**



**Skew Arches**

$\phi$  = 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

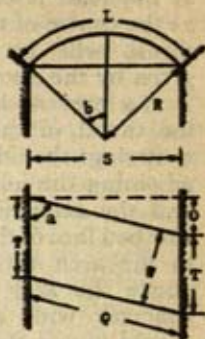
$$= S \operatorname{cosec}.a = \frac{T}{W} \times S$$

$T$  = length of impost =  $W \operatorname{cosec}.a$

$L$  = length of arc =  $.0349Rb$ .

For semi-circular arch :

$$L = 1.571S = \frac{S}{2} \times 90 \times .0349 = 90 \times R \times .0349.$$



**Jack Arches** are usually made for 4 ft. to 6 ft. spans,  $4\frac{1}{2}$  ins. thick. Rise is about  $\frac{1}{4}$  to  $\frac{1}{8}$  of the span. Where bigger arches are required thickness should be 9 ins. for spans from 8 ft. to 20 ft. and rise  $\frac{1}{4}$  th of span from 12 ft. to 20 ft.

**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. They are generally  $\frac{1}{2}$  in. to  $\frac{3}{4}$  in. dia. for small spans and 1 in. to  $1\frac{1}{2}$  in. dia. for big spans, and 4 to 5 ft. apart. Spacings of the tie rods should not exceed 20 times the width of the supporting beams. Tie-rods must be put in place and the nuts tightened up correctly before the centerings are fixed. On no account must the centerings be hung from them.

Tie-rods take the thrust of the arches which can be calculated from the following formula :

$$T = \frac{1.5 WL^2}{R}$$

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.

$T$  = hor. thrust of the arch in lbs. per lineal foot which has to be resisted by the tie-rods (hor. thrust is tensile stress.);

$W$  = live and dead load over the arch in lbs. per sq. ft.

If the load is concentrated at the centre of the arch the thrust will be twice that given by the above formula.

$L$  = span in feet;

$R$  = rise in inches.

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 ( $9'' \times 4\frac{1}{2}''$ ) 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  $1\frac{1}{2}$  ins. over the crown of the arch, which may be with fine lime concrete. Roof beams, tie-rods, and wall plates before being erected, should be painted with red lead paint two coats.

**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.

**Centerings for Common Arches.** Before turning arches exceeding 40 ft. span the middle half of the centre should be loaded with all the bricks to be used in that portion of the arch. The centres should be provided with an arrangement for lowering them to the extent of one to two inches (*i.e.*, "striking"); this arrangement may be hardwood wedges for small spans and sand boxes for moderate and large spans.

**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, elliptical 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 cased 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  $1\frac{1}{2}$  to 1 when not more than 20 ft. high, and  $1\frac{1}{2}$  or 2 to 1 when in greater height. (iii) In fine sands:  $2\frac{1}{2}$  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 soil and they also draw moisture from it which adds to the strength. Alternatively, pitching may be used.

**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 retaining 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

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When slopes are expressed as  $1\frac{1}{2}$  to 1, 2 to 1, the first figure represents the horizontal measurement, the second vertical measurement. Similarly is  $1\frac{1}{2} : 1$ , 2 : 1, or 1 : 4.



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 unaffected 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.

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°
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°

Average angle of repose for common designs may be taken as per table appended



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 lbs./sq. ft. which may be taken for very soft clay, 200 to 375; soft clay, 375 to 750; firm clay, 750 to 1000; fairly stiff clay, 1000; stiff clay, 1500 to 2000; very stiff clay, 2000 to 3000. Sands predominating with some clay, 400; cemented sand and gravel, 500; sand-gravel mixture cemented with clay, 1000.

**Coulomb's Wedge Theory.** In this theory it is assumed that there is a wedge having the wall as one side and a plane called the "plane 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  $\frac{1}{3}$  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 :—

$$P = \frac{wh^2}{2} \times \frac{\cos^2 \phi}{\left[ 1 + \sqrt{\frac{\sin \phi \cdot \sin \phi (\phi - \delta)}{\cos \delta}} \right]^2} = \frac{wh^2}{2} \times K_1$$

For a wall with a vertical back and a level backing,  $\delta = 0$ ,

$$\text{Hence } P = \frac{wh^2}{2} \times \frac{\cos^2 \phi}{(1 + \sin \phi)^2} = \frac{wh^2}{2} \left( \frac{1 - \sin \phi}{1 + \sin \phi} \right)$$

**Rankine's formula** for walls without surcharge:

$$P = \frac{wh^2}{2} \times \frac{1 - \sin \phi}{1 + \sin \phi}$$

$1 - \sin \phi$

The intensity of pressure at any depth is

$$wh \times \frac{1 - \sin \phi}{1 + \sin \phi}$$

The earth pressure acts horizontally at  $\frac{1}{3}$  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 previously under "Design of Wall Against Wind Pressure".

P = total lateral earth pressure per ft. length of wall due to the backing,

w = weight of backing per c. ft.,

h = height of backing,

$\phi$  = angle of repose of backing, — 33° common

$\delta$  = 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 30 lbs./c. ft. (Dry earth pressure is generally equal to  $\frac{1}{3}$  of water pressure.)

**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.

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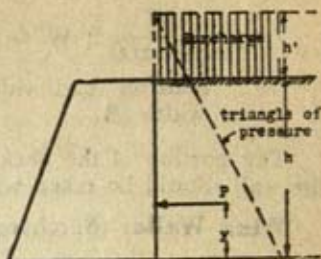
\*Some authorities consider the centre of pressure exerted by the backfill, when considered dry, located at  $0.42 h$  for a level fill, instead of  $\frac{h}{3}$

$$P = \frac{1}{2} w (h + h')^2 \times \frac{1 - \sin \phi}{1 + \sin \phi}$$

$$y = \frac{1}{3} (h + h')$$

$P$  = resultant pressure per ft. of wall length,

$h'$  = height of earth-fill equivalent to surcharge in ft. (superimposed load per sq. ft. divided by weight per c. ft. of fill.)



ABUTMENT

Another method is :

$$P = \frac{1}{2} w h (h + 2h') \times \frac{1 - \sin \phi}{1 + \sin \phi} ; \quad y = \frac{1}{3} h \times \frac{h + 3h'}{h + 2h'}$$

**Return Walls.** A return wall is a retaining wall built parallel to the centre line of a road to retain the embankment.

Road traffic is assumed to exert a uniform load of 220 lbs. per sq. ft. on the backing, which is equivalent to a surcharge of 2 ft., when considered acting at level with the wall top. (Which may be taken 3 ft. for heavy traffic. See under "Bridges".)

Live load due to road traffic or any isolated load is assumed to be dispersed through the backfill at an angle of  $45^\circ$ . In case the  $45^\circ$  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.

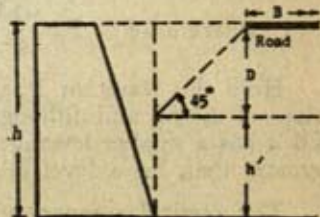
$$P_1 = \frac{1}{2} w h^2 \times \frac{1 - \sin \phi}{1 + \sin \phi}$$

$$y_1 = \frac{1}{3} h$$

$$P_2 = w_1 h' \times \frac{1 - \sin \phi}{1 + \sin \phi}$$

$$y_2 = \frac{1}{2} h'$$

Total pressure moment is  $P_1 y_1 + P_2 y_2$ .



RETURN WALL

$P_1$  = resultant pressure due to fill only,

$P_2$  = resultant pressure due to surcharge,



$w_1 = \frac{W}{(B+2D)}$  ;  $W$  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)

$$P = \frac{1}{2}wh(h+2h') \times \frac{1-\sin \phi}{1+\sin \phi}$$

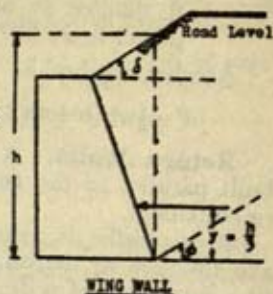
$$y = \frac{1}{3}h$$

$h'$  = equivalent level surcharge due to the sloping fill

$$= \frac{1}{3}h \cot \phi \tan \delta$$

$\phi$  = angle of fill,

$\delta$  = angle of surcharge.



### Surcharged Retaining wall with Rankine's Theory

For angle of surcharge  $\delta$  between  $0^\circ$  and  $\phi^\circ$

$$P = \frac{wh^2}{2} \cos \delta \frac{\cos \delta - \sqrt{\cos^2 \delta - \cos^2 \phi}}{\cos \delta + \sqrt{\cos^2 \delta - \cos^2 \phi}} = \frac{wh^2}{2} \times K_2$$

$$\text{where } \delta = \phi \quad P = \frac{wh^2}{2} \cos \phi$$

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  $K_2$ 

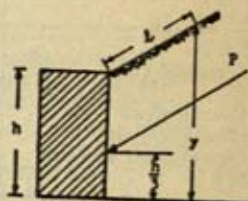
Angle of Surcharge	Angle of Repose						
	15°	20°	25°	30°	35°	40°	45°
10°	0.70	0.57	0.46	0.37	0.29	0.23	0.17
20°	—	—	0.54	0.42	0.32	0.25	0.19
25°			—	0.49	0.37	0.27	0.21
30°				—	0.44	0.32	0.23
35°					—	0.39	0.27
40°						—	0.34

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  $\frac{h}{3}$



Slope	1 : 1	1½ : 1	2 : 1	3 : 1	4 : 1
Value of $y$	1.71h	1.55h	1.45h	1.31h	1.24h

**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 ft. length

of wall, acting horizontally at  $\frac{1}{3}d$  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 previously described 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  $\times$  weight of water) — weight of water.

Material	Average angle of repose under water	Weight under water, lbs./c.ft.
Sand .. ..	27°	60
Sand and clay ..	18°	65
Clay .. ..	16°	80
Gravel, clean ..	27°	60
Gravel and clay	18°	65
Gravel, sand and clay	18°	65
Soil .. ..	16°	70

Retaining walls with backfills of earth charged with water may be designed for a total pressure of 75 lbs./c.ft. and other walls where precautions are necessary against cracking, such as water reservoirs or dams, may be designed for a total pressure of 98 lbs./c.ft.

**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 superimposed 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 10 ft. 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.

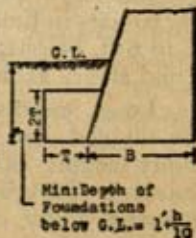
If the resistance to sliding is too little, base width of the wall will have to be increased or preferably, the base should be sloped down towards the backfill. Foundations will be normal to the face in the case of walls with battered fronts and in the case of vertical fronts the foundations can be sloped 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  $h/10 + 1$  ft. below ground level. The projection of any footing course should not exceed half the depth of the course. 3-in. 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 $T=0$	pressure	$p$ becomes	$=P$
For $T=B/6$	"	"	$=.62p$
For $T=B/5$	"	"	$=.56p$
For $T=B/4$	"	"	$=.48p$
For $T=B/3$	"	"	$=.37p$



*Example:*

$$P = \frac{1}{2} wh (h + 2h') \frac{1 - \sin \phi}{1 + \sin \phi}$$

Effective height for earth pressure =  $H'$

Equivalent level surcharge due to the sloping fill

$$= \frac{1}{3} H \cot \phi \tan \delta$$

$H$  for  $1\frac{1}{2} : 1$  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  $\frac{1}{3} H \cot \phi \tan \delta$  will be  $\frac{1}{3} H'$

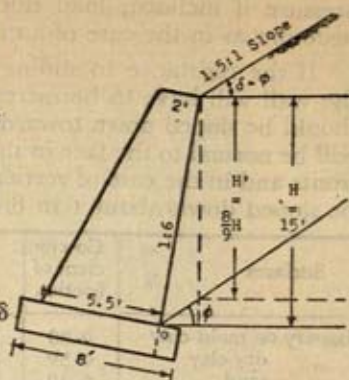
$$= \text{level surcharge} = 4.44'$$

$$\text{Then } P = \frac{1}{2} w H' (H' + \frac{2}{3} H') \frac{1 - \sin \phi}{1 + \sin \phi}$$

$$y = \frac{1}{3} H' (H - H') = 6.1 \text{ ft.}$$

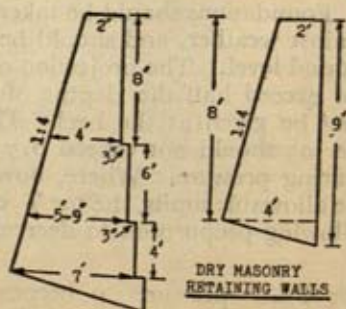
Earth Pressure =  $P_y$

Take moments about  $o$  for test at ground level.



## Dry Retaining Walls

Take top width about 2 ft.; 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. provide 12-in. courses in lime at every 2 to 6 ft. height; through bond stones at intervals of 5 ft.

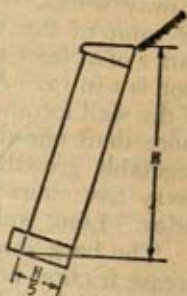


Long lengths of dry rubble retaining walls should be divided into panels separated from one another by short lengths of walls 5 to 7 ft. long built in mortar at intervals of say, 20 to 30 ft., in order to confine subsequent damage, if any, only to the panels affected.

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.: ver.) with top 2 ft. 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 buttresses 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 10 ft. for low walls to 20 ft. 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  $\frac{1}{4}$ th to  $\frac{1}{3}$ th 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 2 ft. for stone masonry and 1 ft.-10½ ins. for brick walls, and which should be 3 ft. 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.

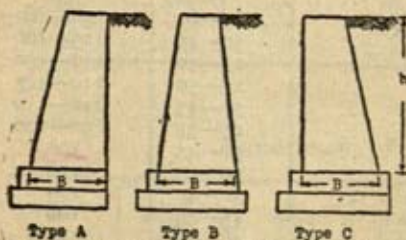
The back should be left rough or built in steps to increase friction between the wall and the backing. Backfill should be deposited in 4-in. to 6-in. layers with moderate compaction sloping downward from the wall to reduce lateral pressure, after the wall has attained sufficient strength. Backfilling 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.

**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 6 to 10 ft. apart, should lead through the base. With more retentive soils, a drain at least 9 or 12 ins. 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 2 to 3 ins. square, or 3-in. pipes may be used at 6 to 7 ft.

intervals (in staggered positions) vertically and horizontally, the lowest being 1 ft. 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.

**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.

**Design Data for Retaining Walls for Level Fills**  
(For surcharged fills increase the base width by 20%.)



K: is the ratio of the weight of wall masonry and weight of the soil material per c. ft.,  
p: is the max. toe pressure in lbs. per sq. ft. 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				Min: width for a rectangular wall that will just stand the earth pressure
	Type A	Type B	Type C	Rectangular	
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=	120 h	160 h	200 h	200 h	

$K=1.25$					
20°	.61	.62	.73	.65	.36
27°	.51	.52	.62	.55	.31
33°	.43	.44	.53	.47	.28
37°	.41	.42	.50	.45	.25
common					
p=	155 h	190 h	230 h	250 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=	190 h	225 h	260 h	300 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 to other reasons.

Material	Angle of Repose Deg.	Wt. in lbs. per c. ft. of soil
Ashes .. ..	40	50
Clay, wet .. ..	15-20	140-160
„ damp, well drained .. ..	30-45	125-160
„ dry .. ..	25-30	110-130
Sand, wet .. ..	15-30	110-125
„ moist .. ..	30-45	110-110
„ dry .. ..	25-35	90-100
Earth, loose .. ..	30-45	100
„ rammed .. ..	50-65	120
„ dry .. ..	30-40	110
„ moist .. ..	45-49	100
Sand and clay, wet .. ..	18-20	125
Gravel .. ..	40-45	90-100
„ wet .. ..	27	125
Gravel and clay, wet .. ..	18	127
Gravel, sand and clay wet .. ..	19	130
Gravel and sand .. ..	25-30	100-110
Shingle .. ..	38-40	90
Shingle and earth, moist .. ..	40	140
Vegetable soil, dry .. ..	30	90-100
„ „ moist .. ..	45-50	100-110
„ „ wet .. ..	15-17	110-120
Peat .. ..	14-45	30
Silt, wet .. ..	10-20	110
„ dry .. ..	20	110
Rubble stone .. ..	45	110

\*The above figures are approximate and are variable.



## Functions of Soil Angles

$\phi$ (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)	$\frac{1 - \sin \phi}{1 + \sin \phi}$	$\left(\frac{1 - \sin \phi}{1 + \sin \phi}\right)^2$
$\frac{1}{4}$ to 1	75°—58'	.970	1.0131	.002	..
$\frac{1}{3}$ to 1	63°—26'	.895	1.118	.055	..
$\frac{1}{2}$ to 1	53°—8'	.800	1.250	.111	..
1 to 1	45°—0'	.707	1.414	.171	.030
$1\frac{1}{4}$ to 1	38°—40'	.624	1.600	.232	.054
1.33 to 1	36°—53'	.600	1.608	.250	.062
$1\frac{1}{2}$ to 1	33°—42'	.554	1.802	.285	.080
$1\frac{3}{4}$ to 1	29°—44'	.496	2.016	.337	.113
2 to 1	26°—34'	.447	2.236	.382	.144
$2\frac{1}{2}$ to 1	21°—50'	.372	2.689	.458	.211
3 to 1	18°—26'	.316	3.162	.520	.289
$3\frac{1}{2}$ 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

## 6. PARTITION WALLS

**Brick Walls.** Where it is desired to keep the thickness of a partition wall 3 ins. or  $4\frac{1}{2}$  ins. it should be reinforced with hoop iron 1 in. wide and 18 gauge, in courses not more than 12 ins. apart and continued for 9 ins. into the main wall on which the partition wall abuts, and folded over. If the partition wall exceeds 20 ft. in length or 15 ft. in height, the hoop iron should be introduced at courses not more than 6 ins. 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 and can be in two strips, but it must be well embedded in the mortar).

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 6 ins. so as to form burrs on both sides of the strips. The punch hole may be  $\frac{1}{4}$  in. 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 3 ins. The joints in brickwork, where the hoop iron is to be laid, should be 1 in. thick to ensure a cover of at least half an inch 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 neces-

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.

### **Lathing and Plaster Partitions**

*Expanded-Metal Lathing* Expanded metal may be of size: No. 1,  $\frac{3}{8}$  in. short-way of mesh and 24 gauge of metal, weight approximately  $3\frac{1}{2}$  lbs. per sq. yd., or 22 gauge  $\frac{3}{8}$  in. mesh. Lathing is stiffened by  $\frac{3}{8}$  in. diameter round iron rods spaced one foot apart vertically, which are tied to lathing at intervals of not more than 4 ins. 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  $1\frac{1}{2}$  parts of clean sharp coarse sand, 1 part of slaked lime and  $\frac{1}{2}$  part of cement. The mortar should be thoroughly mixed with chopped hemp in the proportion of not less than 1 lb. of hemp to 3 c. ft. 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 6 ins.



by 5 ins. are fixed at central distances of 5 ft. Horizontal pieces 6 ins. by 2 ins. are fixed 3 ft. 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.

Wooden 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 100 to 150 ft. 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 75 to 100 ft. In the case of local roads the above distances may be reduced to 50 ft. and 35 ft. respectively.

Trees also reduce sound to some extent.

**Walls.** Sound transmission through partitions can be reduced by the following methods :—

- (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 2 ins. of concrete is poured in-situ upon a resilient 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 coverings such as, carpets, cork or linoleum, if the materials are reasonably thick.

To prevent echoes the back wall behind the audience should be highly absorbent 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 w. c. 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  $\frac{1}{2}$  in. to 1 in. 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 R. C. 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 landings 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 23 inches.

$$\text{Pitch} = \frac{\text{Rise}}{\text{Tread}} \qquad \begin{array}{l} 2 \text{ Rise} + \text{Tread} = 23 \text{ inches} \\ \text{or Rise} \times \text{Tread} = 66 \text{ inches} \end{array}$$

Standard sizes :

Rise = 7 ins. }	for ordinary residential
Tread = 9 ins. }	buildings,
Rise = 6 ins. }	for public buildings with
Tread = 11 ins. }	wide stairs.

Pitch should not be more than 42 degrees.

No staircase in a residential building should have a rise of more than 9 ins. and a tread of less than 9 ins. In the case of public buildings (including warehouse and industrial) a rise of not more than 7 ins. and a tread of not less than  $10\frac{1}{2}$  ins. 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 at 18 ins. from the inside or the smaller end of the tread.

Winders should not be used if they can be avoided but if they are necessary they should be at least 9 ins. in width at about 16 ins. 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 8 ft. or 12 steps (max. 15) without landings or turns. Greater heights should only be reached by flat pitches such as 11 ins.  $\times$  6 ins., 12 ins.  $\times$   $5\frac{1}{2}$  ins.

The min. clear head-room should be 7 ft. 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**

Min : width for a residential building	..	2 ft.-6 ins.
"      "      public building	..	4 ft.-6 ins.

In a public building if the number of users is more than 20 and less than 100, the min. width should be 6 ft., and for number of users above 100, it should be 7 ft.=6 ins. The width should not, however, be less than the width of the stairs.

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 staircase its min. width should be equal to the widest of such staircases plus half of the total width of the remaining staircases.

**Width of Staircases**

Min : clear width of a staircase for a single family residential house is 2 ft.-3 ins. which should be 3 ft. 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 min : width :—

Number of users up to 10	..	..	..	3 ft.-6 ins.
"      "      from 11 to 20	..			4 ft.
"      "      from 21 to 100				4 ft.-6 ins.

Stairs should be at least 3 ft.-6 ins. wide to allow two persons to pass easily.

For a public, warehouse, or an industrial building :—

Number of users up to 200	..	..	5 ft.
"      "      from 200 to 350	..		6 ft.

Increase by 1 in. for every additional 15 persons until a max: of 9 ft. is reached.

A single staircase of the width mentioned above may be replaced by two staircases each of a width at least equal to  $\frac{2}{3}$ rd 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  $1\frac{1}{2}$  ins. 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 9 ins. Steps supported at both ends should rest at least 6 ins. on the walls at either end, which may be  $4\frac{1}{2}$  ins. min. for 3 ft. wide stairs and should be increased up to 9 ins. 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 1 ft.-6 ins. for access to a terrace.

### **Handrails, Parapets and Balustrades**

The height of railings is usually 2 ft.-6 ins. to 3 ft. If the railing is composed of balustrades, the spacing between them should not be more than 3 ins.

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 200 lbs./ft. run acting at handrail level.

### **Dowels, Cramps, Joggles, etc.**

Cramps may be of copper or lead, 6 ins. to 2 ins. long,  $\frac{5}{8}$  in. to 1 in. thick and 1 in. to 2 ins. wide, having each end turned at right angles. 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 or 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 galv. wire of  $\frac{1}{2}$  in. mesh and No. 20 S.W.G., fixed to wooden battens or rafters at not more than 18 ins.



centres by means of  $1\frac{1}{2}$  ins. wire nails, the nails to be spaced at intervals not exceeding 6 ins. Where netting passes over timber or iron framing, a space of  $\frac{1}{2}$  in. 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 to be of  $\frac{3}{8}$  in. mesh (shortway), No. 22 gauge, weighing about  $4\frac{1}{2}$  lbs./sq. yd.

**Cloth ceiling** is fixed on frames not exceeding 5 ft.  $\times$  5 ft. in size with  $\frac{3}{8}$  in.  $\times$   $1\frac{1}{2}$  ins. wooden fillets. The cloth should be first damped, stretched over frames and fastened with tacks on the outside. A coat 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 one-third of an inch to the foot. 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 25 ft. below the top should be at least 9 ins. and should be increased to one-half brick for every additional 20 ft. or part thereof measured downwards. If the inside diameter at the top exceeds 4 ft.-6 ins., the top length should be  $1\frac{1}{2}$  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  $\frac{3}{8}$  in.  $\times$   $2\frac{1}{2}$  ins., built into the brickwork 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  $4\frac{1}{2}$  ins. thick and separated from the masonry enclosing the shaft by a cavity at least 1 in. (prefer about 3 ins.) and the cavity should be covered at the top with corbelled brickwork.

Wind pressure for calculating stresses in chimneys and tall structures has been explained elsewhere.

### **Height of Chimneys**

No exact formula for the height of a chimney can be given which will be satisfactory for all types. Height is generally based on the horse power of the boiler or boilers served and the amount of coal burnt.

Chemical works are usually required to have their chimneys at least 250 ft. high to ensure that fumes are discharged well above the town.

Power station chimneys should have a height not less than  $2\frac{1}{2}$  times that of the highest point of the station roof or adjacent buildings.

The *area of chimneys* is proportioned to the amount of coal consumed and differ slightly with the quality of the coal. In England the common practice is to have the chimney area one-eighth of the grate area. Coal burnt per hour per sq. foot of grate is about 20 lbs. average steam coal.

### **Domestic Chimneys and Fireplaces**

Fireplaces in a house of more than one storey usually stand one over the other, the flues from the lower rooms being carried to one side or the other of the fire places above. The depth of ordinary grates is not less than 14 inches and not more than 18 inches. The width of fire-place openings vary from 18 inches to 27 inches for small rooms to 42 inches and upwards for large rooms. The height of a fireplace should be 3 ft.-3 ins. for ordinary grates.

Floors beneath and around every fireplace should be of concrete or similar fire-proof material which should



project 18-ins. in front of the jambs and extend 6 ins. on each side of the fireplace opening. The jambs of a fireplace opening should be at least 9 ins. in width, and the back of the chimney opening in a party wall should be 9 ins. thick up to 12 ins. above the top of the opening. Where the flues in a party wall are not back to back the required 9 ins. of solid wall at back of the fireplace should be carried up to the floor of the room above. In an external or internal wall the back of the opening and all sides of the flues should be at least  $4\frac{1}{2}$  ins. thick. Whenever possible flues should be placed on an inside wall as with flue on an outside wall most of the heat gained from the flue is lost.

No timber should be placed nearer than 9 ins. to the inside of any flue or chimney opening and within 15 ins. from the upper surface of the hearth.

*Size of Flues:—*

- |           |                                      |
|-----------|--------------------------------------|
| 9" × 9"   | — for small grates                   |
| 14" × 9"  | } — for ordinary domestic fireplaces |
| 16" × 9"  |                                      |
| 14" × 14" | — for large kitchen ranges.          |

A flue should change its direction by gradual curves and inclinations at an angle of not less than  $45^\circ$ , otherwise soot will accumulate. Every fireplace should have a separate flue, but all flues may be grouped on the top in a single stack. Render the inside of a flue as it is built up, with 1 lime and 3 cowdung or with lime and hair mortar. Lime mortar may be with 1 lime and 3 surkhi, but preferably add cowdung. Sometimes, 1 cement, 3 sand and 1 part cowdung are also used. This is called *Pargetting*. Big size flues are lined with fire-bricks  $4\frac{1}{2}$  ins. thick for a height of at least 10 ft.

*Shafts and Stacks*

Chimney stacks or smoke flues should be carried up at least 3 ft. above flat roofs and at level of ridge in pent roofs. Shafts should be  $4\frac{1}{2}$  ins. thick minimum.

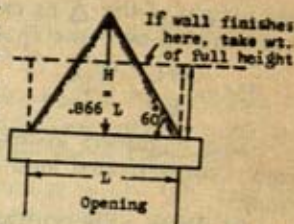
**Steel Chimneys.** Are usually cylindrical in shape with a wide curved flare at the bottom. A heavy base plate is provided to which the chimney is riveted and



the plate is secured to the foundation by holding-down bolts. A steel chimney 3 ft. dia. and 70 ft. high requires a 3/16 in. thick sheet.

## 11. 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  $BM = WL/6$ .



$W = 0.433 L^2 \times \text{thickness of wall} \times \text{wt. of one c. ft. of masonry.}$

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  $0.433L^2$ . 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  $1\frac{1}{2}$  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 the supporting beam. The lintel should be kept propped during bricklaying till the mortar has fully hardened.

*Typical cases :*

(a) If the height of the wall above the lintel is less than  $H$  ( $0.866L$ ), 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  $\Delta$ , take weight of the  $\Delta$  as explained before plus the load concentrated over the span.

$$BM = \frac{WL}{6} + \frac{W_1L}{4} \quad W_1 \text{ is the concentrated load.}$$

Some engineers assume the load distribution at  $60^\circ$  from a concentrated load occurring inside or near the load triangle.

(d) With a concentrated load above the  $\Delta$ , add the weight from the concentrated load to the weight of the  $\Delta$  and consider the whole as uniformly distributed as before unless the concentrated load is more than  $L$  feet above the apex of the  $\Delta$  in which case it may be neglected.

Uniformly distributed floor loads above the equilateral triangle are disregarded, 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

Depth = 1 in. for each foot of span + 1 in.

For 5 ft. span it will be 6 ins.

(Also see under "Stones")

### R. C. Lintels

$D = 0.318 L^{\frac{3}{4}}$   $D$  = effective depth of lintel in inches,  
 $L$  = effective span in feet.

For small spans and ordinary loads, take 6 ins. depth for  $\frac{1}{4}$  ft. span or under and add 1 in. for each foot of span.

### Reinforcement for R. C. Lintels:

For each  $4\frac{1}{2}$  ins. thickness of wall carried the reinforcement may be as follow:—

One  $\frac{3}{8}$ " dia. bar for spans under  $\frac{1}{4}$  ft.

One  $\frac{1}{2}$ " dia. bar for spans  $\frac{1}{4}$  to 7 ft.

One  $\frac{5}{8}$ " dia. bar for spans 7 to 10 ft.



All bars should be hooked and alternate (central) bars bent up. It is advisable to provide two bars of  $\frac{1}{4}$  in. or  $\frac{3}{8}$  in. diameter at the top to assist in assembling the reinforcement cage and to bind all with  $\frac{1}{4}$  in. dia. stirrups at 6 to 12 ins. centres. Pre-cast lintels may be used up to 4 ft. 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").

Bearing of lintels over walls should not be less than the overall depth of the lintel with minimum of 6 ins. for brick or coursed rubble walls and 9 ins. for random rubble walls.

### **Lintels and Beams over Foundation Piles**

The above simple design methods of supporting a triangular weight of brickwork is considered uneconomical for low superimposed loads. The following method is recommended by the Building Research Station, Watford, England :—

When there are openings near the support, take Bending Moment of  $WL/50$ , and  $WL/100$  when there are no openings, or openings are at mid-span.  $W$  is the weight of the rectangle of brickwork immediately above the beam plus the superimposed load on the span. If there are live loads at beam the same should be taken independently in addition to the composite action of the wall. The ratio of depth of beam to span should lie between  $1/15$  and  $1/20$ .

For deep walls the reinforcement may be considered effective up to  $1/10$  span from the lower edge; for a shallow panel up to  $\frac{1}{4}$  span. The application of the above design rules should be restricted to where the depth is not less than  $0.6 \times \text{span}$ . It is desirable to provide some top steel in the beam over the supports and also some nominal shear reinforcement near the supports where openings occur near the supports.



The steel reinforcement may be designed with a "moment arm" of  $\frac{3}{4} \times$  depth, with a limit of 0.7 times the span of the wall.

These design rules are not applicable to rolled steel beams.

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.

## 12. 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  $22\frac{1}{2}$  deg. on the East-West line towards East-South.

#### *Hot humid areas :*

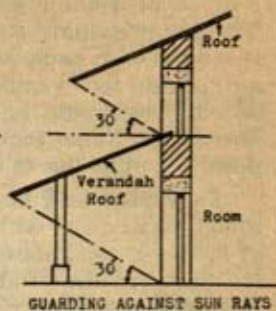
West coast regions—Orient along SE and NW, facing SW.

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.



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 w. cs. 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  $63\frac{1}{2}^\circ$  rule.

**Height of Living Rooms.** Beyond a certain point increasing the height of a room in preference to floor area would not be of much use as regards ventilation. Maximum useful height considered is 10 to 12 ft., but a height of less than 12 or 14 ft. does not make a good residential room. Height of a cellar or basement, mezzanine floor, store, gallery or a verandah shall in no part be less than 7 ft.-6 ins. Lofty rooms are cooler.

### Window Area for Dwellings:

A window or windows (clear of the sash frames) should have openings into open air space of not less than  $1/10$ th, or an aggregate opening of doors and windows of not less than  $1/7$ th, of the floor area of the room; or a window opening of 2.5 sq. ft. per head in living rooms, whichever is more. Galton prescribes: 1 sq. ft. of window space per 100 to 125 c. ft. of room content in dwelling houses, and



1 sq. ft. of window space per 50 to 55 c. ft. in hospitals. Ventilators should be provided at the rate of  $1\frac{1}{2}$  sq. ft. for every 500 c. ft. capacity of such rooms. The min: area of an opening for ventilation should be 3 sq. ft. The sills of windows should be about 2 ft.-6 ins. above the room floor level. The ventilators should be fixed as high as possible under the ceilings.

**Factories and Warehouse Buildings.** Every room should have doors with clear opening not less than  $1/15$ th of the floor area, abutting on open air space, of width not less than  $\frac{1}{3}$ rd the height of the part of the building abutting such open space.

### Temperature Variations in Structures

In India the max. daily range of shade temperature is about  $30^{\circ}$  F., and the yearly range about  $80^{\circ}$  F., and in the sun, about  $80^{\circ}$  F. and  $130^{\circ}$  F. respectively. The normal average temperatures may be taken as  $70^{\circ}$  F. in the plains and  $60^{\circ}$  F. in hill stations.

Expansion and contraction due to temperature changes may be provided for in various structures as follows:

Steel exposed to the sun	..	$\pm \frac{1}{2}$ " in 100 ft.
Steel shaded from the sun	..	$\pm \frac{5}{16}$ " "
Masonry and concrete exposed to the sun		$\pm \frac{5}{16}$ " "
Masonry and concrete shaded from the sun		$\pm \frac{3}{16}$ " "
Structures in contact with water	..	$\pm \frac{3}{16}$ " "

In hill and coast stations the variations in the temperature are about  $\frac{1}{3}$ th of the above.

### 13. PROTECTION AGAINST WHITE-ANTS

Damp-proof courses or cement concrete floors give good protection against white-ants. The following methods also offer protection :

(i) A solution of 1 lb. of copper sulphate in 4 gallons of water in the mortar. This solution also protects wood, takes two days steeping per inch of wood.

(ii) A layer of about 3 ins. washed sand under floors.

(iii) The filling under the floor is made up of sand and cinders over which a sodium arsenate solution is spread.

(iv) Yellow arsenic in mortars in the proportions of :



Concrete, 4 lbs.; masonry and plaster,  $\frac{1}{2}$  lb. per 100 c. ft.

(v) 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 10 ft. all round shall be excavated to a depth of 6 ins. 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.”

*(Arsenic being a poison should not be used on surface works.)*

#### 14. PLASTERING

**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; 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  $\frac{1}{2}$  in. 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 back ground. The thickness of the first coat should be just sufficient to fill up all unevennesses in the surface. No single coat should exceed  $\frac{1}{4}$  in. 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 and shrink properly before subsequent coats are applied. 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 100 mesh B. S. sieve or more than 20 per cent through a 50 mesh 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.

**Lime plaster.** The lime used for plastering ranges from fat lime to strong hydraulic lime. Fat lime is most commonly used on account of its yield of lime putty and its ease of application. Hydrated lime is generally preferred to the lime which has to be slaked by hand as the hydrated lime can be used at once 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 until it is of proper thickness. An equal volume of fine



sand is then added. Lime mortars should preferably be gauged with cement. 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 such a surface. Proportions of mortars have been given before 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 before in this Section 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 4 ins. to 6 ins. square or wooden screeds 3 inches wide and of the thickness of the plaster may be fixed vertically about 6 to 10 ft. apart to act as gauges. Trowels for plastering have face measuring about 10"  $\times$  4½". Wooden trowels produce a sandy granular surface.

**Defects in Plasterwork.** (1) *Cracks* are chiefly due to : (i) Structural defects in building and discontinuity of surface; (ii) Plastering 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  $\frac{1}{4}$ -in. to  $\frac{1}{2}$ -in. shingle or crushed stone, which is thrown on to the wall by means of a scoop or plasterer's trowel.

(b) *Pebble-dash.* A  $\frac{3}{8}$  in. coat of 1 part cement, 1 part lime and 5 parts sand upon which, while it is still soft, is thrown  $\frac{1}{4}$ -in. to  $\frac{1}{2}$ -in. shingle.

(c) *Ornamental finishes.* A mix of approximately 1 part cement,  $1\frac{1}{2}$  parts lime and 6 parts sand which after application is figured by the use of combs, trowels or special tools.

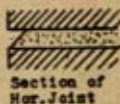
**Plastering on Lathing.** Lathing forms a convenient base in some forms of construction for plastering on walls and ceilings. Metal lathing is most commonly used which is fixed to timber supports by galvanized wire nails or staples at short distances. It is also often used to bridge the junction of two dissimilar backgrounds, or to provide a suitable key for plastering over a wooden beam. Metal lathing may be expanded metal, or woven wire, etc., which should weigh not less than 3 lbs. per sq. yd. except when used to provide a key. 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. 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 previously 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 3 ft. to 4 ft. long, and 1 in. by  $\frac{3}{4}$  in. to  $\frac{3}{4}$  in. in section. Wooden laths should be thoroughly wetted before plaster is applied.

## 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.



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. Also see under "Cement/Lime Mortars" described before 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.

### 15. WHITE-WASHING & COLOUR-WASHING

The white-wash is made from pure fat lime (white stone) or shell lime. The lime should be brought to the work in an unslaked condition. After slaking it should be allowed to remain in a tank of water for two days and then stirred up with a pole until it attains the consistency of thin cream. Gum or rice water is added where necessary (2 ozs. of gum to 1 c. ft. of lime). 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. Lime putty is used to make good all holes and irregularities of surface or minor repairs, which should be let to dry before white-washing. All greasy spots should be 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.

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. If about  $14\frac{1}{2}$  lbs. of salt dissolved in hot water is added to 1 cwt. of lime and  $1\frac{1}{2}$  lbs. of glue, the white-wash will not easily rub-off.

**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  $\frac{1}{2}$  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.



### Colour-Washing

*Buff Colour* : Slake 4 lbs. lime with 2 gall. of water. Dissolve 10 ozs. brown earth in  $\frac{1}{4}$  gall. water and mix. Boil 2 ozs. gum in  $\frac{1}{4}$  gall. of water. Strain all the above through a cloth and mix.

*Green Colour* : Lime as for above. Boil 7 lbs. fresh mango bark in  $\frac{1}{4}$  gall. of water for 2 minutes. Boil 2 lbs. tootya (blue stone or vitriol) in  $\frac{1}{4}$  gall. of water. Mix. as above.

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 lb. of zinc sulphate in one gallon of water and then allowed to dry; before distempering it should be wiped with clean cloth to remove any efflorescence. No distemper need be done till after 12 months of the walls having been plastered or white-washed. 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. To get good results it is necessary to apply a priming coat (as recommended by the makers). It often happens that fresh coats of distemper pull off the old coats as 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.

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 result. 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. (Also see under "Painting").

## 16. STABILIZED SOIL FOR BUILDING CONSTRUCTION

(Also see "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 needed. 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.

(ii) 4 per cent. lime ;

(ii)  $1\frac{1}{2}$  per cent. cement +  $2\frac{1}{2}$  per cent. lime ;

(iii) 0.5 per cent. (about 1 gall. to 20 c. ft. 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 kneeding 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.

(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).

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%
Liquid limit	Not more than 25%
Plasticity Index	Not less than 8.5% and not more than 10.5%.
Total solids	Not more than 0.5%
Sodium sulphate	Not more than 0.10%.

**Tests.** The crushing strength of blocks 2.5 inches in diameter, made out of the cement-soil mixture actually used, shall not be less than 400 lbs. per sq. in. 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° 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 12 lbs. per sq. in. when two blocks, as for compression strength, are joined together with the plaster.

Plasticity Index of the soil is determined roughly by the "syringe test," which consists in extruding the wet soil from  $\frac{1}{4}$  in. diameter holes and judging the plasticity index from the state of surface on the filaments of soil extruded.

Freshly dug soil is so pulverised that not less than 85% of it possess through 5/16 in. 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  $\frac{3}{4}$  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* should serve the purpose.

**Walls in superstructure.** 2.5 per cent cement-soil mixture at optimum moisture, rammed *in situ* between shutterings, and 12 inches in thickness.

**Roofing.** The conventional type of roofing used in the area. Cement concrete bed plates placed over grooves made on the wall.

**Parapet.** Cement soil, with burnt bricks in cement coping, projecting slightly beyond the wall on each side.

**Plaster.** On the outside walls with 1:5 cement: sand,  $\frac{1}{2}$  in. thick over a coat of cement wash.

The cement-soil mixture shall be poured into the shutterings in layers of 3 ins. at a time. Compaction shall be done by means of iron rammers 16 lbs. in weight. Vertical joints shall be provided not more than 6 ft. apart and horizontal joints 3 to  $4\frac{1}{2}$  ft. apart. (Joints are made with the same principles as for R.C.C. 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 9 ins. wide and at least 2 ins. thick. The boarding shall be supported by vertical stiffeners at least 4 in.  $\times$  4 in. if wooden, or  $1\frac{1}{2}$  in.  $\times$   $1\frac{1}{2}$  in.  $\times$   $\frac{1}{4}$  in. if of angle iron, spaced not more than  $3\frac{1}{2}$  ft. apart. These stiffeners shall be held in position by means of spacer bolts  $\frac{5}{8}$  in. 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 11 ft. each and in case of angles and tees of wall corners, each leg shall be about 8 ft. 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 unevenness 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  $1\frac{1}{2}$  in. gauge, compacted 3 inches thick. (b) Soil-cement mixture containing 5 per cent cement compacted at optimum moisture with hand rammers in 7 ft.  $\times$  5 ft. slabs 3 inches thick. After the slabs are cured for a week with a frequent sprinkling of water, 1:3 cement-sand (fine) plaster is applied  $\frac{1}{4}$  in. thick in 2 ft.  $\times$  2 ft. 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, Punjab.)

## 17. 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 4 inches lime concrete over 4 inches 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 6 ft. and with parallel sides, as well as a 10 inch 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  $\frac{1}{4}$  in. thick, and shall be flush pointed after being raked out 1 inch 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 half inch thick bed of mortar and the joints shall not exceed  $\frac{1}{4}$  inch 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  $\frac{3}{4}$  in. thick: (b)  $\frac{1}{2}$  in. cement plaster 1:3; (c) Cement pointing 1:3.

**Flagged Flooring.** The slabs may be of unequal sizes but shall not be less than 14 inches in width or greater than  $2\frac{1}{2}$  ft. in length, and not less than  $1\frac{1}{2}$  inches 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  $\frac{1}{8}$ th inch in width and must be struck off flush with a trowel while laying the flags. Where pointing is specified, joints should not be less than  $\frac{1}{4}$  inch wide.

**Marble Flooring.** All marble slabs shall have a minimum thickness of  $\frac{3}{4}$ th of an inch and shall be bedded in  $\frac{3}{4}$  inch 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 emery

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".

### 18. LIGHTNING CONDUCTORS

(Based on the "Draft Code of Practice for the Protection of Buildings Against Lightning (1954)" issued by the Institution of Engineers (India), and the B.S. Code of Practice, CP 326. 101 (1948)).

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 1 foot min. 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  $\frac{1}{4}$  in. diameter copper or phosphor bronze solid rods (pointed at the top).

In case the height of a structure is 120 ft. or more and width at the top 5 ft. 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 100 ft. Salient points, even if less than 50 ft. 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.

The following sizes of conductors are recommended in the Draft Code:

Material	Above ground	Below ground
Round galv. iron wire	0 S.W.G. (0.324 in. dia.)	4/0 S.W.G. (0.400 in. dia.)
Galv. iron strip ..	$\frac{3}{4}$ in. $\times$ $\frac{1}{2}$ in.	$1\frac{1}{2}$ in. $\times$ $\frac{1}{2}$ in.
Round copper wire or copper clad steel wire ..	4 S.W.G. (0.232 in. dia.)	0 S.W.G. (0.324 in. dia.)
Stranded copper wire	0.075 sq. in. or 7/136 in. dia.	Not allowed
Copper strip ( 5.73 oz./ft. run)	$\frac{3}{4}$ in. $\times$ $\frac{1}{2}$ in.	$\frac{3}{4}$ in. $\times$ $\frac{1}{2}$ in.
Round aluminium wire	4/0 S.W.G. (0.400 in. dia.)	Not allowed
Aluminium strip	1 in. $\times$ $\frac{1}{2}$ in.	Not allowed

All iron roofs of a building, all metallic finials, chimneys, 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 4 ft. of the course of a conductor should be connected with it; any other heavy metals even beyond 4 ft. should also be connected. Where a ridge 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 7 ft. 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  $\frac{1}{4}$  in. 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, up turns and kinks, and bends in any direction should not be less than 1 ft. radius, as 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 10 ft. into the ground with charcoal or powdered coke filling extending 6 ins. around it on all sides. Coke should not be used with copper plates. No "earth" should be fixed within 10 ft. 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 2 to 3 ft.

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 the 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  $1\frac{1}{2}$  ins. galv. pipe at least 20 ins. 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 below.

**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  $\frac{1}{2}$  in. diameter and driven into the ground to a depth of at least 8 ft.

(ii) *Pipes:* 2-inch galv. iron pipe driven down to at least 8 ft. below ground level. The pipe should preferably go down into the sub-soil water for about 6 ft. The lower 6 ft. of the pipe should be perforated with  $\frac{1}{8}$  in. 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 18 ins. deep, in straight lines or parallel lines, or in radial formations. The distance between any pair of parallel strips should be not less than 6 ft. Strips have to be used where rock is encountered at a depth less than 8 ft. below ground level or 7 ft. below the lowest level of the excavation, thus prohibiting the use of vertically driven pipes.

**Plates:** The "earth" plates may be of size 3 ft.  $\times$  3 ft.  $\times$   $\frac{1}{8}$  in. thick if of copper, and copper plates must be used when the conductor is of copper. The plate should be  $\frac{1}{4}$  in. thick if of iron. Copper plates to be tinned on both sides and iron plates heavily galvanized. 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 1000 sq. ft. may have one conductor only. One additional conductor is required for every 3000 sq. ft. or part thereof in excess of 1000 sq. ft. or at least one down conductor per 100 ft. of perimeter, whichever is less.



### 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 1 ft. from the roofing material.

#### **Structures with Explosive or Inflammable Contents.**

Such structures should not have spires, or flagstuffs. Radio aerials should not be provided within 50 ft. 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 50 ft. 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 500 ft. All metal forming parts of the structure should be bonded or welded together and connected to the ring conductor at least at two points.

### 19. WATER STORAGE TANKS AND DAMS

#### **Design of a Circular Water Storage Tank Without Reinforcement**

$$t = \frac{hD}{92}$$

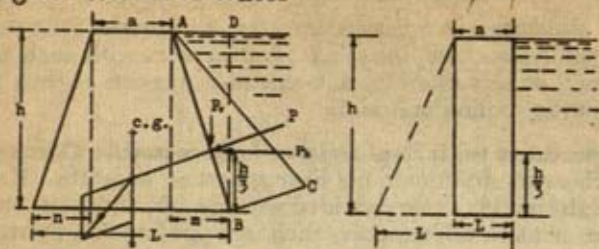
t—is the thickness in feet of a brick wall in cement,

h—is the depth of water for design in feet,

D—is the diameter of tank in feet.

This is on the assumption that all the tension due to water pressure will be taken by the brickwork.

For Reinforced Brickwork tanks, see under "Reinforced Concrete".

**Design of Walls for Water****Pressure Due To Fluids:**

Pressure at B =  $wBD$  lbs./sq. ft.

Total pressure on the wall  $AB = w \times BD \times AB/2$  lbs.

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 120 lbs./c.ft. 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:

$$L = \sqrt{h^2 - (a+n)^2(2-l) + n^2g} + \frac{n^2g}{4} - \frac{ng}{2}$$

Front face vertical ( $n=0$ ):  $L = \sqrt{h^2 - a^2(g-l)}$

Back vertical ( $m=0$ ):  $L = \sqrt{\frac{5}{4}a^2 + \frac{h^2}{g}} - \frac{a}{2}$

$$\text{or } L^2 + aL - a^2 - \frac{1}{g}h^2 = 0,$$

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  $\frac{1}{3}$  to  $\frac{1}{4}$  of the height.

Stress at the edge (toe) =

$$\frac{2V}{3\left(\frac{L}{2} - e\right)} = \frac{V}{L} \left( \frac{4}{3 - 6\frac{e}{L}} \right) \quad V \text{—is the total vertical load}$$

*Sliding*—To be safe against sliding the weight of dam  $\times$  co-efficient of friction should be greater than the total water pressure  $P_h$  (or any lateral force tending to slide the dam or wall).

Co-efficient of friction for sliding are given at page 7/45

Total pressure per unit length is equal to area of the trapezoid

S T U V

$$R = \frac{D}{2} (2H + D) w; \quad d = \frac{D}{3} \left( \frac{3H + D}{2H + D} \right)$$

$$SV = wH \times TU = (D + H)w$$

Pressure at V is SV

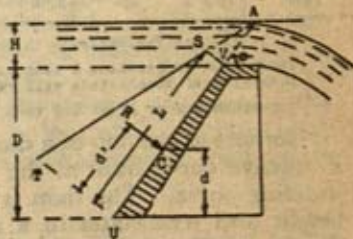
„ „ U is TU

All pressures act normal to the surface.

Total pressure P per unit length of dam is given by the area of the 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.



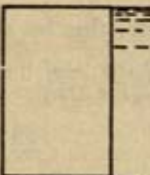



$$\text{If } H=0, \quad R = \frac{D^2 W}{2 \sin \phi} \quad \text{and} \quad d' = \frac{L}{3}$$

When there is possibility of upward water pressure (due to infiltration) at the base, this force should be subtracted from the weight (or mass) of the wall. (Also see 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

								For Weight of Masonry in lbs/cft.
Base	P	Base	P	Base	P	Base	P	
.96H	200H	.78H	175H	.74H	240H	.65H	160H	120
		.74H	190H	.70H	260H	.62H	175H	130
		.71H	205H	.67H	280H	.60H	190H	140
.95H	230H	.68H	215H	.64H	300H	.58H	200H	150

P is the max. Toe Pressure in lbs/sft. at the Base level.

Foundation projections & concrete will be below the base line.  
Extension of foundations will reduce the toe pressure.

Top width may be about 0.2 to 0.3 H, except for rectangular section.

Corners at the top of a dam should be rounded up and a concave curve built at the downstream toe to create a standing wave. The dam is curved up for a few feet in height and terminates in a drop wall. (Also see under "Irrigation").

#### 20. STONE MASONRY—General Specifications

(For characteristics of stones, see Section 12).

##### (a) Random Rubble:

The stones are hammer-dressed on the face, sides and beds to such an extent that the stones will come into close proximity. No stones to tail to a point, nor into the wall less than  $1\frac{1}{2}$  times height, and shall not be of greater height than either breadth of face or length of tail into wall.

*Beds and joints*—Not to exceed 1 in. thick.

*Height of course*—Uncoursed.

*Bonds or through stones*—Bond stones running right through the wall shall be given two per 10 ft. super of face, and staggered.

*Quoins*—Face beds to be squared back carefully at least 4 ins. and joints  $2\frac{1}{2}$  ins.

*Hearting*—Stones to be not less than 6 inches 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  $\frac{1}{3}$  of the face stones shall tail into the wall twice their height.

*Beds and joints*—Not to exceed  $\frac{1}{2}$  in. 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 6 ins. in height and brought up to level beds as may be ordered, 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  $1\frac{1}{2}$  ft. long shall be given so as to approximately provide through-bond of long stones every 5 ft.

*Quoins*—As for (a) above.

*Hearting*—As for (a) above, but vertical plums to be placed wherever possible, projecting not less than 6 ins. to form bond between successive courses

**(c) Square Rubble, uncoursed:**

*Beds and joints*—Not to exceed  $\frac{1}{2}$  in. thick. Face beds to be squared back at least 4 ins. and joints  $2\frac{1}{2}$  ins. No spalls or pinnings to show on face.

*Height of course*—Uncoursed, but no stone to exceed 8 ins. in height to avoid long vertical joints.

*Bond or through stones*—As for (a) above.

*Quoins*—As for (b) above, but corner of each quoin to have chisel drafted margin of 1 in. 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 thicker than  $\frac{3}{8}$  in. The face stone shall be laid alternate headers and stretchers.

*Height of course*—6 ins. to 9 ins. No course to be of greater height than any course below.

*Bond or through stones*—5 ft. apart in the clear in every course and to be staggered, and as for ashlar masonry below.

(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 3 ins.

*Beds and joints*—Not to exceed  $\frac{1}{2}$  in. 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 5 ins. 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  $\frac{1}{2}$  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  $2\frac{1}{2}$  ft. thick, shall be inserted in each course at 5 ft. intervals breaking joints with similar stones in courses above and below at least 2 ft.

*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 back 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

\*"One-line dressed" means sparrow picked or chisel dressed so that no portion of the face dressed is more than  $\frac{1}{4}$  in. from edge of a straight edge laid along face of stone.

\*"Two-line dressed" means sparrow picked or chisel dressed so that no portion of the face dressed is more than  $\frac{1}{8}$  in. from edge of a straight edge laid along face of stone.



horizontal joints with the adjoining stones or brickwork. (Three—line dressed).\*

*Beds and joints*—No joint shall be thicker than  $\frac{1}{2}$  in. 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 12 ins. No stone to be less in breadth than in height, or less in length than twice its height.

*Bond or through stones*—Not exceeding 6 ft. apart in the clear, and to be staggered. In walls  $2\frac{1}{2}$  ft. thick and under, the headers run right through the wall, if more, overlap at least 6 ins.

**Stone Arching.** In arches up to 15 ins. thick all stones shall be through stones extending from the intrados to the extrados. In arches over 15 ins. 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 not be less than 3 ins. thick on their least dimension and shall break joint for not less than 6 ins. and all stones in one course shall be of approximately the same thickness. The thickness of the joint at the intrados shall not exceed  $\frac{1}{4}$  in. 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 12 ins. long and 50 per cent of the stones shall be 18 ins. long or over. The thickness of the joints shall not exceed  $\frac{3}{16}$  ins. and all joints shall radiate properly from the centre of the curve.

\* "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".

In the case of block-in-course, stones shall be not less than 6 ins. thick on their least dimensions and thickness of joints shall not exceed  $\frac{1}{4}$  in.

### **Plumb Concrete and Plumb Masonry**

The proportion of plumbs (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 interstices which should not be less than 6 inches, 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.

## **21. 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 further water after 15 minutes soaking). The bricks should be sufficiently soaked before use but not excessively so.

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 1 in. 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.)

No mortar joint should exceed  $\frac{1}{4}$  in. for 1st class brickwork in cement and  $\frac{3}{8}$  in. for 2nd class brickwork in lime,  $\frac{1}{2}$  in. for 3rd class brickwork in mud mortar, and no joint should be less than  $\frac{3}{16}$  in. 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.



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  $\frac{1}{2}$  in. 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 3 ft. 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 2 ft. above ground level, as bricks which are not thoroughly burnt rapidly corrode away in such a situation. (This subject has been dealt with in detail elsewhere).

**Lime Concrete** :— (see page 6/53)

Brick ballast where used must be well soaked with water before mixing. The ballast should be spread evenly 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.

## 22. 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 (usually of wood) 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 a 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  $2\frac{1}{2}$  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.

*Hydrosopic:* 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 the fire-place.

*Mastic*: A preparation of bitumen used for water-proofing and dampproofing, 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 (of some inches) 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 sunshades, etc., to prevent rain water from running inside towards the walls.

**Underpinning** : 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.

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# PLAIN & REINFORCED CEMENT CONCRETE & REINFORCED BRICKWORK

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Spacing of Round Bars in Slabs

Areas, Perimeters and Weights of Round Bars

The following Notations have been used generally :

- $f_c$  = working compression (in bending) stress in concrete,  
 $f_s$  = working tensile stress in steel,  
 $E_s$  = modulus of elasticity of steel,  
 $E_c$  = modulus of elasticity of concrete,  
 $m$  = modular ratio of steel and concrete,  
 $BM$  = moment of resistance or bending moment,  
 $M_c$  = " " " " relative to the concrete,  
 $M_s$  = " " " " relative to the steel,  
 $A_s$  = total area of tensile steel in sq. inches,  
 $C$  = area of concrete in compression in sq. inches,  
 $b$  = 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; 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  $d$ ,  
 $p$  = ratio of steel to concrete,  
 $S$  = total shear in lbs. at the section under consideration;  
     resistance to shear,  
 $f_s'$  = permissible shearing unit stress,  
 $s$  = pitch or spacing of stirrups,  
 $s$  = permissible unit punching shear stress,  
 $A_w$  = cross sectional area of stirrups,  
 $t$  = thickness of flange of T-beam,  
 $f_b$  = permissible bond stress,  
 $Q = \frac{BM}{bd^2}$   
 $I$  = moment of inertia.

Units: Stresses are in lbs./sq. inch.



## 1. THEORY OF REINFORCING

The use of plain concrete in structural work 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 240 times as much in tension and 24 times as much in compression as concrete. Therefore, a combination of concrete and steel makes for economy.

It has been explained in Section 3 that stress is proportional to the strain, and this relationship is expressed by a constant  $E$  the elastic modulus of the material which  $= \text{stress/strain}$ , where strain is the extension or compression of the material per unit length. For R.C. design purposes steel and concrete are considered perfectly elastic materials although it is not true for concrete.

$$\frac{\text{Stress in steel}}{\text{Stress in concrete}} = \frac{\text{Elastic modulus of steel}}{\text{Elastic modulus of concrete}} = \frac{E_s}{E_c} = m.$$

This is called **Modular Ratio**.

The elastic modulus of steel is taken at a constant value of 30,000,000 lbs. per sq. in. The elastic modulus of concrete is 1000 times the ultimate strength of concrete at 28 days, and the value varies from 2,000,000 to 4,000,000 but is usually taken at 2,000,000 lbs. per sq. in. which gives a modular ratio of 15, which is the ratio usually adopted in design for all mixes and grades of concrete (although it is not correct).

Modular Ratio varies from 18 for a 1:2:4 mix to 14 for a 1:1:2 mix in the case of ordinary grade concretes, and from 14 for a 1:2:4 mix to 11 for 1:1:2 mix in the case of high grade concretes. These values are derived from the relation:

$$\frac{40000}{3 \text{ fc}} \text{ (i.e., } \frac{40000}{3 \times 750} = 17.8 \text{ for 1:2:4 mix).}$$

Adoption of a higher modular ratio would result in reduction in the section and therefore the weight of concrete but an increase in the quantity of steel is necessary.

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 strains in both the materials will be equal. Since stresses are proportional to the respective elastic moduli, the stress in the steel will be 15 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 15 times more stress than the concrete when strained equally, the steel will carry 15 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 entirely 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, variable conditions during placing and other reasons, strength of the concrete will be found to differ quite 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.

### **PROPERTIES AND STRENGTH REQUIREMENTS OF STRUCTURAL CONCRETE AND STEEL**

**Tests on Concrete.** Tests on concrete may be carried out either on site (Field Tests) or in a laboratory. For field grade concrete, tests may be limited to "workability" and the "slump test". Workability means the correct proportioning of coarse and fine aggregates and the water. Slump test 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", with the help of a



machine, which is often difficult. 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 6-in. cubes or 6 ins. diameter and 12 ins. 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  $60^\circ$  and cubes fail at  $45^\circ$  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 2.2 to 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 in flexure 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.

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 strength of the concrete suffer accordingly. Strength of concrete increases with age. For ordinary Portland cement concrete, strength at 3 days is  $\frac{1}{3}$  of the strength at 28 days, strength at 7 days is  $\frac{2}{3}$  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. (Also see under "Curing".)



**Table I. Working**  
in lbs. per sq. in. at 28 days

Concrete Mix		Controlled Concrete*		Good
				1:1:2
Compression	due to bending	0.34 F	0.42 F'	1150
	direct compression	0.26 F	0.33 F'	950 920
	"	..	..	1250
	Bearing Pressure	0.20 F	0.25 F'	675
Tension				115
				180
				250
Shear	without reinforcement	0.034 F	0.042 F'	115
	" "	..	..	92
	max. ..	..	..	155 460
	"			255
	punching shear			210
Bond	straight bars ..	0.04 F	0.05 F'	135
	hooked bars ..			170
	" ..			112

\*F is min : crushing strength at 28 days using cube test specimens

For plain concrete permissible tensile stress in bending shall be taken in the table, which is measured as inclined tension.

For members in direct tension when full tension is taken by the section shall not be greater than four times the values of the

# **Stresses for Concrete**

after placing when properly cured

Ordinary Grade		Type of Work
1:1½:3	1:2:4	
950	750	Beams and slabs.
750	600	Water tanks walls and bottom.
760	600	Columns; whole area of column with independent binders and area of core with helical binders.
1040	840	Columns supporting continuous beams, where neg. bending moment is formed in the beams on the top of the columns.
562	450	Pressure on foundations.
95	75	Common works.
150	110	Cylindrical water tanks to avoid cracks and leakage.
220	180	Rectangular walls of water tanks to avoid cracks.
95	75	Where whole shearing force is resisted by the concrete.
76	60	Water tanks, —ditto.—
128	112	When half bars are bent up.
380	300	When whole of the shearing force is resisted by reinforcement bars, shearing stress in concrete only should not exceed these values.
230	190	Bridge girders, heavy beams, and other important works, —ditto.—
190	150	Shearing stress around edges of columns, heads of piles, etc.
112	100	Beams and slabs.
140	125	" " "
93	85	Water tanks. "

and  $F'$  is using cylinder test specimens.

as equal to the permissible shear stress without reinforcement, given

reinforcement alone the calculated tensile stress on the effective concrete permissible tensile stress given.

**Table II. Permissible Stresses in Reinforcement**

Mild Steel Bars	Lbs./sq.ins.
Tensile bars in beams, bridges and retaining walls, etc.	18000
Tensile bars in walls, floors and slabs of water tanks in contact with water	12000
Tensile bars in ribs of beams of water tanks not in contact with water	16000
To resist shear in beams and slabs, etc.	18000
To resist shear in water tanks	12000
Tension in helical reinforcement of columns	13500
Compression bars in columns, beams, slabs	18000

The yield point stress of rolled mild steel is usually taken at 36,000 lbs. per sq. in., therefore, the usual permissible stress of 18,000 lbs. has a factor of safety of 2 against the yield point.

For reinforcement other than mild steel (high tensile steels and twisted steel bars, etc.) the permissible tensile stress (except in helical binding which is not more than 18,000 lbs.) is half the yield-point stress, but not more than 27,000 lbs. per sq. in. or, in shear reinforcement, not more than 20,000 lbs.; the permissible compressive stress is half the yield-point stress, but not more than 20,000 lbs. per sq. in.

### Strength Requirement for Ordinary Concretes

Mix	Min. Crushing strength in lbs. per sq. in.		Min. Transverse strength			
			Modulus of Rupture in lbs. per sq. in.		Corresponding load in lbs. on specimens 16 in. $\times$ 4 in. $\times$ 4 in.	
	at 7 days	at 28days	at 3 days	at 7 days	at 3 days	at 7 day
1:1:2	2250	3450	225	333	800	1190
1:1½:3	1875	2850	210	300	744	1036
1:2:4	1500	2250	185	265	656	940

The crushing strength is on 6-in. cubes; where cylinders are used, the figures given in the table should be modified according to the ratio cylinder strength/cube strength = 0.8. (Based on the IS : 456-1953.) "Controlled concretes" in which the proportions of aggregates, cement and water are determined by preliminary tests in a



laboratory give higher strengths by about 25 per cent. Vibrated concretes give still higher strengths by about 10 per cent. For rapid-hardening cement minimum cube strength in a works test shall be taken as strength at 7 days.

Poisson's ratio .. .. . 0.15 to 0.20

Co-efficient of linear expansion—0.0000055 per deg. F.

Shrinkage: Strain co-efficient upon drying  
completely .. .. . 0.005

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 as tabled may be taken to have a factor of safety of about  $2\frac{1}{2}$  at 28 days after placing the concrete.

The permissible stresses in concrete and reinforcement in buildings may exceed those given in the foregoing tables by  $33\frac{1}{3}$  per cent if loads due to wind pressure and temperature changes are taken in a design; and may exceed by 50 per cent when seismic loads are added, so long as the stress in the reinforcement does not exceed 27,000 lbs./sq. in. Some engineers recommend that tensile stress in reinforcement may be up to 20,000 lbs. instead of 18,000 lbs.

### SHEAR

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 support. (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.

A beam to be safe against failure by shear must have sufficient thickness of concrete. Permissible shear stress is 10 per cent of the maximum permissible stress in concrete for compression in bending. The intensity of shear stress

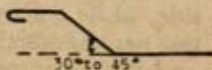
at any cross section  $= \frac{S}{b \times jd}$  for rectangular beams and

$\frac{S}{b' \times jd}$  for T or L beams, where  $S$  is the total shear force at the beam section. Provided the intensity of shear stress does not exceed the permissible shear stress (as given in Table I—75 lbs. per sq. in. for 1:2:4 mix) the section is safe in shear and no shear reinforcement is required. If this stress is exceeded, shear reinforcement must be introduced. Where the permissible shear stress in concrete is exceeded, reinforcement is provided to resist the whole of the shear and the strength of the concrete in shear is ignored as it is assumed that the concrete will develop hair cracks and cease to function.

Even where shear reinforcement is provided to resist the whole of the shear, the cross section of the concrete should be large enough so that the intensity of the maximum shear stress does not exceed four times the permissible shear for the concrete. If not, the section should be increased. 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.

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 though a depth of beam equal to or greater than the lever arm,  $jd$ . Shear resistance of bent bars is calculated from the equation :





$$S = A_s \times f_s' \sin \phi$$

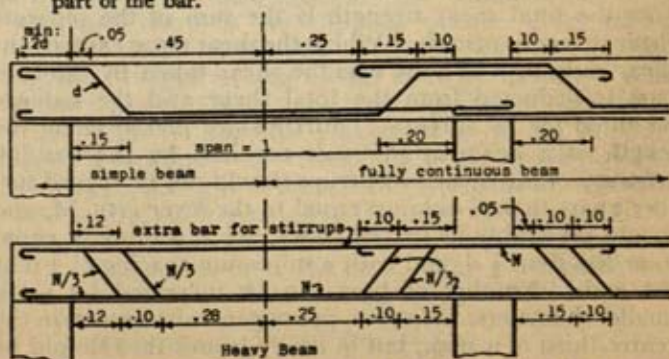
where:  $S$  = shear force resisted by each set of bars;  
 $A_s$  = cross-sectional area of one set of bent-up bars;  $f_s'$  =  
 permissible unit shear stress (18000);  $\phi$  = the angle of the  
 inclined bar with the axis of the beam (30 to 45 deg.).

### Shear Resistance of Inclined Bars

Values in lbs. per each bar

Dia. of bar	$f_s' = 16000$ lbs./sq. in.		$f_s' = 18000$ lbs./sq. in.	
	bend-45° $H/V=1$	bend-30° $H/V=1.73$	bend-45° $H/V=1$	bend-30° $H/V=1.73$
$\frac{1}{8}$ "	1220	860	1370	950
$\frac{1}{4}$ "	2220	1570	2500	1750
$\frac{3}{8}$ "	3470	2460	3900	2750
$\frac{1}{2}$ "	5000	3550	5600	3900
$\frac{5}{8}$ "	6800	4830	7650	5400
1"	8880	6300	10000	7050
$1\frac{1}{8}$ "	11240	7950	12560	8950
$1\frac{1}{4}$ "	13880	9850	15600	11050
$1\frac{3}{8}$ "	16800	11920	18900	13350
$1\frac{1}{2}$ "	19990	14150	22500	15900

$H$ —is the horizontal and  $V$  the vertical projections of the bent part of the bar.



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 excess shear force. Stirrups should invariably be provided along the bent-up bar zone even though not actually necessary for calculated shear strength.

(b) **By 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 should whenever possible be anchored round the top bars, where no compression steel occurs, top bars of light section should be specially provided. All stirrups should be securely wired to the bars round which they pass.

Stirrups may be used either to take the full shear stress or together with the bent-up bars. Shear force resisted

$$\text{by stirrups} = S = \frac{A_w \times f_s' \times j d}{s'} \quad \text{or} \quad s' = \frac{A_w \times f_s' \times j d}{S}$$

Since a stirrup has two or more legs,  $A_w$  will be the sum of the cross-sectional areas of the legs;  $s'$  is the pitch or spacing of stirrups.

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 the 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. Stirrups should not be spaced further apart than a distance equal to the lever arm,  $j d$ , and should preferably be spaced closer, say at a distance equal to or less than  $\frac{1}{2} d$ , and with a minimum spacing of  $\frac{1}{4} d$  at the ends. 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 end of the beam to the wider spacing at mid-span which should not exceed  $\frac{1}{2} d$ . This does not apply to beams under bridges carrying heavy rolling loads.

The stirrup bars used are of size  $\frac{1}{8}$  in. to  $\frac{3}{8}$  in. 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 =  $\frac{1}{2}$  effective span—(shear which beam can take without web reinforcement  $\div$  load per ft. on the beam.)

**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 support, the permissible shear stress for the section is taken  $\frac{4}{3}$  of the usual permissible stress and is given in Table I.

Also see under "General Design Principles for Slabs and Beams".

## 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 stress and both stretch or compress together. Slight rusting of the steel increases the bond. In order that the bar is not pulled out when subjected to tensile stress, 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. The permissible bond stress for different mixes is given in Table I.

$$\frac{\pi d^2}{4} \times f_s = \pi d \times L \times f_b \text{ or } L = \frac{f_s \times d}{4 \times f_b}$$

Taking  $f_s = 18000$  lbs./sq. in.,  $f_b = 100$  lbs./sq. in.

$$\text{for } 1:2:4 \text{ mix, } L = \frac{18000 \times d}{4 \times 100} = 45d.$$



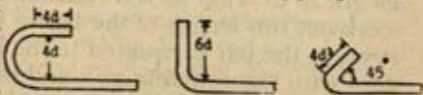
Where:  $d$  = dia. of bar;  $f_s$  = tensile stress in steel;  $L$  = length of bar to be embedded;  $f_b$  = permissible bond stress.

Therefore, a bar must be embedded 45 times its diameter (which includes for hooks and bends) in order to develop full bond stress. This will be greater with high tensile steels (about 68d) 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 amount of reduction being 4 diameters for each 45 deg. through which the bar is bent to form the hook. The reduction for a standard semi-circular hook (as described hereafter) is 16d. According to B.S. Code of Practice, the bond length should not be less than 36  $d$  measured up to the end of the hook. 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:**

Compression steel is normally stressed only to  $m \times$  stress in surrounding concrete and not to the full 18000 lbs. per sq. in. Therefore, for compression bars in beams, a bond distance of 24d\* should be adequate. 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 and security against slipping. In slabs hooks are not essential, but for any tensile bars which are curtailed and do not run through to the supports, hooks are essential. The most



According to B.S.—CP 114, the bond length for a bar in

$$\text{compression} = \frac{\text{compressive stress in the bar}}{5 \times \text{permissible bond stress}}$$

According to IS: 456—1953.

$$= \frac{\text{compressive stress in the bar}}{4 \times \text{permissible bond stress}}$$



common and the best form of anchorage is a semi-circular hook as shown in the illustration which is the standard form of hook. The length of the straight part of the bar beyond the end of the curve should be at least  $4d$ , but not less than 3 ins. The total length of the bar required for the hook is  $12d$  measured from the point the bar starts to bend.

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  $6d$ , but prefer  $10d$ . This is called a square hook.

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  $20d$  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  $2d$  or 1 in. (min.) on sides and  $3d$  or  $1\frac{1}{2}$  ins. (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.

**Local bond stress or shear bond stress** is due to the horizontal shear caused in the beam under load, which is maximum at the ends and varies as the bending moment, (and is distinguished from the direct bond stress or anchorage mentioned above), is calculated from the following equation:

$$\text{Local bond stress} = \frac{S}{jd \times O}$$

where:  $S$  = max. shear force at the ends;  $jd$  = lever arm of the beam;  $O$  = the sum of the perimeters of all the tensile bars carried to the support.

The local bond stress may exceed 1.75 times the appropriate permissible direct bond stress.

If the stress is beyond the permissible 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.

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 bars should be continued a small further distance beyond the calculated point of curtailment for anchorage and should be hooked. It is preferable to bend the bars up for shear. In all cases the number of bars run through to the support must be sufficient for local bond strength.

**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: 45d for tensile bars and 24d for compression bars for the particular mix. The ends of the overlapping tensile bars should preferably be hooked in which case the length can be reduced as described previously. 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 24d. For works in water, take 30d for vertical bars and 40d for hooked circular rings.

Where steel fabrics are used in slabs, ends should be lapped 12 ins. and wired together. Side laps are not necessary.

Butt 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.

**Cutting and Bending Bars.** Bars up to  $\frac{1}{2}$  in. diameter can be cut by hand shears or chisel. Large bars require a simple handlever machine bolted down to a base. For bars of 1 in. diameter and larger, oxy-acetylene 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 preferably be bent



cold. Usually hot bending should be limited to bars  $1\frac{1}{2}$  ins. 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	..	dia. of bar—(min. $\frac{1}{2}$ " )
Beams—side, top, bottom	..	dia. of bar—(min. 1" )
„ ends	..	twice dia. of bar—(min 1" )
Columns	..	dia. of bar—1" min. up to $7\frac{1}{2}$ " side, $1\frac{1}{2}$ " above $7\frac{1}{2}$ ".
Piles	..	$1\frac{1}{2}$ " min. (sides)
Water tanks	..	$1\frac{1}{4}$ " with rich concrete, $1\frac{1}{2}$ " with 1 : 2 : 4 mix.
Marine works	..	twice ordinary structures
Foundations	..	$1\frac{1}{4}$ "

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  $1\frac{1}{2}$  times beyond the figures given above.

The equivalent dia. in the case of a square bar may be assumed to be about  $1\frac{1}{8}$  times the side of the bar, and for a twin-twisted bar  $1\frac{1}{2}$  times the actual dia. of one of the bars.

## 2. GENERAL DESIGN PRINCIPLES FOR SLABS AND BEAMS

### Practical Rules

(a) *The effective span* of a beam or slab should be taken as the lesser of: (i) the distance between centres of



supports, (ii) the clear distance between supports plus the effective depth of the beam or slab. Effective depth is from top of concrete to the centre of tensile steel.

(b) Slabs are not generally made less than 3 ins. overall thickness.

(c) Simply supported slabs should have an effective depth of not less than  $1/20$  of the clear span. In the case of slabs spanning in two directions, it may be  $1/30$  of the span. Continuous slabs may have a thickness of  $1/30$  and semi-continuous  $1/22$  of the span. For cantilevers the effective depth near the support should not be less than  $1/7.5$  of the projection. These depths obviate deflection. (Also see under "Deflection" at page 8/23).

The top surface of centering should be given a camber of  $1/12$  in. for every foot of span subject to a maximum of  $1\frac{1}{4}$  ins., to allow for initial settlement.

**Reinforcement in Slabs.** Generally, the spacing of main bars may not exceed twice the effective depth or 12 ins., whichever is less. Maximum spacing permitted in the Code is  $3d$  for tensile bars and  $4d$  for transverse reinforcement ( $d$  is effective depth). Closer spacing is, however, better. The minimum spacing of bars is as prescribed for beams.

Main tensile reinforcement bars in slabs should not be less than  $\frac{1}{4}$  in. diameter, and bars of diameter greater than  $\frac{1}{2}$  in. should not be generally used.  $\frac{3}{8}$  in. and  $\frac{1}{2}$  in. are the most common and convenient sizes for slabs. Use as few different diameters as possible. Well distributed steel reinforcement reduces the shrinkage and expansion of reinforced concrete.

In simply supported single span slabs, it is not normally necessary to bend up any bars. In partially fixed conditions (which is the most common and occurs where roof slabs are built into walls), every third bar should be bent up. In slabs continuous 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. Former arrange-

ment is more usual and also economical. Stirrups are not used in slabs.

It is not necessary to check shear stress on a slab except with a superimposed load of over 500 lbs. per sq. ft. (See also under "Shear in Thin Slabs").

**Distribution Bars.** Also called "Temperature Reinforcement", "Transverse Reinforcement", or "Secondary Reinforcement." Cross bars of diameter  $3/16$  to  $7/16$  in. (usually  $1/4$  in. and  $3/8$  in.) are provided on the top of the main tensile bars in slabs spanning in one direction. 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.

The transverse bars should be at least 25 per cent of the main tensile bars for roof slabs subject to much temperature changes where additional bars should 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 transverse bars should not be placed further apart than four times the effective depth of the slab, or 18 ins., whichever is less. Main and cross bars are tied together with 16 gauge soft iron wires. About 10 lbs. of wire is required for one ton of bars.

Distribution bars should not be hooked or bent as hooks will tend to localize the cracks and make the distribution steel non-effective.

### Practical Rules—Beams

(a) The effective depth of any rectangular beam should not be less than  $1/18$  of the clear span to minimize deflection.

(b) The width should not be less than  $1/20$  of the clear span to avoid buckling of the compression flange. (See under "Lateral Stability of Beams" in Section 3.) Where the span/width ratio exceeds twenty and it is not practicable to support the compression flange laterally, the compression stresses must be reduced by  $2\frac{1}{2}$  per cent



for every unit by which the ratio exceeds twenty. The width, however, should not be less than  $1/32$  of the span.

(c) The minimum depth of a singly reinforced rectangular beam should not be less than the width, should normally be about twice its width, but should not be more than three times the width. A good rule for the width of a rectangular beam is to take  $3/5$ th of the total depth of the beam.

(d) The top surface of centering should be given a camber of  $1/18$  in. for every foot of span subject to a maximum of  $1\frac{1}{2}$  ins., to allow for initial settlement.

**Reinforcement in Beams.**—The clear space between two parallel reinforcement bars should be not less than the diameter of the larger bar, or  $\frac{1}{4}$  in. (prefer  $\frac{1}{2}$  in.) more than the maximum size of the coarse aggregate, or  $1\frac{1}{2}$  times the maximum size of the aggregate, whichever is more.

The clear vertical space between the main horizontal reinforcement bars should be more than the maximum size of the coarse aggregate, and not less than  $\frac{1}{2}$  in. The bars may also be placed one above the other without any space in between. Steel spacer bars should be introduced to maintain correct horizontal and vertical distances apart of the bars.

Main tensile reinforcement bars in beams should not be less than  $\frac{1}{4}$  in. diameter. Use as few different diameters as possible. Stirrups and secondary reinforcement bars are  $3/16$  in. to  $\frac{1}{4}$  in. diameter. 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  $\frac{3}{8}$  in. when not required to take any bending moments.

**Bearings on Walls.** Allow the following minimum bearings:—

Solid slabs	4 ins.	} The bearing of the ends of a beam on a wall beyond a certain distance does not strengthen a beam.
Lintels	6 ins.	
Concrete joists	6 ins.	
Concrete beams	8 ins.	



**Deflection.** To avoid deflection in slabs that might impair the strength or efficiency of the structure or produce cracks in finishes, the ratio of span to effective depth should not exceed 35 for simply-supported slabs spanning in one direction, 45 for continuous slabs spanning in one direction, and 12 for cantilever slabs.

### Calculating Moment of Inertia of R.C. Sections

One of the following methods may be adopted, and the same method should be followed throughout the design for both beams and columns otherwise it will lead to different results:

(i) Consider the entire concrete section including the reinforcement area at its appropriate modular ratio, *or*

(ii) Take the full area of the section, ignoring the reinforcement, *or*

(iii) Take the compression area of concrete ignoring the tension area and allowing for steel at its appropriate modular ratio.

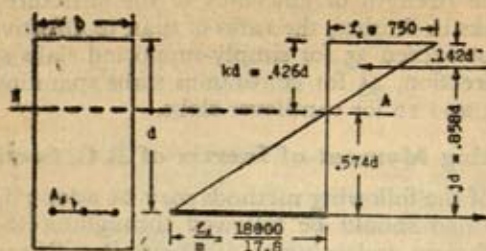
The **stiffness factor** of a member of constant cross-section is the moment of inertia divided by the length of the member.

### DERIVATION OF MOMENT OF RESISTANCE OF SINGLY REINFORCED BEAMS AND SLABS IN BENDING based on the following data for a 1:2:4 ordinary mix:—

Tensile stress in steel ( $f_s$ )	= 18000 lbs./sq. in.
Comp. stress in concrete ( $f_c$ )	= 750 "
Modular ratio ( $m$ )	= 17.8
Percentage of steel ( $p$ )	= 0.89% $bd$
Neutral axis ratio ( $kd$ )	= 0.426 $d$
Lever arm ( $jd$ )	= 0.858 $d$
Bending Moment (BM)	= 137 $bd^2$

For a design to be based on other values of  $f_s$ ,  $f_c$  or  $m$  see Table at page 8/26.

The above values are arrived at as follows:—



**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 above diagram represents the stresses in concrete and steel. The "effective depth",  $d$ , of the beam is measured from the compression edge to the centre of the tensile steel.

$$\frac{f_s}{m} \times kd = f_c \times (d - kd) \quad \text{from which}$$

$$k = \frac{m \times f_c}{m \times f_c + f_s} = \frac{17.8 \times 750}{17.8 \times 750 + 18000} = 0.426.$$

(It is not always necessary to work out the neutral axis for simple designs.)

**Stresses.** The stresses vary uniformly on a cross-section from zero at the neutral axis to a maximum at the extreme edges. Total compressive force in the concrete on a cross-section  $= \frac{1}{2} f_c \times kd \times b$  lbs. This force acts at the centroid of the compression triangle, i.e., at a depth  $\frac{1}{3} kd = 0.142d$  from the top

Total tensile force  $= f_s \times A_s$  lbs. acting at a depth  $d$  below the top edge.

For equilibrium, total compression = total tension, i.e.,  $\frac{1}{2} f_c \times kd \times b = f_s \times A_s$ .

Therefore the **area of steel**,  $A_s$ , in a balanced section

$$= \frac{f_c \times kd \times b}{2 f_s} = \frac{750 \times 0.426d \times b}{2 \times 18000} = 0.0089 \text{ bd.}$$

This is referred to as the "economic percentage" of steel.

**Moment of resistance.** The total compression and the total tension which are equal and act in opposite directions form a couple with a "lever arm",  $jd$ , equal to the distance between the centroid of the compression triangle and the centre of the steel. Moment of resistance of the beam = either total compression or total tension  $\times$  lever arm.

Moment of resistance due to concrete in compression

$$= \frac{1}{2} f_c \times kd \times b \times jd = \frac{1}{2} \times 750 \times 0.426d \times b \times 0.858d$$

$= 137 bd^2$  in lbs. This is equal to the moment of resistance due to steel in tension.

$$= A_s \times f_s \times jd = 0.0089 bd \times 18000 \times 0.858 d = 137 bd^2$$

$A_s = \frac{BM}{f_s \times jd}$ . 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  $137 bd^2$  in. lbs., which is equated to the bending moment of the beam or slab under loading.

**Procedure for design.** Singly reinforced beams and slabs: For beams, assume breadth about  $1/20$  of the span and depth  $2 \times$  breadth. Calculate the maximum bending moment including an allowance for the weight of the beam itself. For slabs, assume 12-in. wide strip for the breadth.

$$d = \sqrt{\frac{BM}{137b}}$$

**Checking an existing design.** With a given depth, breadth and reinforcement in a beam, the following formulae give the stresses in concrete and steel under a given bending moment,  $BM$ :—

$$f_c = \frac{2 BM}{kd \times b \times jd};$$

$$f_s = \frac{BM}{A_s \times jd}$$

Also

$$f_c = \frac{f_s \times k}{m \times (1 - k)};$$

$$f_s = \frac{m \times f_c (1 - k)}{k}$$



## Constants for Balanced Design

Stress in steel	Stress in concrete	Modular ratio m	Neutral axis kd	Lever arm jd	Economic percentage of steel p	Moment of resistance
18000	750	17.8	0.426d	0.858d	0.89	137bd <sup>3</sup>
18000	750	18	0.428d	0.860d	0.89	138bd <sup>3</sup>
18000	750	15	0.385d	0.872d	0.80	126bd <sup>3</sup>
18000	850	15	0.415d	0.862d	0.98	152bd <sup>3</sup>
18000	950	15	0.440d	0.853d	1.17	179bd <sup>3</sup>
18000	680	15	0.362d	0.880d	0.68	108bd <sup>3</sup>
18000	600	15	0.333d	0.889d	0.56	89bd <sup>3</sup>
16000	600	15	0.360d	0.880d	0.675	95bd <sup>3</sup>
16000	750	18	0.458d	0.847d	1.08	146bd <sup>3</sup>
16000	680	15	0.390d	0.870d	0.830	116bd <sup>3</sup>
16000	750	15	0.413d	0.863d	0.970	133bd <sup>3</sup>
12000	680	15	0.460d	0.847d	1.30	132bd <sup>3</sup>
12000	750	15	0.483d	0.840d	1.51	152bd <sup>3</sup>
12000	850	15	0.515d	0.828d	1.80	179bd <sup>3</sup>

## High Grade Concretes

18000	1000	15	0.45d	0.85d	1.26	193bd <sup>3</sup>
18000	1200	15	0.51d	0.83d	1.77	264bd <sup>3</sup>
18000	1500	15	0.56d	0.82d	2.32	339bd <sup>3</sup>

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):—(For 18000, 750, 15)

Steel percentage	0.80	0.63	0.43	0.28	0.17	0.09	0.04
MR/bd <sup>3</sup>	125	100	69	46.2	28.5	15.4	6.9

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	0.88	1.23	1.66	2.24	3.00	4.03	5.45
MR/bd <sup>3</sup>	126	130	144	157	168	180	192	202

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.

## CALCULATION OF 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.): Two 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 support (partially fixed) | $-wL/24$ |

+ sign is for the positive bending moments occurring 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 contraflexure 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.

**DATA FOR R.C. SLABS**

Simply supported

Safe loads (including weight of slabs) in. lbs./sq. ft.

On spans of——ft.

Moment of Resistance			Total depth of slab		Depth to centre of steel		DATA FOR R.C. SLABS Simply supported									
Safe loads (including weight of slabs) in. lbs./sq. ft.																
On spans of ———— ft.																
ft. lbs.	in.	in.	5	5½	6	6½	7	7½	8	8½	9	9½	10			
410 690	2½ 3	1½ 2½	133 219	110 181	92 153	79 130	112	98	87	77	69					
1020 1430	3½ 4	2½ 3½	343	271 381	228 319	195 272	167 233	146 203	128 179	114 159	103 141	127	115			
1670 2180	4½ 5	3½ 4			372	317 412	274 382	238 310	209 273	185 240	165 215	149 193	134 174			
2760 3400	5½ 6	4½ 5						392	345 425	306 378	273 336	244 301	221 273			
3750 4480	6½ 7	5½ 5½							468 565	415 495	371 443	333 398	300 359			
5300 6200	7½ 8	6½ 6½									525	470	425 496			
8200 10400	9 10	7½ 8½											656			
12300 15000	11 12	9½ 10½														

For other conditions of support, the spans should be multiplied by  
 WL/10 by 1.12; WL/12 by 1.25; WL/16 by 1.41; WL/2 by 0.5

Beams and slabs constructed monolithically with the supporting girders, beams or columns, are taken as continuous and also the slabs spanning between T beams.

**Doubly Reinforced Beams** are beams reinforced in compression as well as in tension. 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  $m \times$  stress in concrete around the steel



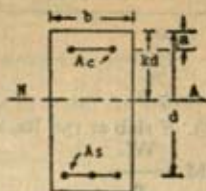
Stresses for 1:2:4 mix						Steel area per ft. width.	REINFORCEMENT							
18000 750 75 m=17.8 Wt. of slab at 150 lbs./c.ft. WL BM. = $\frac{WL}{8}$							(For distribution bars see page 8/31)  Round Bars Spacings in inches of main tensile bars for diameters of:							
11	12	14	16	18	20		$\frac{1}{4}"$	$\frac{5}{16}"$	$\frac{3}{8}"$	$\frac{1}{2}"$	$\frac{5}{8}"$	$\frac{3}{4}"$	$\frac{7}{8}"$	
						.187	$2\frac{1}{4}$							
						.240	$2\frac{1}{4}$	$3\frac{1}{4}$	$5\frac{1}{4}$					
						.294		3	$4\frac{1}{4}$					
						.347		$2\frac{1}{4}$	$3\frac{1}{4}$	$6\frac{1}{4}$				
104						.374		$2\frac{1}{4}$	$3\frac{1}{4}$	6				
144	125					.427			3	$5\frac{1}{4}$				
182	153	114				.481			$2\frac{1}{4}$	$4\frac{1}{4}$				
225	189	140				.534			$2\frac{1}{4}$	$4\frac{1}{4}$	7			
248	208	153				.561			$2\frac{1}{4}$	$4\frac{1}{4}$	$6\frac{1}{4}$			
296	249	183	145			.614			2	$3\frac{1}{4}$	6	$8\frac{1}{4}$		
352	294	216	170			.668				$3\frac{1}{4}$	$5\frac{1}{4}$	8	$10\frac{1}{4}$	
409	345	252	195			.721				$3\frac{1}{4}$	5	7	10	
541	455	335	257	202		.828					$4\frac{1}{4}$	$6\frac{1}{4}$	$9\frac{1}{4}$	
		425	326	258	209	.935					4	$5\frac{1}{4}$	$8\frac{1}{4}$	
		500	383	303	245	1.02					$3\frac{1}{4}$	5	7	
					300	1.11					$3\frac{1}{4}$	$4\frac{1}{4}$	6	

the following factors, for the same loads:—  
(Cantilevers).

i.e.,  $m \times \frac{2}{3} f_c$  (approx.) For  $m=17.8$ , and  $f_c=750$ , stress in compression steel is only about 8,900 lbs. per sq. in. instead of its capacity of 18,000 lbs. per sq. in. It is not possible to increase this compressive stress in the steel without overstressing the concrete above its permissible working stress. Such beams are used where the depth is fixed due to design considerations.

A doubly reinforced beam may be designed by the following simple and approximate method:

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 economic percentage of steel. Let this bending moment be  $B'M$ . 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  $=$

$$\frac{BM - B'M}{f_s \times (d - a)} = A_s' \quad \text{Total tensile reinforcement } A_s$$

$$= A_s' + (p \times bd)$$

Compressive reinforcement at top for the balance bending moment will be:—

$$A_c = \frac{m \times A_s' (d - kd)}{(m - 1) \times (kd - a)}, \text{ or approx. } A_c = \frac{A_s' (d - kd)}{(kd - a)}$$

Assuming that  $a = \frac{1}{3} kd$ , total compression on concrete and steel then becomes  $= \frac{1}{2} \times f_c \times kd \times b + \frac{2}{3} \times m \times f_c \times A_c$ . This is equal to the tension on tensile steel, i.e.,  $f_s \times A_s$ . Moment of resistance of the beam due to compression concrete and steel  $= [\frac{1}{2} \times f_c \times kd \times b \times (d - a)] + [\frac{2}{3} \times m \times f_c \times A_c \times (d - a)]$

Moment of resistance of the beam due to tensile steel  $= f_s \times A_s \times (d - a)$ . Both the moments must be equal to maintain equilibrium.

The calculated moment of resistance of a rectangular beam, with equal top and bottom reinforcement, will be as under for  $f_c = 750$ ,  $f_s = 18000$  and  $m = 15$ .

Percentage of steel for tension and compression	4.0%	3.0%	.0%	1.5%	1.0%
Moment of resistance	$405bd^2$	$311bd^2$	$265bd^2$	$230bd^2$	$157bd^2$

Moment of resistance due to compression concrete and steel are less than moment of resistance due to tensile steel, for percentage of steel above 1 per cent. Therefore,

provision of equal tensile and compressive steel is only wasteful, and tension cracks also develop in these beams. Compression steel should not exceed 4 per cent.

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.

Details of Distribution  
bars for the  
R.C. Slabs Table  
on page 8/29

**Data for R.C. Beams Reinforced  
for Tension**  
per 12 inches width of beam

Total depth of slab in.	Round bars	Bending moment	Total depth	Effective depth	Area of tensile rein- forcement
	ins.	1000 in. lbs.	ins.	ins.	sq. ins.
2½"	④ @ 7"	45.4	6	5.25	0.56
3"	⑤ " 9"	54.4	6½	5.75	0.62
3½"	⑥ " 9"	64.3	7	6.25	0.67
4"	⑦ " 8"	75.0	7½	6.75	0.72
4½"	⑧ " 8"	83.0	8	7.0	0.76
5"	⑨ " 7"	95.0	8½	7.5	0.81
5½"	⑩ " 6"	108.0	9	8.0	0.87
6"	⑪ " 5"	121.5	9½	8.5	0.92
6½"	⑫ " 5"	133.2	10	9.0	0.96
7"	⑬ " 10"	148.3	10½	9.5	1.02
7½"	⑭ " 9"	164.4	11	10.0	1.07
8"	⑮ " 8"	181.2	11½	10.5	1.12
9"	⑯ " 8"	199.0	12	11.0	1.18
10"	⑰ " 7"	253.5	13	12.0	1.28
11"	⑱ " 6"	277.8	14	13.0	1.39
12"	⑳ " 5"	322.3	15	14.0	1.50
		447.5	18	16.5	1.76
		760.0	24	21.5	2.30

Stress in steel 18000 lbs./sq. in.

" " const. 750 "

Modular Ratio-m 17.8 "

BM=137 bd<sup>2</sup>



**SLABS SUPPORTED ON FOUR SIDES**

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 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	1.1	1.2	1.25	1.3	1.4	1.5	1.75	2.0
Across long span	.50	.41	.32	.29	.25	.20	.17	.10	.06
Across short span	.50	.59	.68	.71	.75	.80	.83	.90	.94

Reinforcement for negative bending moments will be  $\frac{4}{5}$ th and  $\frac{1}{3}$ rd 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.

(The above 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. Long span reinforcement is placed above the short span reinforcement. 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  $1\frac{1}{2}$  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:—

$$\text{Load on short span beam: } W_s = \frac{wL_s^2}{4}$$

$$\text{Load on long span beam: } W_L = \frac{wL_sL_L}{2} - W_s$$

$L_L$  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  $1/32wd^2$  for simply supported,  $1/40wd^2$  for partially fixed, and  $1/48wd^2$  for fixed ends.  $w$  is the load per unit area and  $d$  is the diameter. The bending moment is maximum at the centre and is reduced towards the circumference, therefore, reinforcement may also be reduced towards the circumference. 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.

### 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 6 to 9 ft., framing into and supported by the main beams. The end spans should be made about 0.8 to 0.9 of the interior spans so as to equalize the bending moments. The advantage of such a section is: the horizontal flange of concrete supplies resistance to compression while the vertical rib gives depth and lever arm.

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-beams;  
 (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 not be 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. The slab is usually designed as a normal continuous slab spanning from rib to rib transversely to the line of ribs. 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 12 ft. The slab is designed first so that the flange thickness,  $t$ , is decided.

**Design of rib.** The over-all depth of a T-beam should not be less than  $1/20$  of the clear span and is usually 4 to 6 times the thickness of the flange. 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— $1/20$  of the span; (b) for medium loads— $1/15$  of the span; (c) for heavy loads (300 lbs. per sq. ft. and over)— $1/12$  of the span.

Economic depth of a T-beam can also be based on the ratio of the cost of concrete and steel one c.ft.

$$\text{Economic depth of a T-beam} = \frac{\sqrt{R \times BM}}{\sqrt{fs \times b'}} + \frac{t}{2}$$

$R = \frac{\text{cost of 4.38 cwts. of steel}}{\text{cost of 1 c. ft. of concrete}}$  i.e., ratio of cost of steel and concrete 1 c.ft. (about 50 to 60); BM is in inch lbs.

Width of rib,  $b'$ : Since the concrete in the rib does not



do any work to carry the bending moment, this may be kept as narrow as possible, but it has to be sufficiently wide to accommodate steel reinforcement. As a rough rule, a width of  $2\frac{1}{2}$  times the sum of diameters of bars is required. The rib cannot be less than 6 ins. wide where more than one bar is required. It is most economical to make  $b' = \frac{1}{2}d$ , but it should not be less than  $\frac{1}{3}(d-t)$  or less than  $2t$ , and not more than  $\frac{3}{4}d$ . Under distributed loads it may have the minimum size prescribed, but under concentrated loads check for shear.

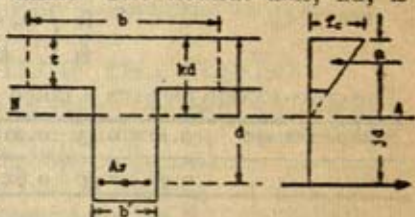
Shear is the deciding factor in fixing the size of a rib and the cross-section must be sufficient to resist the shear force. The rib takes the entire shear, and its depth is measured from the top of the flange. The flanges of T-section are of negligible value in resisting shear.

Total shear force  $= b' \times (d - \frac{1}{2}t) \times fs'$

$fs'$  is the permissible shear stress and is given in Table I.

### 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.



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  $t/2$  below the compression edge.

**Stresses.** The compressive stress varies from a maximum,  $f_c$ , at the extreme compression edge to zero at the neutral axis. Total compressive force on the concrete

$$= \left( \frac{kd - \frac{1}{2}t}{kd} \times f_c \right) \times b \times t \text{ lbs.}$$

which for equilibrium of the section of the beam = total tensile force in steel =  $A_s \times f_s$  lbs. acting at a depth  $d$  below the top edge of the flange.

**Moment of resistance of beam.** The compressive force acts at a distance  $\frac{1}{2}t$ , and tensile force acts at a distance  $d$  from the extreme compression edge. Therefore, the lever arm  $jd = d - \frac{1}{2}t$ .

Moment of resistance due to concrete in compression =  $BM$

$$= \left( \frac{kd - \frac{1}{2}t}{kd} \times f_c \right) \times b \times t \times jd, \quad \text{which is} = A_s \times f_s \times jd.$$

$$\text{Therefore, } f_c = \frac{BM \times kd}{bt(kd - \frac{1}{2}t) \times jd} \quad \text{and } A_s = \frac{BM}{f_s \times jd}$$

$f_c$  is compressive stress in the concrete and  $f_s$  is tensile stress in the steel.

**Steel percentage,  $p$ , at equal strength ratio**  
as

$$= \frac{f_c}{f_s} \left( \frac{kd - \frac{1}{2}t}{kd} \right) \frac{t}{d}.$$

The cross-section for area is taken =  $b \times d$

Steel percentage	0.10	0.15	0.20	0.25	0.30	For 750, 18000, and $m=15$ .
$t/d$	0.36	0.50	0.62	0.71	0.76	

The following table will be found useful for designing  
T-Beams

$f_s$	$f_c$	$m$	$Q = \frac{BM}{bd^3}$						
			ratio of $t/d$						
			0.10	0.15	0.20	0.25	0.30	0.35	0.40
18000	750	18	64	84	102	118	127	134	137
18000	750	15	62	86	100	114	120	124	..
16000	600	15	50	66	78	86	92	94	..

**Continuous T-beams.** The design is similar of 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 rib 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.

### **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 R.C. 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 the lower hollow tiles. The tiles are spaced 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 R.C. slab floor of similar depth.

Ribs and slab are placed monolithically and the ribs are not further apart than 3ft. face to face. The ribs should have a minimum width of 3 ins. to 6 ins. and depth not over three times the width. Slab thickness is  $1\frac{1}{2}$  ins. to 3 ins.

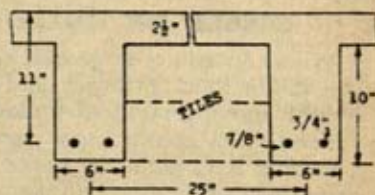


A strip of solid slab at least 4 ins. wide should be left adjacent to the supporting beams or walls. Light reinforcement is provided in the slab at right angles to the ribs and just above the hollow tiles.

Design follows the same principles as for T-beams. The concrete above the tiles takes compression and for calculation of neutral axis and lever arm, the depth of compression flange is usually taken = actual depth of concrete slab over the tiles  $+\frac{1}{2}$  in. = say,  $x$ . If the neutral axis lies within the above depth,  $x$ , of the compression flange, the floor is designed as a solid slab with lever arm  $jd = d - \frac{1}{3}kd$ . If the neutral axis lies below the depth  $x$  of the compression flange, design the floor as a series of T-beams. The compression flange depth ( $t$ ) =  $x$ . Lever arm  $jd = d - \frac{1}{3}x$ . 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—22 ft.  
Live load—60 lbs.  
$$BM = \frac{WL}{10} = 178900 \text{ in. lbs.}$$
  
$$As = 1.02 \text{ sq. in. in each rib.}$$



(Concrete is very much under stressed)

### 3. COLUMNS

**Types of Columns.** 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 to prevent the concrete from splitting outwards on planes of greatest shear stress which

is at  $45^\circ$  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 are spirally round the vertical reinforcements in continuous helix equally spaced. 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 is one in which a steel or cast iron section is completely encased in concrete.

### 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 R.C. column can carry with longitudinal bars and lateral ties:

$$P = f_c \times A + f_s \times A_s$$

(ii) Safe axial load  $P$  a short R.C. column spirally reinforced can carry:

$$P = (f_c \times A') + (f_s \times A_s) + (2f_h \times A_h)$$

Where:  $f_c$  = permissible stress in concrete in direct compression;  $A$  = cross-sectional area of concrete excluding reinforcing bars;  $f_s$  = permissible compressive stress in steel for column bars;  $A_s$  = cross-sectional area of concrete in core of column;  $A_s$  = cross sectional area of longitudinal bars;  $f_h$  = permissible stress in helical reinforcement;  $A_h$  = volume of helical binder per unit length of column.

The value of  $f_c \times A + 2f_h \times A_h$  should not exceed  $\frac{1}{4} \times$  crushing strength of concrete  $\times A$ .

Permissible stresses are given in Tables I and II.

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.

**Short and Long Columns.** Columns are considered short or small when the ratio of the "effective" column



length to its least lateral dimension does not exceed 15. This is called "slenderness ratio" or "buckling factor." Columns generally fail by buckling unless the length is small. Safe loads obtained from the above equations for short columns should be multiplied by the reduction factors given in the following table, to obtain safe loads on long columns.

Reduction Factors for Long Columns

Ratio $l/d$	15	18	21	24	27	30	33	36	39	42	45
Factor	1.0	0.9	0.8	0.7	0.6	0.5	0.4	0.3	0.2	0.1	0

$l$  is the "effective length",  $d$  is least lateral dimension.  $d$  is based on gross cross-sectional area of column with independent binders and on core diameter with helical binders.

#### "Effective" Length of Columns :

(i) Fixed at both ends in position and direction e.g., a column with a deep footing at bottom and whose top end is fixed by four beams from four directions,  $-0.75L$ . ( $L$  is actual length.)

(ii) Fixed at both ends in position but not in direction,  $-L$ .

(iii) Fixed at one end in position and direction, but imperfectly fixed at the other end,  $-L$  to  $2L$  depending on the degree of fixidity of the imperfectly fixed end. Where the upper end is free, it is  $-2L$ .

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 R. C. C. 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 R. C. C. column and beam works, actual length will be the effective length.



The length of a column should not be more than  $45d$  for axially loaded columns and  $20d$  for columns subjected to bending in addition to direct loads.

### **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 bracket may be calculated as explained in Sections 3 and 10, with the equation  $\frac{W}{z} \pm \frac{M}{z}$ .  $A$  is the equivalent concrete area when the column is reinforced and is  $=bd \times (m-1)A_s$ .

$$I = \frac{bd^3}{12} + \left[ (m-1)A_s \times \left( \frac{d}{2} - a \right)^2 \right]; \quad Z = \frac{I}{\frac{1}{2}d}$$

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 R. C. columns if,  $e$ , (eccentricity) is less than about  $0.2d$ , 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 I (750 for 1 : 2 : 4, and not 600), 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 8 per cent of the gross cross-sectional area of the column. From 1 to 4 per cent of steel is commonly used in tied column and greater per cent in spiral columns. Sizes of longitudinal bars usually vary from  $\frac{1}{2}$  in. min. to  $1\frac{1}{2}$  ins. 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 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  $1\frac{1}{2}$  times the diameter of round bars or two times the side dimensions of square bars.

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 3 ins. 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 must be bent inwards at floor level, or must be stopped off short below floor level.

**Transverse reinforcement.** The diameter of the transverse reinforcement (binders) should not be less than a quarter of the diameter of longitudinal bars nor less



than  $\frac{3}{8}$  in. except for very small columns when up to 16 gauge wire can be used which is the size of wire used for tying the bars. The maximum size of binders used is  $\frac{1}{2}$  in. for independent ties and up to 1 in. 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 the transverse reinforcement should not exceed the least lateral dimension of the column, or twelve times the diameter of the smallest longitudinal bar, or 12 ins., whichever is least, and need not be less than 6 ins. If there are more than four longitudinal bars, additional ties should be provided.

The volume of transverse reinforcement should not be 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 3 ins. nor more than one-sixth of the core diameter of the column, whichever is less, and need not be less than 1 in., 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.

### COLUMN FOOTINGS

The following stresses have to be considered in footing design :—

(a) **Bearing pressure** on ground which must not exceed the permissible pressure on the ground and this will decide the maximum area of the footing. Area of footing ( $a \times a$  for a square footing)

$$= \frac{\text{Load on column} + \text{weight of footing block}}{\text{Safe bearing pressure of soil}}$$

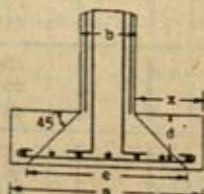
(b) **Punching shear** on footing. The footing slab or block must be thick enough to resist the tendency of the column to penetrate and punch a hole through the foundation block. The permissible punching shear stress on concrete is equal to twice the ordinary shear and is normally taken at 150 lbs. per sq. in. for 1 : 2 : 4 mix.

The area over which the punching shear would be 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  $t$  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.

(c) **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 in Table I. Total shear = unit upward soil pressure ( $W/a^2$ )  $\times (a^2 - e^2)$  and acts on the area  $4e \times 0.858d$ .

Therefore

$$d = \frac{W(a^2 - e^2)}{a^2 \times 4e \times .858 \times 75}$$



.058 is lever arm;  
75 is shear stress for 1 : 2 : 4.

(d) **Bending moment** due to the cantilever action of the footing projection  $x$ . (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.)

$$\text{BM.} = \frac{W}{8a}(a - b) \text{ on unit width of foundation block.}$$

Equate this with moment of resistance due to concrete  $= 137bd^2$  (for 1:2:4 mix, Table at page 8/26) 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.

### Safe Loads on Short R.C.C. Columns

for 1:2:4 mix with mild steel bars

Square Columns			Circular Columns					
Size in. (Out- side)	Vertical bars in.	Safe load in 1000 lbs.	Dia. in. (Out- side)	Vertical bars		Spiral rein- forcement		Safe load in 1000 lbs.
				No.	Dia. in.	Dia. in.	Pitch in.	
6×6	4-½	32	6	8	⅜	½	1½	43
8×8	4-½	52		8	½	½	1½	59
	4-1	93		8	⅝	½	1	75
10×10	4-⅝	81	8	8	⅝	½	1½	79
	4-1½	129		8	¾	½	1½	95
12×12	4-¾	117	10	8	¾	½	1½	139
	4-⅞	108		8	¾	½	1½	106
14×14	4-¾	156		8	1	½	1½	139
	4-1	148		8	1	½	1	184
16×16	4-1	208	12	8	1	½	1½	125
	4-⅞	196		12	1	½	1½	198
18×18	8-¾	256	15	8	⅞	½	1½	257
	4-1	249		8	1½	½	2½	189
20×20	8-¾	324		12	1½	½	2½	437
	8-⅞	302		8	⅞	½	1½	232
22×22	8-1	400	18	8	1½	½	2½	384
	8-⅞	374		12	1½	½	2½	648
24×24	8-1	455	21	8	⅞	½	1½	296
	8-⅞	429		8	1½	½	1½	510
Binders may be 10 gauge wire at 6 ins. spacings up to 12"×12"; ½ in. dia. at 12 ins. spacings up to 18"×18"; and 5/16 in. dia. at 12 ins. spacings for bigger sizes.			24	12	1½	½	2½	800
				8	⅞	½	1½	378
				8	1½	½	1½	636
				12	1½	½	2½	909



## Reinforcement

Area of steel will be (as usual) =  $\frac{BM}{18000 \times .858d}$

for unit width of block. 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.

To ensure 45 diameters bond length from the centre of the slab to either end of a bar, the length of the bar (hooked) must be  $2 \times 36$  dia. ins., *i.e.*, bar diameter must be at least: length of block/72 in. All the bars should be hooked at the ends.

## 4. WATER TANKS AND SMALL RESERVOIRS

(Reservoirs have also been described elsewhere; see Index.)

Ordinary 1:2:4 cement concrete does not generally make a water-proof construction, therefore, a richer mix of 1:1½:3 is usually prescribed. "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. For thick structures and mass work 1:2:4 mix may be used. Working stresses adopted are given in Tables I and II. 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 1 in. + 1/40 depth below top water level with a minimum value of 4 inches.

(ii) In wall and floor slabs there should be in each direction at right angles, not less than 0.3 per cent of reinforcement based on the gross cross-section.

(iii) The bars used should be of the smallest diameter possible. Laps in bars and minimum cover to reinforcement have been described before.

(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 ( $\frac{3}{8}$  in. or less) at fairly close spacing (say, 9 ins. to 12 ins. centres). Distribution rods placed outside the main rods are easier to fix and are more effective.

(v) Walls of tanks built below ground should be reinforced in both faces to resist the bending moment due to pressure of backing behind wall on the tank when empty and the water pressure neglecting pressure of backing.

(vi) The height of any layer of concrete shall not exceed 6 ft. unless precautions are taken to ensure thorough consolidation throughout the height of the layer. Superimposed layers shall be cast at as short intervals as are practicable.

(vii) 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 "Elevated Tanks"—"Core Walls".)

(viii) 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 : 1½ 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.



(ix) Structures may be lined with impervious materials such as asphalt, tiles, to prevent percolation or chemical action.

$$\text{Internal diameter of a circular tank} = \sqrt{\frac{\text{capacity in gallon}}{4.91 \text{ height}}}$$

Dia. in. ft.	Capacity in galls. per ft. of height	Dia. in. ft.	Capacity in galls. per ft. of height	Dia. in. ft.	Capacity in galls. per ft. of height
5	122	11	595	17	1420
6	176	12	730	18	1583
7	240	13	830	19	1770
8	314	14	953	20	1960
9	396	15	1100	25	3115
10	490	16	1247	30	4410

**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  $=w \times h$  lbs. per sq. ft. at the bottom of  $h$  ft. height. The total pressure in the curved surface (semi-circular) is the same as that in the diametrical plane which is  $=w \times h \times d$ . This pressure is resisted by the two sides of the ring, which is called hoop tension, ring tension or circumferential tension.

Circumferential tension  $T$  at any depth  $h$  in a horizontal ring of 1 ft. height :

$$T = \frac{w \times h \times D}{2} \quad \left| \begin{array}{l} w = \text{weight of water per c. ft.,} \\ D = \text{internal dia. of tank in ft.} \end{array} \right.$$

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 not 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. To keep a reasonable factor of safety against cracking and leakage,



only half the ultimate tensile stress of the concrete is considered to take the tension (see Table I). The maximum tensile stress developed on the face away from water should not exceed the ultimate tensile stress.

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.,

$$\frac{w \times h \times D}{2} = 110[12'' \times t + A_s \times (m-1)]$$

110 is the allowable stress in 1 : 2 : 4 concrete for cylindrical tanks ; 12'' is the height of wall considered under pressure;  $A_s$  is area of tensile steel in sq. ins. This may be taken :

$$t = \frac{hD}{45} \text{ for } 1 : 2 : 4 \text{ mix}$$

$$t = \frac{hD}{60} \text{ for } 1 : 1\frac{1}{2} : 3 \text{ mix}$$

$h$  and  $D$  are in ft.,  
and  $t$  is in inches

Minimum thickness  
prescribed is 4 inches.

Thickness may be reduced towards the top and walls made tapering, according to the height of water.

Cross-sectional area of steel to resist the circumferential tension per ft. depth :

$$A_s = \frac{w \times h \times D}{2 \times 12000} = \frac{hD}{385} \text{ (say)} \quad \left| \begin{array}{l} 12000 \text{ is allowable stress in} \\ \text{steel for ring tension.} \end{array} \right.$$

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, and partly on the inner and partly on the outer side in big tanks.

The walls of cylindrical 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. (See under "Rules for Distribution Reinforcement" and also "Practical Rules").

The above method of designing may be used for cylindrical tanks where the wall is not fixed to the floor slab and a sliding joint at the bottom of the wall is provided. 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.**

If wall and base of a tank are monolithic, in addition to the hoop tension due to the weight of the water, negative bending moment (inside) is produced by the restraint at the base of the wall for a short distance above the base and which changes to positive moment above. At the bottom of the wall the hoop stress is zero and the whole load is resisted in a vertical direction by cantilever action. At sections, a short distance above the floor, the load is resisted partly by hoop action and partly by cantilever action, and higher above the resistance is wholly due to hoop action. Fixed base gives an economical design, but it involves laborious calculations to work out the exact stresses. The following simple method based on Reissner's theory is suggested for practical purposes.

(i) Max. negative bending moment (called restraint moment) at the bottom of the tank wall :—

$$BM = a \times wh^3 \text{—ft. lbs. per ft. length of wall.}$$

From this the vertical reinforcement is worked out.

$$\text{Area of steel per ft. length} = \frac{BM}{ft \times j \times t}$$

where :

ft=permissible stress in steel for ring tension=12000;

j=lever arm factor=0.847 for 12000 and 680 concrete;

t=effective thickness of concrete wall. Thickness of wall

is assumed to arrive at the value of the factor  $a$ ;

$a$ —is a factor—value given in the table following.

Equivalent vertical reinforcement is provided on the water side of the wall for a distance of about  $\frac{1}{3}$ th 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  $\frac{1}{3}$ rd 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 :

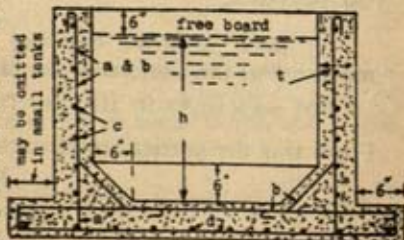
$$T = b \times \frac{w \times h \times D}{2} \text{—lbs.} \quad \left| \begin{array}{l} b \text{ is a factor, value given} \\ \text{in the table following.} \end{array} \right.$$

(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 maximum ring reinforcement per ft. height will be  $T/12000$  sq. ins. This may be reduced both above and below the max. hoop tension point.

**Corners and Edges.** If the base is not perfectly fixed with the wall the design will not be safe. A fillet (also called 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 6 ins.  $\times$  6 ins. to 9 ins.  $\times$  9 ins. as shown in the figure. Additional horizontal steel must be added at the corners in L shape through the walls and the 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√h, it is economical to design the walls as cantilever beams anchored to the floor.

FACTORS FOR CIRCULAR TANK DESIGN  
ACCORDING TO REISSNER'S THEORY

Factor	h/D	0.1	0.2	0.3	0.4	0.5	1	2	3
a	h/t=10	.075	.045	.030	.024	.020	.012	.006	.005
	h/t=20	.048	.028	.018	.014	.012	.008	.005	.003
	h/t=30	.035	.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 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. **Rectangular tanks with walls fixed at the base**

*Longitudinal* walls are designed as vertical cantilevers 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  $\frac{1}{3}$ th from the top which is :  $T = \frac{1}{3}h \times w \times \frac{1}{3}$  short walls length, for 1 ft. height of long wall. Area of steel required to resist this horizontal tension =  $T/12000$  sq. ins. per ft. 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 mid-span and a negative (inward) moment at each corner of  $wL^2/12$ . Maximum bending moment at any depth

$$h \text{ ft.} = \frac{w \times h \times L^2 \times 12}{12 \text{ or } 16} \text{ in lbs. per ft. of height.}$$

$w$  is weight of water, and  $L$  is effective span.

6-in. by 6-in. fillets should be provided to all corners to give added strength and extra horizontal reinforcement on the inside face for a distance of about 3 ft. in both the long and short walls. See also "Corners and Edges" under Circular Tanks.

For reservoirs up to about 12 ft. 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 counterforts at about 3 ft. 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 not less than 3 ins. thick shall first be spread over the ground and covered with a sliding layer of paper or other suitable material. Floors of tanks resting on ground are designed for the weight of walls and water 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.

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 5 to 8 ins. is common with reinforcement of  $\frac{3}{4}$  in. dia. bars at 6 to 9-in. 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 super-imposed layers of which the bottom layer may comprise or replace the mass-concrete screed described above.

Floor layers should be placed in "squares" or segments so that the joints are not more than 25 ft. apart in the case of reinforced slabs and not more than 15 ft. apart in the case of plain concrete slabs. 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 R.C. 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.

**Floor Slabs of Elevated 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 or sections as described above.

**Joints.** (See also under "Joints in Concrete Structures" in the following pages.) Joints should be provided in walls every 20 to 25 ft. intervals which should be with complete discontinuity in both reinforcement and concrete, with very



small initial gap between the faces. These are more of contraction joints in small structures than expansion joints which need be provided only 100 to 130 ft. 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 8 ins. wide and 14 or 16 gauge 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".  $\frac{1}{4}$ " dia. holes at 8-in. 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 concreting 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 2-in. 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 "Elevated 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

DATA FOR SMALL R. C. TANKS WITH 1:2:4 CONCRETE  
 Circular Tanks (See Fig. at page 8/52)

No.	Size of Tank		Thickness of concrete for walls and floor	Reinforcement		
				Walls		Floor
	Capacity in gallons	Inside dia.		Depth	Spacing of horizontal rods at bottom half depth (circular)	
						bars c
1	500	5'-6"	3'-6"	1" dia @ 6" c/c	1" dia. @ 8" c/c	1" dia. @ 9" c/c
2	750	6'-6"	4'-0"	" " " "	" " 8" "	" " "
3	1000	7'-0"	4'-6"	1" " 9" "	" " 8" "	" " 8" "
4	1500	8'-0"	5'-0"	" " 8" "	" " 6" "	" " 8" "
5	2000	9'-0"	5'-6"	" " 7" "	" " 6" "	" " 6" "
6	3000	10'-0"	6'-6"	" " 6" "	1" " 8" "	" " 7" "
7	5000	12'-0"	7'-6"	" " 5" "	" " 8" "	" " 6" "
8	7500	14'-0"	8'-6"	" " 4" "	" " 7" "	" " 6" "
9	10000	15'-0"	9'-6"	1" " 6" "	" " 6" "	" " 6" "
10	20000	20'-0"	10'-3"	" " 5" "	" " 6" "	" " 5" "

## Square Tanks

No.	Size of Tank		Thickness of concrete for walls and floor	Reinforcement		
				Walls		Floor
	Length & breadth inside	Depth		Spacing of horizontal rods	Spacing of vertical rods	Spacing of rods each way
				bars c	bars a & b	
1	5' × 5'	3'—6"	4"	$\frac{1}{2}$ " dia. @ 6" c/c	$\frac{1}{2}$ " dia. @ 6" c/c	$\frac{1}{2}$ " dia. @ 9" c/c
2	6' × 6'	3'—6"	4"	" " " "	" " 6"	" " " "
3	6½' × 6½'	4'—0"	4"	" " " "	" " 6"	" " 8"
4	7½' × 7½'	4'—6"	4½"	$\frac{1}{2}$ " " 9"	$\frac{1}{2}$ " " 9"	" " 7"
5	8' × 8'	5'—6"	4½"	" " " "	" " 6"	" " 6"
6	9' × 9'	6'—6"	5"	$\frac{1}{2}$ " " 7"	$\frac{1}{2}$ " " 7"	" " 7"
7	11' × 11'	7'—0"	5½"	" " 6½"	$\frac{1}{2}$ " " 8"	" " 6½"
8	13' × 13'	7'—6"	6"	" " 6"	" " 7"	" " 6"
9	14' × 14'	8'—6"	7"	$\frac{1}{2}$ " " 9"	" " 6"	" " 5"



by making the reservoir sufficiently heavy to resist upward pressure, or by providing underground drainage where site and soil conditions permit.

Notes for tank tables at pages 8/57 and 8/58.

(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  $\frac{1}{2}$  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) Tanks up to No. 5 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 B.M. at the centre of each such beam will be  $=WL/6$ . The slab should be designed two-way reinforced. Nos. 5 to 10 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.

### Elevated Tanks

The illustration shows general arrangements for an elevated tank and is a typical design for 50,000 gallons 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 5 ft. 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 3 ft. 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.

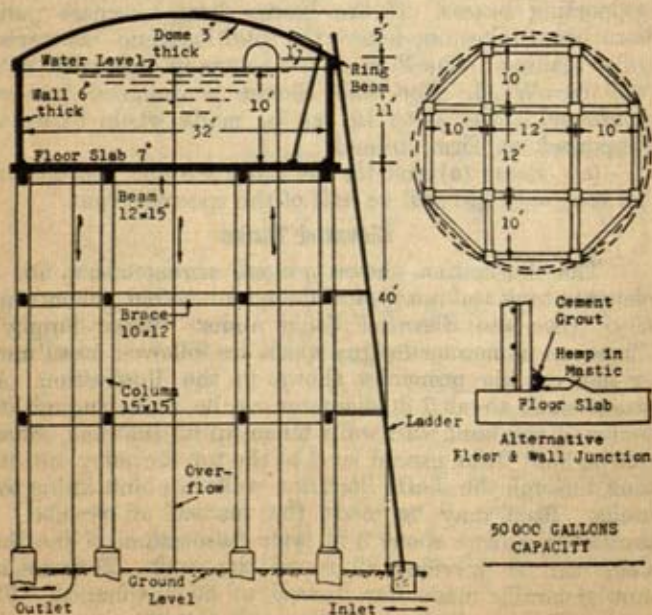
Foundations are designed according to the bearing capacity of the soil. If the soil is weak, raft foundations of R.C.C. can be made over the whole area. There should be no possibility left for any settlement.

**Data:**

Inside diameter	.. .. .	32 ft.
Depth of water..	.. .. .	10 ft.
Freeboard	.. .. .	1 ft.

(Freeboard may not be provided in all cases. If allowed, additional weight of this water should be taken.)

Height above ground level	.. .. .	40 ft.
Wall thickness	.. .. .	6 ins.
Floor slab : (for 10 ft. depth of water)		7 ins.



Central slab has the maximum stresses. Design for full water load as two-way reinforced slab fixed on all sides. (The thickness should be increased by 1' if 1:2:4 concrete



is used). Reinforce with  $\frac{1}{2}$ " dia. bars, 4" c/c both-ways. For side slabs reinforcement may be reduced.

Where floor slab is projected as cantilever, reinforcement at top for the cantilever portion will be necessary.

Roof (dome) thickness .. .. . 3"

Provide  $\frac{1}{2}$ " dia. bars 9" c/c radially and circumferentially. Binding wire can be No. 16 gauge.

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 .. .. . 9" x 12"

Provide 4 Nos.  $\frac{1}{2}$ " dia. bars and  $\frac{1}{2}$ " dia. stirrups at 9" c/c.

Beams : Central beams 12 ft. span .. .. . 12" x 15"  
(rib portion)\*

Design central beams as continuous with WL/12; for this size of beam reinforcement consisting of 4 bars  $1\frac{1}{2}$ " 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 12 ft. span reinforcement of 4 bars 1" dia. at top and bottom is required; while for outside diagonal beams, 4 bars  $\frac{3}{4}$ " dia. at bottom and 2 bars at top are necessary.

Columns .. .. . 15" x 15"

Reinforce with 8 bars  $\frac{1}{2}$ " dia. and  $\frac{1}{4}$ " helical binders at 3" c/c, as these columns are subjected to bending in addition to vertical load.

Bracings .. .. . 10" x 12"

Reinforce with 4 bars of  $\frac{3}{4}$ " dia. at top and 4 bars at bottom and  $\frac{1}{4}$ " dia. stirrups at 9" 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 joining members.

\*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.



*Ladder-Iron*

Can be made of the following size—

Sides—	..	..	..	Plates $2\frac{1}{2}" \times \frac{3}{4}"$
Rungs—	..	..	18" wide—	$\frac{3}{4}"$ dia.

Another arrangement for overhead tanks in hot climates can be with "core wall", that is,  $4\frac{1}{2}"$  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 R. C. 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.

## 5. MISCELLANEOUS STRUCTURES

**Design of Staircases**

Staircases in general have been described in the Section on "Masonry Structures". 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 60 lbs./sq. ft. for residential and office buildings and 100 lbs./sq. ft. for public places and warehouses, measured horizontally, whichever is more. The corresponding alternative minimum imposed loads as prescribed in Section 6 need not be considered (as recommended in the Codes). Some engineers, however, recommend to take  $\frac{1}{4}$  ton as the minimum load for design to safeguard against local overloading due to concentrated loads which might occur in practice. To the imposed load, the dead load of the structure is added, acting at right angles to the flight.

Stairs may be supported (and designed as such) either (i) transversally (parallel to nosing) on the two side walls or on stringer beams; or (ii) longitudinally (parallel to flight). 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. L is the horizontal distance from centre to centre of the supporting beams; where landings are continuations of the "waste" slab and no transverse sup-

porting beams are provided, L is from centre to centre of the landings, or centre of the landing to the end of the staircase where only one landing is provided. Some engineers take L from end to end of the landings. The end of each step going into the wall should be rectangular.

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 longitudinal stringer beams are more economical than the longitudinal slabs.

Steps are sometimes cantilevered with one end fixed into the wall where the wall is of ample thickness. Where the steps ("waist" slab) are supported transversely, the thickness of the "waist" may be 2 ins. up to  $3\frac{1}{2}$  ft. wide stairs,  $2\frac{1}{2}$  ins. up to  $4\frac{1}{2}$  ft. and 3 ins. up to 6 ft. wide stairs with ordinary loads. Reinforcement may be  $\frac{3}{8}$ " dia. bars 6 ins. apart laid across and distribution bars  $\frac{1}{4}$ " dia. laid 9 ins. apart parallel to the supporting beams.

The following sizes of reinforced concrete "wastes" may be taken for the stairs supported longitudinally in residential, office and light warehouse buildings:—

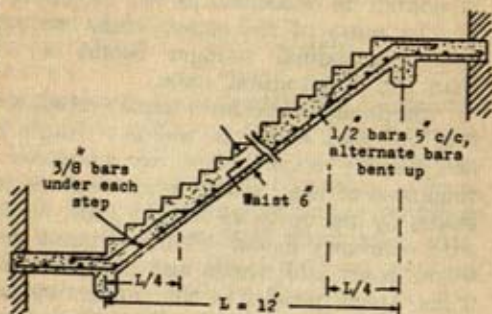
Span L in ft.	Thickness of "waste"	Spacing of reinforcement bars in inches	
		Main-longitudinal	Distribution-transverse
6'	4"	@ 6" c/c	@ 12" c/c
7'	4"	@ 5" c/c	@ 10" c/c
8'	$4\frac{1}{2}$ "	@ 4" c/c	@ 9" c/c
9'	5"	@ 6" c/c	@ 7" c/c
10'	5"	@ $5\frac{1}{2}$ " c/c	@ 6" c/c
11'	$5\frac{1}{2}$ "	@ 5" c/c	@ 12" c/c
12'	6"	@ 5" c/c	@ 12" c/c
13'	$6\frac{1}{2}$ "	@ $5\frac{1}{2}$ " c/c	@ 10" c/c
14'	7"	@ 5" c/c	@ 9" c/c
15'	8"	@ $4\frac{1}{2}$ " c/c	@ 7" c/c
16'	$8\frac{1}{2}$ "	@ 4" c/c	@ 6" c/c
17'	9"	@ 4" c/c	@ 5" c/c
18'	10"	@ $3\frac{1}{2}$ " c/c	@ 5" c/c
19'	$10\frac{1}{2}$ "	@ 3" c/c	@ 5" c/c
20'	11"	@ 3" c/c	@ 5" c/c



Sometimes pre-cast steps consisting of a tread only (simply a slab without riser) are used which are supported in stringer beams about 12 ins. high. Each step slab is  $12\frac{1}{2}$  ins. wide and is reinforced with 3 longitudinal bars  $\frac{1}{4}$ " dia. Each upper step projects about  $1\frac{1}{2}$ " over the end of the lower step. Stringer beams are designed as ordinary rectangular beams with half the load of the steps, as explained above for the "waist" slab—taking horizontal span.

Half the bars should be bent up at the ends of the slab for the negative bending moments.

Balustrades of stairs may be designed as cantilevers with a horizontal load of about 25 lbs. per linear foot for residential buildings and 50 lbs. for stairs to be used by crowds. These



loads will be taken as acting at the top of the balustrades.

It has been stated earlier that the "pitch" or slope of a staircase should not be more than 42 degrees; a 30 deg. pitch is usual. Treads commonly adopted are 9 ins. to 12 ins. with risers of 6 ins. to  $7\frac{1}{2}$  ins. For concrete steps, the normally adopted tread width is  $10\frac{1}{2}$  ins., with a riser height of 7". The following proportions may be taken:—

Tread, ins.	9	10	11	12	13	14	15
Riser, ins.	7 to $7\frac{1}{2}$	$6\frac{1}{2}$ to 7	6 to $6\frac{1}{2}$	$5\frac{1}{2}$ to 6	5 to $5\frac{1}{2}$	$4\frac{1}{2}$ to 5	4 to $4\frac{1}{2}$

### Cement Concrete Blocks

Concrete blocks may be made of dense or light weight concrete and may be either solid or hollow. Blocks 4 ins. or more in thickness are often cast hollow.



Hollow concrete blocks shall comply with the following regulations :—

(a) The sides of each block shall be not less than  $2\frac{1}{2}$  ins. in thickness in any part. In hollow blocks the size of the the cavity is governed by the general limitations that the volume of concrete in any block shall not be less than half the gross volume of the block, and that the total width of the cavities shall be not less than two-thirds of the overall thickness of the block at any point.

(b) Concrete blocks may be of any size but some standard sizes should be used so that they can be bonded with bricks if necessary. 18 in.  $\times$  9 in.  $\times$  9 in., or 18 in.  $\times$  9 in.  $\times$  6 in. sizes have been found satisfactory. A length/height ratio of 3:1 is probably the most desirable from the aspect of wall strength. Blocks are generally referred to by their nominal dimensions which include the block and an allowance for joints.

(c) Steel wires may be embedded in each block.

(d) During the process of manufacture each block shall be subjected to a pressure of not less than 1000 lbs./sq. in.

(e) No hollow block shall be used in any position where the max: pressure on it will exceed 6 tons/sq. ft.

(f) The blocks shall be able to withstand a test load of 25 tons/sq. ft.

(g) Where walls are exposed to the weather, 50 ft. should be the max: desirable length without an expansion joint. Where walls are not exposed to the weather, they may be made 100 ft. long.

(h) The course immediately below each floor shall be built of solid blocks.

(i) Blocks  $2\frac{1}{2}$  ins. thick should weight 6.25 lbs./sq. in. for each inch of overall thickness of block as laid.

(j) Blocks should be thoroughly cured and dried out before placing.

(k) Introduce wall reinforcement in horizontal joints at points of local weakness. It is desirable to reinforce one course above and one course below window and door openings, to a point at least 2 ft. beyond the jambs of such openings.

(l) Where walls are to be later rendered, a vertical joint should be left open in each course of masonry, say

every 8 or 10 ft., and staggered between courses.

Hollow blocks are manufactured in special machines. Dense concrete is made in the 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 should generally be used.

Hollow blocks have better thermal properties than solid blocks of the same material and total thickness. Light weight concrete provide 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 movements 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  $\frac{1}{4}$  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.

#### **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.

#### **Fence Posts**

Ordinary line posts may be fixed 9 ft. apart unless steel droppers are fixed to the fencing wires, usually at 6 ft. intervals, to keep the wires the correct distance apart. If the fencing wires are pulled sufficiently tight the use of



droppers enables the line posts to be spaced as much as 24 ft. apart without the wires sagging unduly. 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." Thus, for a fence with posts to stand 4 ft. high above ground level the line posts should be 6 ft. long, and straining posts at least 7 ft. long. Where the ground is soft it is a wise precaution to set main posts in concrete bases.

Posts are reinforced with four rods, two rods, or a single rod. In the four-rod reinforced post, a rod is placed near each corner, and the further apart the rods are the stronger is the post. It is very important, however, that the reinforcement be protected by adequate thickness of dense concrete, otherwise the moisture may penetrate and rust the reinforcement. In factory made posts the reinforcement is usually covered with a thickness of  $\frac{3}{4}$  in. of concrete and for "home-made" it should be at least 1 in. In two-rod reinforced posts, a simplification is to bend a single rod in the form of a hair pin, with the legs firmly wired together in two or three places with No. 16 gauge wire to prevent springing. With this type of reinforcement give concrete covering of at least 1 in.

Where posts are not required to have holes through them for wires, single rod reinforced posts should be preferred for small sizes. The rod is placed in the centre.

Usual Dimensions of Factory Made Fence Posts

Type	Line posts		
	Total length	Bottom size	Top size
1	4'-6"	3" x 4"	3" x 4"
2	6'-0"	4" x 4"	2½" x 2½"
3	6'-6"	4" x 5"	4" x 5"
4	6'-6"	5" x 5"	2½" x 2½"

Straining Posts

1	5'-3"	4" x 4"	4" x 4"
2	7'-0"	5" x 5"	4" x 4"
3	7'-6"	5" x 5"	5" x 5"
4	7'-6"	5" x 5"	4" x 4"



Stays are generally of the same size as straining posts.

Stays should have base blocks of size about  $1'-6" \times 9" \times 5"$  for anchorage.

For "home-made" posts increase the size by 1 in. on each size.

#### Reinforcements for Factory made Posts

Size of post	$3" \times 3"$ or less	$4" \times 4"$	$5" \times 5"$	$7" \times 7"$	$8" \times 8"$
Reinforcement rods of dia.	$4-\frac{1}{4}"$	$4-\frac{1}{4}"$	$4-5/16"$	$4-\frac{1}{4}"$	$4-\frac{1}{4}"$

#### Single Rod Reinforcement

Size of post	$4\frac{1}{2}" \times 4\frac{1}{2}"$	$5" \times 5"$	$5\frac{1}{2}" \times 5\frac{1}{2}"$
Reinforcement rod of dia.	$1-\frac{1}{4}"$	$1-\frac{1}{8}"$	$1-\frac{1}{4}"$

**Note :—**For a post that tapers to a smaller section at the top the correct reinforcement for the whole post is that suitable for the sectional dimensions of the bottom.

#### Poles

**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  $4\frac{1}{2}$  in.  $\times$   $4\frac{1}{2}$  in. to 5 in.  $\times$  5 in., with a taper of 0.2 in. per foot length. A tapering central duct of diameter not less than  $1\frac{1}{2}$  in. at the top is left for taking the supply from the base to the fitting at the top and for also reducing the weight of the pole. Perforations are also sometimes made to reduce the

weight and cost. The concrete used may be of 1:2:4 mix where the poles are mechanically vibrated or spun, and of 1:1½:3 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 1 inch. Suitable apertures are provided in the poles 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 1½ 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.

The quantum of the thrust as prescribed by the B.S.S. for different classes of poles for transmission up to and including 11 kV is given in the following table :

Class of pole	Max: over-all length	Min : ultimate transverse load at 2 ft. from top for causing pole failure	
1	34	625	Ultimate load is working load with a factor of safety of 3.
2	36	875	
3	40	1250	
4	46	1750	
5	50	2500	
6	50	3500	

### 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 5 to 6 ft.

### Cantilevers, Balconies & Canopies

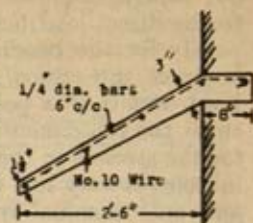
Thickness of a cantilever slab at the support, as stated earlier at page 8/20, is—span/7.5. Tensile reinforcement is provided at the top. End of the cantilever slab in the wall

can be anchored down by means of iron rods embedded in the supporting wall or pillars, where there is not sufficient weight of wall over the end. Sufficient anchorage to counter-balance the weight of the projection is essential. In cantilever slabs bars of larger diameter should be used than usual, and which may not be less than  $\frac{1}{2}$  in. dia. if adequate, as less stiff bars are apt to be bent while concreting.

### R. C. Sunshades

No. 10 wire is provided at 12-in. centres, at right angles to the main bars. (Dotted line in the illustration indicate main bars.)

Minimum head-room from top of pavement is 7 ft.-6 ins.



R. C. SUN SHADE

### Reinforced Concrete Walls

Where reinforced concrete walls are intended to carry vertical loads, they should be designed generally according to the rules given for the design of columns. Strength calculations should be made if the wall panel exceeds 15 ft. between floors or 20 ft. between columns. Panel and enclosure walls of buildings should have a thickness of not less than 4 inches and which should not be less than  $\frac{1}{30}$ th the distance between the supporting or enclosing members. Bearing walls should have a thickness of at least  $\frac{1}{25}$ th of the unsupported height or width, whichever is the shorter. Exterior basement walls, foundation walls, and fire walls should not be less than 8 inches thick whether reinforced or not.

**Reinforcement.** The cross-sectional area of the vertical reinforcement should not be less than 0.4 per cent and the lateral reinforcement parallel to the wall face not less than 0.2 per cent, for load bearing walls. This reinforcement may be halved for non-bearing walls. The diameter of a vertical reinforcement rod should be not less than  $\frac{3}{8}$  in., and the distance between two vertical reinforce-



ments and the distance between two lateral reinforcements should not exceed 12 inches—(18 ins. max.). In addition to the reinforcement prescribed above, there should be not less than two  $\frac{5}{8}$  in. dia. bars around all window or door openings. Such bars should extend at least 24 ins. beyond the corners of the openings. Walls more than 10 ins. in thickness should have the reinforcement for each direction placed in two layers parallel with the faces of the wall.

Walls should be anchored to the floors, columns, or intersecting walls with reinforcement of at least  $\frac{3}{8}$  in. bar 12-in. 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 the wall is cast in 2 ft. or 3 ft. 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 6-in. layers and each layer thoroughly punned before the next is shovelled in. Each lift of 2 or 3 ft. 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 as 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.

## 6. PHYSICAL PROPERTIES OF CONCRETE

The principal properties of concrete include :

Workability; Strength; Durability; Impermeability and Volume changes.

**Workability** is that property of a concrete which determines the ease with which it can be placed in position and compacted, i.e., worked. A workable concrete is one which can be compacted with a minimum of labour, and an unworkable or harsh concrete is one which requires an inordinate amount of work to compact fully. The degree of workability required is determined by the nature of the work and the method of compaction that is to be used. In order to obtain concrete of maximum strength and durability, 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. Concrete that is to be placed in narrow forms congested with reinforcement will require a much higher degree of workability than that for unreinforced mass concrete. Concrete that is to be compacted by mechanical vibration may be much drier, and hence less workable, than that which is to be tamped by hand. The principal factors which effect the workability of concrete are :—

(i) *Water content.* The workability increases as the water content of the mix is increased. But, as will be explained later, the strength of a concrete depends upon its water content, the quantity of water has to be restricted within certain minimum limits as increase in water content would cause a decrease in strength. Increase in workability without impairing strength can be achieved by increasing cement content.

(ii) *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.

(iii) *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.)

(iv) *Cement content.* The higher the cement content 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).

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. As will be seen later, the best mix is one which gives the maximum workability with the minimum amount of water.

**Tests for Workability of Concrete.** There is not, at present, any reliable field test but two tests are, however, generally employed for measuring the workability or consistency of a concrete mix: The "slump test" which is a field test, and the "compacting factor test" which is a laboratory test.

**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 of cement-water paste occurs in mixes with high water content, and segregation of coarse aggregate occurs with lean dry mixes. Cement paste is displaced by the coarse aggregate and rises to the top of the concrete to form a layer of mortar (laitance). 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.

#### **Water/Cement Ratio**

Is the ratio of the weight of water in a mix (exclusive of that absorbed by the aggregate) to the weight of cement



therein, and is the most important factor for the strength of a concrete. Ultimate crushing strength of a fully compacted concrete depends primarily on the water/cement ratio. 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.

Excess of water weakens a concrete, produces shrinkage cracks (shrinkage increases with increase in water content) and decreases density. Water occupies space in concrete and as it evaporates, it leaves voids. 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 50 per cent of water may reduce the strength by half. Concrete should be just plastic enough to be worked around the reinforcement rods.

The correct quantity of water required for a particular mix depends upon various factors such as : mix proportions, type and grading of aggregate, method of compaction applied, and weather conditions. Therefore, there is an optimum value of the water/cement ratio for every mix. 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, as the optimum quantity of water does not produce concrete of sufficiently fluid consistency to pass through narrow spaces between the reinforcing rods. Accurate control of the water/cement ratio requires the use of a laboratory.

**Bleeding** is the term given to the formation of a layer of water on the upper surface of concrete after compaction. This is called *laitance* which is a watery "scum" and is due to excess of water in the mix or deficiency of fine material. This layer lacks strength and resistance to abrasion and increases shrinkage. Bleeding gives rise to weak joints between successive lifts in structural work. *Laitance* is also formed with too much floating or trowelling. If *laitance* is not removed the upper portion of the concrete

will be porous. An increase in water content must be accompanied by a proportionate increase of cement if strength is to be maintained. Bleeding can generally 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. For a normally well graded aggregate and for a slump of 3-ins., the approximate quantity of water required for 100 c. ft. of concrete for various maximum sizes of coarse aggregate is given below :-

Max. size of aggregate in inches	$\frac{1}{4}$ "	$\frac{1}{2}$ "	$1\frac{1}{2}$ "	3"
Quantity of water in galls. per 100 c. ft. of concrete	145	124	113	102

The quantity of water indicated above will have to be increased or decreased by about 3 per cent for each increase or decrease of 1 inch in slump.

Concrete that is to be compacted by mechanical vibration may be much drier, and hence less workable, than that which is to be tamped by hand. Such a concrete needs about 20 per cent less water and about 15 per cent less



cement. A mechanically compacted concrete gives more strength and density for the same materials.

Rapid hardening cement needs about one gallon more water per cwt. than the ordinary cement.

Quantity of water required can be worked out roughly taking 30 per cent by weight of cement plus 5 per cent by weight of aggregate, for hand compaction. Plain concrete may need about half gallon more and mass concrete about one gallon more of water per cwt. of cement for very dry conditions. Structures in contact with water should be made of drier mixes. The following quantities of water may be taken for dry materials with uncrushed gravel aggregates :—

Nominal mix.	1:1:2	1:1½:3	1:2:4	1:3:6
Galls. of water per cwt. of cement	4½	5½	6½	8
Water/cement ratio (by weight)	0.43	0.51	0.58	0.72

Water/cement ratio is usually expressed as a decimal fraction, i.e., weight of water/weight of cement, but for practical purposes it is usually expressed as so many gallons of water to a bag (or cwt.) of cement. To convert water/cement ratio (by weight) to gallons per cwt., multiply by 11.2.

Water/cement Ratio

Galls. per cwt.	4	4½	5	5½	6	6½	7	7½	8
By weight ...	0.36	0.40	0.45	0.49	0.54	0.58	0.63	0.67	0.72
	Mix too dry for hand compaction.				Mix workable for hand compaction.				

In general, mechanical compaction is necessary for water/cement ratio of less than 0.50 in a nominal 1:2:4 mix.

**Hydration of Cement.** When water is added to cement, the cement hydrates, calcium hydroxide or hydrated lime is liberated. During the chemical reactions which take 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.

### **Test for Workability or Consistency of Freshly Mixed Concrete**

#### **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. 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. The test is useless for lean dry mixes as the slump recorded is very small. All aggregates of size 2-ins. 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 4 ins., the bottom diameter 8 ins., and the height 12 ins., open at both ends and fitted with handles and foot pieces on sides. The cone is placed on a smooth surface and the freshly mixed concrete is placed in the mould in four successive layers, each layer being rodded 25 times with a bullet-pointed rod  $\frac{3}{8}$  in. in diameter and 24 ins. long. When filled to top (after ramming) 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. "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. Slump is measured to the nearest  $\frac{1}{4}$  in. The mould should be placed by the side of the concrete heap after removal for measurement.

### Recommended Values for Slump in Inches

Type of work	With vibrations	Without vibrations
Mass concrete, large sections, roads.	0 to 1	1 to 3
Foundations, footings, sub-structures., walls and other heavy sections.	1 to $2\frac{1}{2}$	$1\frac{1}{2}$ to $4\frac{1}{2}$
Thin sections such as slabs, beams, columns, with congested reinforcement.	$1\frac{1}{2}$ to 3	4 to 7

A concrete with 0 to 1 inch slump has very low degree of workability, and with 4 to 7 inches 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 the cement, (iii) the degree of compaction obtained in the concrete, (iv) curing, and (v) the age of the concrete. The water/cement ratio is the most important factor and the ultimate crushing strength of a fully compacted concrete depends principally on it. The strength increases as the water/cement ratio is decreased and 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. Richer mixes shrink more than leaner mixes. 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.

**Durability.** The durability of concrete, especially its resistance to attack by frost or chemicals, depends largely on the proportion of voids and the permeability of the concrete. The quantity of mixing water and good compaction are the most influential factors. Excess water forms voids and enable the destructive agencies to take effect, therefore, the mixing water must be reduced to the minimum consistent with good compaction.

**Load Tests on Built Structures.** If the strength of a structure is doubtful, a test load of  $1\frac{1}{2}$  times the superimposed 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

### 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). Access 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 cleanliness, is to make two identical pats of size 3 ins. dia. and  $\frac{1}{2}$  in. thick of neat cement paste, one with the water under test and the other with water of known suitability. 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; firstly, to hydrate the cement and secondly, to lubricate the mix so as to aid compaction.

## CEMENTS

**Manufacture of Cement.** Cement is made by intimately mixing together chalk or limestone and clay 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.** Cements are classified by their properties and chemical composition. Cements used for engineering works are :—

**Normal Setting Portland Cement.** This is the type in common use for general works ; ordinary cement.

**Rapid-hardening Portland Cement.** Also known as "high early strength cement." This cement has the same composition as ordinary cement but is ground more finely and is used where high early strength is required. It sets and hardens in a much shorter time than the ordinary 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 ordinary 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 ordinary cement in 28 days. It is comparatively costly. The setting time is about the same as of the ordinary cement. This cement is useful for repair work.

**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. (Difference has been explained further.)



**High-Alumina Cement.** This cement differs radically both in composition and properties from the normal Portland cement. It has high alumina content (over 35 per cent as against under 10 per cent in normal Portland cement). The initial setting of this cement does not take place till after 2 hours and final setting 2 hours after the initial setting. It hardens much more rapidly than Portland cement and develops strength very early; up to 75 per cent of its ultimate strength being attained during the first 24 hours after mixing. A concrete made with this cement becomes as strong in about a day as a full matured concrete made with ordinary cement.

High alumina cement has a great resistance to heat, is immune from attack by magnesium salts and sulphates, and is unaffected by the corrosive action of acids and is therefore particularly useful for refractory works and works subject to attack by sea water, chemicals, etc., The cement gives out great heat during setting for which special precautions are necessary during curing, and cannot be used for massive structures. Mixes richer than 1:2:4 should not be employed unless special precautions are taken to dissipate the heat generated. This cement may be unsuitable for use with certain aggregates which may liberate appreciable amounts of soluble alkali or lime. Owing to its high cost (between two and three times of ordinary cement), this cement is used only where it is out-weighed by the advantages gained or where there are no practicable alternatives. No water-proofers are necessary with this cement. Permissible stresses in concrete using high alumina cement should be decided on the basis of the results of preliminary tests.

Aluminous cements must not be mixed with other cements or mortars (lime) or be placed against Portland cement concrete less than 7 days old, neither must Portland cement be placed against high alumina concrete which is less than 24 hours old. This cement is useful for emergency repair works.

**Low-heat Portland Cement** is a cement in which the rise in temperature on setting is less than that of the ordinary cement. This cement is used for works where it is necessary to restrict heat generation (due to hydration



of cement) during concreting, to avoid cracking, in large masses of concrete such as dams, bridge-abutments and retaining walls. The rate of development of strength is somewhat lower than for ordinary 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.)

**White Portland Cement** is made of chalk or a pure carboniferous limestone and china clay. The whiteness is due to the absence of impurities (iron oxide) which impart colour to the ordinary cement. It is about 4 to 6 times more costly than ordinary cement, but has the same properties.

**Coloured Cements** are made by adding suitable mineral pigments. White and coloured cements are used for decorative purposes.

Pats or briquettes made from ordinary Portland cement, when broken, exhibit a bluish grey colour in the fracture.

**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.** 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 are 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 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 :

Ordinary cement—Not less than 300 lbs./sq. in. after 3 days and greater than 375 lbs./sq. in. at 7 days.

Rapid hardening cement—Not less than 300 lbs./sq. in. after 1 day and not less than 450 lbs./sq. in. after 3 days.

#### **Compressive strength**

Ordinary cement—Not less than 1600 lbs./sq. in. after 3 days and not less than 2500 lbs./sq. in. after 7 days.

Rapid hardening cement—Not less than 1600 lbs./sq. in. after 1 day and not less than 3500 lbs./sq. in. after 3 days.

**Test for Setting Time.** Make a stiff paste of neat cement and water and form it into a pat about 3 ins. diameter and  $\frac{1}{2}$  to 1 in. 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 as to make it impossible to scratch the surface with the thumb-nail. It should be difficult to break 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 increases 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. It sometimes happens that cement becomes quicker setting with age and storage.

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 at the end of one year. (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 worthless 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 becomes hydrated and loses strength. 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 exceeds 5 per cent, the cement is for all ordinary purpose, 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 6 ft. 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 2 ins. 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.

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 following figures show the average reduction of strength in a mixture of 1 part of cement to 5 parts of aggregate (1:2:4 mix.) as a result of storage:—

Cement fresh		strength	100 p.c.
Cement after 3 months storage,		strength reduced by	20 p.c.
"        6    "	"	"	30 "
"       12    "	"	"	40 "
"       24    "	"	"	50 "

Ordinary Portland cement which has been stored over six months, and rapid hardening cement which has been stored over two months, from the time of leaving the factory, should always be tested before use. If deteriora-

tion is suspected the test for "setting time" described above 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 should feel like an impalpable powder such as flour; there should be no grittiness. Any caked or lumpy cement should be summarily rejected.

**Storage of Cement.** Cement can be safely stored in sacks for a few months if kept in a dry and air-tight room. If prolonged storage of cement is unavoidable, 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 6 to 9 ins. above the floor level and 12 ins. away from walls. Bags to be stacked in not more than 10 layers high (max: 15 ft.) 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. A bag of cement is considered to occupy  $1\frac{1}{2}$  c. ft. and not less than  $3\frac{1}{4}$  sq. ft. of floor space.

**Weight of Cement.** Normal Portland cement weighs 75 to 100 lbs. per c.ft. when loosely packed and 110 lbs. per c.ft. when well compacted and its weight per c.ft. varies between these two limits depending upon the degree of compaction. Therefore, the method of measurement by volume is inaccurate. Cement should be measured by weight. 1 cwt. bag of cement is taken to contain 1.22 c.ft. The weight of a gunny bag is about 20 oz. and that of a paper bag about 14 oz. Net cement in a cwt. bag may be



only 109 or 110 lbs. The permissible tolerance on the weight of cement supplied in bags is  $2\frac{1}{2}$  per cent.

For estimating purposes cement is taken to weigh 90 lbs. per c.ft. Rapid hardening cement is taken to weigh 80 lbs. per c.ft. and 1.4 c.ft. to a cwt. bag. (A sack of cement is 112 lbs. in Britain and 94 lbs. in America.)

As cements manufactured at different works are not of a uniform quality, use cement from one mill only for one job whenever possible.

### STEEL—REINFORCEMENT

The steel used for reinforcement is either mild steel or high tensile steel, the former is more common. Reinforcement is commonly made in three forms, viz., (i) plain bars, (ii) deformed bars, and (iii) fabric.

Plain bars may be either round or square, the former being the more common. Common standard diameters are:  $\frac{1}{4}$ ",  $\frac{5}{16}$ ",  $\frac{3}{8}$ ",  $\frac{1}{2}$ ",  $\frac{5}{8}$ ",  $\frac{3}{4}$ ", 1",  $1\frac{1}{4}$ ", and  $1\frac{1}{2}$ ". (Areas and weights of bars are given in tabular forms at the end of this Section.) Sizes smaller than  $\frac{1}{4}$ " dia. (sometimes smaller than  $\frac{3}{16}$ ") are known as wires and are manufactured in standard wire gauge sizes. Mild steel rods for reinforcement should be of some standard manufacture and of adequate strength. Re-rolled rods are not generally of full strength.

Deformed and twisted bars have higher initial cost than that of plain bars but a lesser weight of steel is required as these bars have higher tensile strength and yield point. Deforming the bars also increases the bond between steel and concrete. These bars are manufactured in a variety of proprietary shapes.

Fabric are used mainly for slab and wall reinforcements. There are a number of proprietary patterns with different strength properties. Expanded metal (XPM) formed from sheet steel is also used.

All reinforcement should be free from loose mill scale, loose or scaly rust, oil and grease, immediately before placing the concrete. A thin rust 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 separately. Loose rust can be cleaned with a wire brush or hessian cloth.

**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 bars 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.

**Placing Reinforcement.** To ensure that reinforcement is correctly placed in position and is not moved during concreting, secure 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 will rust, expand and eventually break up the concrete. Cover blocks or packing pieces can be made of concrete of size  $\frac{1}{2}'' \times 1'' \times 1\frac{1}{2}''$ . 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 an inch 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 end towards the hooked end. Use a steel tape for measuring bars. The dimensions given should follow the convention of measuring from "outside to outside" except where shown otherwise. Mark out the first bar according to the given dimensions and check it after bending. Base the dimensions of subsequent bars on this first one, making alterations where necessary. This is particularly important where a bar has a number of bends.

## 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 purpose required. Stones absorbing more than 10 p.c. 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 varieties of granite, quartzite, trap and basalt, and those with rough non-glossy surface are considered best. All sand-stones tend to be porous. Soft varieties of sand-stones make poor concretes and also produce shrinkage cracks. Limestone is quite good provided it is hard, crystalline and entirely free from dust. Limestones should not be used in works subject to excessive heat. Both lime and sandstones 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 reduces the strength of the concrete.

The material above  $\frac{3}{8}$ " size is generally classified as "coarse aggregate" and below that size as "fine aggregate" or "sand". B.S. sieves usually adopted for grading of concrete are: For coarse aggregate— $1\frac{1}{2}$ ",  $\frac{3}{4}$ ",  $\frac{3}{8}$ " and  $\frac{3}{16}$ "; For fine aggregate— $\frac{3}{16}$ ", sieve Nos. 7, 14, 25, 52 and 100.

Coarse aggregate should be ordered in separate sizes and recombined in the proper proportions while batching. A  $1\frac{1}{2}$ " nominal maximum size aggregate will be ordered in three sizes,  $1\frac{1}{2}$ " to  $\frac{3}{4}$ ",  $\frac{3}{4}$ " to  $\frac{3}{8}$ ", and  $\frac{3}{8}$ " to  $\frac{3}{16}$ ". A  $\frac{3}{4}$ " nominal maximum size will be ordered in two sizes,  $\frac{3}{4}$ " to  $\frac{3}{8}$ " and  $\frac{3}{8}$ " to  $\frac{3}{16}$ ". Separate stockpiles should be maintained for the different sizes.

### 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 classified 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.

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-bajree) require one c.ft. of less cement and about two c.ft. of less sand, per 100 c.ft. of concrete and about one-third of a gallon less 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 structures.



**Maximum Aggregate Size.** The maximum size of aggregate is governed by the nature of the work. The maximum size of aggregate may be up to 6 ins. for mass concrete, but size up to 9 ins. has also 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 3 ins. Large stones which are embedded in mass concrete works are called "plums". Plums should be sound and hard and should not be placed nearer than 6 ins. to one another or to an exposed surface.

To obtain high crushing strength, the maximum size of aggregate should be as large as conveniently possible, 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  $\frac{1}{4}$  in. less than the minimum distance between the reinforcement bars or 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.

**Maximum size of Aggregate Recommended for Various Types of Construction (Based on American Concrete Institute) :—**

Min: size of section	2½" to 5"	6" to 11"	12" to 29"	30" or over
Reinforced walls, beams and columns	½" to ¾"	¾" to 1½"	1½" to 3"	1½" to 6"
Unreinforced walls or mass concrete	¾" to 1"	1½" to 2"	3" to 6"	6"
Heavily reinforced slabs	¾" to 1"	1½"	1½" to 3"	3" to 6"
Lightly reinforced or unreinforced slabs	¾" to 1½"	1½" to 3"	3"	3" to 6"

For most of the common reinforced concrete works, a maximum size of  $\frac{3}{4}$  in. for coarse aggregates is generally suitable. For heavy sections a maximum size of  $1\frac{1}{2}$  ins. and for mass concrete works size up to 3 ins. may be used. For thin members such as ribs and top slabs, the largest size of aggregate is generally  $\frac{3}{8}$  in.

### Broken Brick Aggregate

This is often quite a good material but great care should be exercised in choosing it as some bricks contain sulphur and unslaked lime, others have been found to possess qualities which render them unsuitable for concrete; where they are to be employed only a hard-burned variety should be selected. Brick aggregate should be saturated with water before use to avoid the absorption of the mixing water which is necessary for the setting and hardening of the concrete.

Coarse aggregate of porous nature with a percentage increase of over 10 per cent 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 constructions.

### 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 2" thick with a slope of not less than 1 in 48; should have another layer of about 1½" waterproof cement concrete over the top (as cinder concrete is not waterproof) with a layer of waterproof paper in between.

### Storing or Stockpiling of Aggregate

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 in 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 which should be flat-topped and not conical. Material should not be thrown from a height as this will also result in segregation by the winds. Piles of different sizes of coarse aggregate and of sand should be kept separate. On large jobs it is preferable to make separate compartments for the various sizes.



## FINE AGGREGATE OR SAND

Sand for concrete works is defined as the aggregate of such a size that it will all pass through a mesh  $\frac{3}{16}$ " square measured in the clear, and not more than 5 per cent by weight shall pass a No. 100 B.S. sieve. 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 organic 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.

**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 be sometimes considered desirable to introduce a small percentage of silt in order to improve its harshness. The material passing a No. 200 B.S. 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 (about 1-oz. in a quart 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  $\frac{3}{16}$  in. 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 Sand

Sand when damped bulks or occupies more space than it does when completely dry. This bulking increases with increasing water content up to 5 or 6 per cent where it is maximum and then increase of moisture content above this optimum gradually reduces the bulking until the sand is completely saturated (or inundated) when the bulking is practically nil and the sand occupies almost the same volume as when dry. A moisture content of 2 to 5 per cent will increase the volume by 15 to 30 per cent or even 40 per cent. Fine sand bulks more than coarse sand. Even gravel of small size has been found to bulk as much as 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 =  $\frac{D-D_1}{D_1} \times 100$ , 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  $2\frac{1}{2}$  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  $\frac{1}{2}$  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 several times this amount of water.

*Approximate Quantity of Surface Water or Free Water Carried by Average Aggregate : (IS : 456).*

Condition of aggregate	Galls. per c. ft. aggregate
Very wet sand .. .. .	$\frac{3}{4}$
Moderately wet sand .. .. .	$\frac{1}{2}$
Moist sand .. .. .	$\frac{1}{4}$
Moist gravel or crushed stone .. .. .	$\frac{1}{2}$ to $\frac{3}{4}$

Wet sand has less voids as water fills in some space.

The total volume of voids between spherical particles contained in a given space is about the same regardless of the size of the particles provided they are all graded to the same size.

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.

### Percentage of Voids in Sand and Aggregate

Aggregate	Voids %
Sand, moist, fine, passing 18-mesh sieve ( <i>i. e.</i> , 324 meshes to the sq. in.)	43 (av.)
Sand, moist, coarse, not passing 18 mesh sieve	35 (av.)
Sand, moist, coarse and fine mixed, ordinary ...	38 (av.)
Sand, dry, coarse and fine mixed ...	30 (av.)
Gravel ...	27—37
Gravel and sand mixed ...	22—25
Ballast 1" and under, 6% coarse sand ...	33 (av.)
Broken stone 1" and under ...	46 (av.)
Broken stone 1½" and under, dust only screened	41 (av.)
Broken stone 2" and under, most small stones ...	45 (av.)
screened out ...	41 (av.)
Ditto. 2½" ...	35—40
Brick ballast	

### 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.

BS series of test sieves follow aperture widths based on the number of meshes per inch, *i. e.*, a No. 10 sieve has 10 divisions per inch length or 100 openings per square inch of the sieve; a No. 25 sieve has 25 divisions per inch length or 625 openings (or sieve meshes) per sq. in. In the case of Indian Standard sieves the designation is the same as the aperture width expressed to the nearest decam micron (or 0.01 mm); thus IS sieve designated as 50 means that the aperture width of that sieve is approximately 500 microns.

Test sieves 3/16 in. and larger are of perforated plates (square holes) and BS No. 7 and smaller are made of fine mesh wire cloth.

## COMPARISON OF DIFFERENT STANDARD SIEVES

Sieve Designation—Size or No.				Sieve Opening or Width of Aperture	
Indian Standard	Equivalent (approx.)				
	BS	ASTM	Tyler	mm.	in.
570	—	3½	3½	5.660	0.2230
480	4	4	4	4.760	0.1870-3/16"
400	—	5	5	4.000	0.1570
340	5	6	6	3.353	0.1320
320	—	—	—	3.180	0.1252-1/8"
280	6	7	7	2.818	0.1109
240	7	8	8	2.411	0.0949-1/10"
200	8	10	9	2.032	0.0800
170	10	12	—	1.676	0.0650
160	—	—	10	1.600	0.0630-1/16"
140	12	14	—	1.405	0.0553
120	14	16	14	1.204	0.0474-1/21"
100	16	18	16	1.000	0.0394
85	18	20	20	0.853	0.0336
80	—	—	—	0.790	0.0311-1/32"
70	22	25	24	0.708	0.0279
60	25	30	28	0.599	0.0236-1/42"
50	30	35	32	0.500	0.0197
40	36	40	35	0.422	0.0166
35	44	45	42	0.351	0.0138
30	52	50	48	0.295	0.0116-1/84"
25	60	60	60	0.251	0.0099
20	72	70	65	0.211	0.0083
18	85	80	80	0.177	0.0070
15	100	100	100	0.152	0.0060
12	120	120	115	0.124	0.0049
10	150	140	150	0.104	0.0041
9	170	170	170	0.089	0.0035
8	200	200	—	0.075	0.0030
6	240	230	200	0.064	0.0025
5	300	270	—	0.053	0.0021
4	—	325	—	0.044	0.0017

(A 200 mesh BS sieve passes only dust through it.)

Specifications of materials for the manufacture of test sieves are given in Indian Standard : 460—1953.

## Grading of Aggregates

A graded aggregate is one which is made up of stones or particles 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  $1\frac{1}{2}$  to  $2\frac{1}{2}$  times the volume of the fine aggregate to suit the size and grading. Grading is of great importance since it affects workability of concrete and hence density and strength.

The combined aggregate when mixed with the required quantity 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 mortar (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. For concrete works to be water-proof, a dense mix should be aimed at with small size of aggregate.

It is a well known concept that the sum of the solid volumes of ingredients (cement, sand, aggregate and water) in a given volume of concrete must be equal to the volume of the resulting concrete when fully compacted. Solid volume of a material is its weight (bulk) divided by its density. Density is specific gravity by weight of one c.ft. of water (62.4 lbs.). Density of cement is about 197 lbs. per c.ft. and density of water is its weight. While drying, water evaporates and leaves air voids; cement expands in the process and occupies some of the air voids.

Sometimes it will be found more economical to increase the cement content above the minimum required and to use a less satisfactory, but more easily obtainable, grading.

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. The proportion of sand to coarse aggregate is usually taken as 50 per cent. Percentage of voids can be tested by a simple method as described later.

The ratio of fine aggregate (material passing a  $\frac{3}{4}$ -in. sieve) should be decreased when the maximum size of aggregate is increased. For aggregate of  $1\frac{1}{2}$ -in. maximum size, it will generally be found that the best results are obtained when the fine aggregate lie between 25 and 40 per cent of the total, and for aggregate of  $\frac{3}{4}$ -in. maximum size, between 30 and 45 per cent. Leaner mixes may be used with the larger size of aggregate.

A somewhat coarse grading is more suitable for a concrete of low workability such as would be used where vibration is employed as a means of compaction. A somewhat finer grading would be better for a more workable concrete such as is required for hand compaction. A coarser grading is one in which a greater percentage of particles is retained on a particular sieve.

Excess of fine materials need more cement and more water. A small amount of very fine material (passing No. 200 BS sieve) may improve the workability of the concrete, but an excess causes reduction in strength. The material passing through No. 100 BS sieve 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.



For high-class concrete batching should always be done by weight. Care must be taken to allow for the bulking of damp sand. (See under "Bulking of Sand".) If the proportions by weight are to be converted into their equivalents by volume the bulk densities of the materials used must be known.

No definite proportions can be prescribed, the best proportions being determined by trial, since successive batches of aggregates may differ greatly. In making the proportional adjustments, the proportion of cement to the sum of the volume of the fine and coarse aggregate measured separately for any nominal mix should remain unchanged. The determination of the proportions and consistence is called "Design of Concrete Mixture".

**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 a particular nominal mix is called a "field mix." (See under "Bulking of Sand".)

Real mix proportion depends on the size of aggregate and the proportion of voids. Field mix proportion depends on the wetness of the sand and aggregate.

### 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.



The limiting proportions of particles of various sizes may be derived by any of the following common methods.

### Grading Limits for Coarse Aggregates

BS Sieve	Percentage passing		
	Nominal size of graded aggregate		
	1½" to 3/16"	¾" to 3/16"	½" to 3/16"
1½ in.	95—100	100	—
¾ in.	30—70	95—100	100
¾ in.	—	—	90—100
¾ in.	10—35	25—55	40—85
3/16 in.	0—5	0—10	0—10

Based on B.S.S. 882.

BS Sieve	Percentage passing				
	Nominal size of aggregate single sized				
	2½"	1½"	¾"	¾"	¾"
3 in.	100	—	—	—	—
2½ in.	85—100	100	—	—	—
1½ in.	0—30	85—100	100	—	—
¾ in.	0—5	0—20	85—100	100	—
¾ in.	—	—	—	85—100	100
¾ in.	—	0—5	0—20	0—45	85—100
3/16 in.	—	—	0—5	0—10	0—20
No 7	—	—	—	—	0—5

Based on IS: 383; BSS: 882. Aggregate complying with the grading requirement of BSS: 63 of the same nominal sizes will conform of these gradings.

### Fine Aggregates or Sand

BS Sieve	Percentage passing	
	Natural sand or crushed gravel sand	Crushed stone sand
3/16 in.	95—100	90—100
No. 7	70—95	60—90
No. 14	45—85	40—80
No. 25	25—60	20—50
No. 52	5—30	5—30
No. 100	0—10	0—15

Based on IS: 383 BSS: 882.

The following conform to the limits defined in the B.S.S. No. 63 :—

(a) Coarse Aggregate :

6 parts	2½"
4 "	1½"
2 "	¾"
1 part	¾"

For small quantities:

3 parts	2½"
1½ "	¾"

(b) Medium Aggregate :

5 parts	1½"
2 "	¾"
1 part	¾"

For small quantities :

3 parts	1½"
1½ "	¾"

(c) Medium Fine Aggregate.

5 parts	¾"
1 part	¾"
1 "	¾"

(d) Fine aggregate :

3 parts	¾"
1 part	¾"

For standard sizes of aggregate, see table under "Selection of Stone Metal" in Section 18. Sometimes standard size is taken as 90 per cent passing the specified size sieve and 95 per cent retained on the next smaller size sieve.

Sand with above gradings :

Whole passing ½-in. screen, not more than 30 p.c. passing a ⅜-in. screen, not more than 60 p.c. passing a ¼-in. screen.

All-in Aggregates

BS Sieve	Nominal maximum size of all-in aggregate	
	1½"	¾"
	Percentage	passing
1½-in.	95—100	—
¾-in.	40—70	95—100
3/16-in.	25—45	30—50
No. 100	0—6	0—6

BSS : 882.

All-in aggregate for ¾" maximum size, small quantities :—

Passing ¾" mesh	100 p. c.	} Coarse aggt.
Passing ¾" mesh	55-65 p. c.	
Passing ⅜" mesh	35-42 p. c.	} Fine aggt.
Passing No. 100 sieve	3 p. c.	

The term "all-in aggregate" or "combined aggregate" is applied to a graded aggregate containing both coarse and fine materials in varying proportions.

Nominal mix.	Real mix.	Field mix.
1:2:4	1:5.1 to 5.5	1:2½ to 2½ : 4½
1:3:6	1:7.65 to 7.8	1:3½ to 4 : 7
1:4:8	1:10.2 to 10.5	1:4½ to 5½ : 9

It should be noted that a nominal mix. of 1:2:4 is not, as is sometimes thought, 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.

If combined (natural) all-in aggregates are available, they need not be separated into fine and coarse, but necessary adjustments may be made in the grading by the addition of single-sized aggregates. All-in aggregate generally contain an excess of fines.

Another method for derivation of optimum proportions of mix :—

BS Sieve	Percentage passing			
	Maximum size of aggregate			
	3"	1½"	¾"	½"
3—in.	100	—	—	—
1½—in.	70—80	100	—	—
¾—in.	55—70	50—75	100	—
½—in.	40—60	36—60	45—75	100
3/16—in.	25—40	24—47	30—48	60—80
No. 7	20—4	18—38	23—42	40—60
No. 14	15—30	12—30	16—34	20—40
No. 25	10—20	7—23	9—27	15—30
No. 52	3—15	3—15	2—12	6—20
No. 100	0—4	0—5	0—5	0—7

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 Proportioning Mixes

(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 : 1½ : 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 : 1½ : 3	Water tanks, top of road surfaces, piles, fence posts, pre-cast R. C. works and other works where dense concrete for impermeability or extra strength is required.
1 : 2 : 4	Normal R. C. works and for ordinary uses of concrete such as in beams, columns, walls, arches, road slabs.
1 : 2½ : 5 1 : 3 : 6	Mass concrete in superstructure, massive R.C. 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 voids in the combined aggregate plus 10 per cent. One cwt. of cement is assumed to produce one cubic ft. of cement paste.

(iii) Proportioning by trial mixes. It has been stated earlier that to produce the 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.

Max. size of aggregate	Percentage retained on sizes of sieves				Fineness Modulus
	1½"	1"	¾"	⅜"	
3"	30—60	50—80	80—95	100	7.6—8.2
1½"	—	30—70	70—90	100	7.0—7.6
¾"	—	—	40—70	100	6.3—6.9

BS sieve No.	7	14	25	52	100	Fineness Modulus
Percentage retained	0—15	15—45	40—70	70—90	95—100	2.2—3.2

Max : size of all-in aggregate	Retained on sieve of max : size (per cent)	Retained on 3/16" sieve (per cent)	Retained on No. 100 sieve (per cent)	Passing No. 100 sieve (per cent)
¾"	Max : 5	40—60	30—50	Max : 10
1½"	Max : 5	50—75	20—40	Max : 10

Roughly 100 c.ft. coarse aggregate mixed with requisite quantity of sand, cement and water produces 110 to 120 c.ft. of concrete plain with normal proportions of mixes and 108 to 116 c.ft. concrete in 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 honeycombing 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 emptied 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 types of batch mixers. A simple method of estimating the performance of a mixer is to measure the uniformity of the concrete in a given mix by taking several samples from the discharged concrete and determining the proportion of each constituent in each sample.

Recent experiments have shown that the concrete produced by a non-tilting type mixer is more uniform, but there are many factors which affect the quality of the concrete produced. Most of the mixers are satisfactory with wetter mixes but difficulty arises 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.

Size of batch mixers is denoted by two numbers, *e.g.*,  $5/3\frac{1}{2}$ ,  $7/5$ . 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. Non-tilting mixers are made in sizes ranging from 5 c. ft. to 4 c. yds., and tilting drum mixers are made in still smaller sizes. 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 necessity 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 and for a 1 : 3 : 6 mix a 14/10 will be required.

The quantity of material mixed should not exceed the rated capacity of the machine, which is not generally more than 60 per cent of gross volume of the drum. (If concrete is central mixed and only transported in truck mixers, 80 per cent of volume is usually allowed.)

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 admitted. This will prevent the accumulation of cement paste around the blade roots. The rest of the water may be admitted simultaneously with the 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  $1\frac{1}{2}$  to 2 minutes. About 30 revolutions per minute are generally considered sufficient. (Drum speeds vary with different makes of mixers, but are usually about 30 r. p.m. for a 7/5 and decrease with increasing size of mixer to about 16

r. p.m. for an 18/12.) 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 7 ft.  $\times$  12 ft. or 10 ft. square with strips 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 shall be used when hand mixing is resorted to.)

The workability and plasticity of concrete will be increased by thorough mixing. To obtain this 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.

As a rule not more than 4 c. ft. of coarse aggregate should be mixed at one time in one batch, but in large works where organization and supervision is very good this quantity may be exceeded. Not more should be spread than can be mixed in ten minutes, and placed in position before the initial set of cement.

Cement and sand should be first mixed dry until the mixture is thoroughly blended and uniform in colour, 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.

### **Placing of Concrete & Preparation of Formwork**

Concrete should be placed and compacted as soon as possible after it has been mixed with water and before initial set of cement; a maximum time-limit of  $1\frac{1}{2}$  hours is allowed (and which may be 2 to 3 hours if conditions are favourable) 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 consistency 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 forms 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 6-in. to 12-in. for reinforced work and up to 18-in. for mass concrete.

The layers are to follow in quick successions to prevent any distinct joint between them and each layer should be placed before the previous one has set. 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 5 ft. 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.

**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 6 ins. to 12 ins. 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.

**Bottom opening bucket or skip.** This is a specially made bucket of which the bottom opens downward and outward when it is lowered down and tipped. The bucket is used for dumping down the concrete on the surface where it is to be deposited. It is essential that the bucket be lowered on to the bottom or working face before the bottom doors are opened, so as to reduce to the minimum the depth of water through which concrete has to pass. Some loss of cement is inevitable with this method of placing, and the cement content of the mix must therefore be rather higher than that actually required for structural purposes. 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. No tamping or ramming shall be done until the concrete surface rises above the water level.

The mixture should have 10 to 30 p. c. extra cement than usual and a slump of not less than 4" and not more than 7". 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.

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.



**By bags.** Old cement jute bags are used 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 bag 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.

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 1:1½) 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").

### FORMWORK

Shuttering, centering and falsework are synonymous terms in common use.

The formwork must be strong and rigidly braced which will not bulge or sag when concrete is placed and it must be so 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.

Loads for design of formwork are : (a) weight of the wet concrete, and (b) live load due to men working, impact, etc. The weight of wet concrete is taken at 144 lbs. per c. ft., or 12 lbs. per inch thickness per sq. ft. The maximum live load is normally taken at 75 lbs. per sq. ft. Allowable bending stress (flexural tensile stress) in soft timbers may be taken 1200 lbs. per sq. in. Posts over 8 ft. in height should be braced both ways at centre. Spans of beam bottoms should be supported at 3 to 4 ft. according to depth of the beam.

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 have long life if good maintenance is given, they can be used up to about 50 times before repair becomes necessary whereas timber formwork cannot normally be reused more than four or five times.

Steel forms may be of steel sheets 16 gauge, or preferably 14 gauge thickness, stiffened with angles etc.

#### **Compacting Concrete**

The purpose of compaction is to expel as much as possible air bubbles in the concrete mass entrapped during mixing. 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 consist of many small holes. Concrete will only develop its full strength if it is thoroughly compacted ; full compaction is essential not only for strength but also for impermeability. If full precautions are taken a concrete will be for all practical purposes water-tight although not necessarily damp-proof. Thorough compaction can only be achieved by the correct proportioning of the mix in relation to the method of compaction to be employed. Over-compaction is also equally 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 vibrators.

**Hand Compaction** is carried out by either : rodding, tamping, hammering on the outside of the moulds or shuttering, or ramming. Rodding consists of inserting a bar (may be a piece of reinforcing rod) vertically into the concrete and moving it up and down until the concrete is thoroughly worked into place.

Special care should be taken to see that concrete is worked well into all corners, cavities and around reinforcement; also in ramming around rods to prevent distortion. Slabs should be tamped with a small wooden mallet or a tamping rule which will also serve for finishing of the concrete. Heavy masses of unreinforced concrete should be rammed with a rammer. Cement concrete is not rammed heavily but is thumped slowly, especially thin sections. 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.

In filling columns, the concrete should be poured into the moulds in three inch layers and constantly tamped and puddled with a rod to expel air bubbles. The work must not stop until the column has been completed.

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. 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.



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**

Compaction by vibrations permits the use of less water which gives higher strength, and coarser gradings, with a consequent reduction in required cement content. A larger proportion of coarse and a smaller proportion of fine aggregates can be used giving less shrinkage and more 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 (5 ins. or more slump) should not be vibrated; 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.

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 the most efficient for general work and should be used whenever conditions permit. The flexible handled types 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.

External vibrators are clamped to the forms and are useful for awkward corners or very thin sections where an internal vibrator cannot be 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.

The minimum frequency of vibration for effective compaction is about 3,600 r. p. m. ; the higher the frequency of vibration the shorter the time required for full compaction. Over-vibration causes segregation and vibration should be stopped when scum appears on the surface.

### Surface 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 consist of equal parts of cement and fine sand made into the consistency of a stiff oil paint. The area to be coated should be thoroughly 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 thoroughly 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 ; the longer the rubbing is delayed, the more difficult the process will be. If however, the work has been delayed and the surface hardened, a cement wash should be used instead of plain water. The second rub should be applied at the end of about a month's time with No. 24 carborundum stone, using plenty of clean water, which is followed by brushing and washing.

A plain smooth surface may be obtained by the use of planed boarding of good quality with tight fitting and and preferably 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.

**Cement wash.** The simplest form of concrete paint is a cement wash which is a slurry made up of simply cement and water 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) 1 part cement, 1 part sand, and 5 per cent of hydrated lime by weight of cement.

(iii)  $1\frac{1}{2}$  lbs. of calcium chloride in five gallons of mixing water. Calcium chloride absorbs moisture from the air and keeps the cement damp.

(iv) 2 lbs. of common salt per bag of cement in the mixing water will also give some improvement.

There are a variety of proprietary 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 observed.

### 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  $\frac{1}{4}$ -in. to  $\frac{3}{4}$ -in.



(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 outerskin 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,  $1\frac{1}{2}$  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 will produce a rough pleasing appearance.

A wet plastic mix of 3 parts cement, 1 part lime, 6 parts sand, and 4 parts of  $\frac{1}{4}$ -in. to  $\frac{1}{2}$ -in. shingle or crushed stone which is thrown on to the wall by means of a scoop or plasterer's trowel. This is called *rough cast*.

A  $\frac{3}{4}$ -in. coat of 1 part cement, 1 part lime and 5 parts sand upon which, while it is still soft, is thrown  $\frac{1}{4}$ -in to  $\frac{1}{2}$ -in. selected shingle which has been well washed. This is called *pebble-dash*.

### Water-proofing Concrete

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 so as to have a minimum of air voids. 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. The following methods can be used for further water-proofing :—

(a) Concrete and masonry surfaces can be made waterproof by giving three alternate coats of alum and soap solutions.  $1\frac{1}{2}$  ozs. of alum is dissolved in one gallon of hot water, and 9 ozs. soap is dissolved in one gallon 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 of 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 :  $2\frac{1}{2}$  : 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 a 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.

Rendering with mortar consisting of cement, hydrated lime and sand in the proportions 1 : 3 : 10,  $\frac{1}{2}$ " thick will also make the 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 $\frac{1}{2}$  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 concrete. 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 make 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. 10 to 12 galls. per 100 sq. ft. of bitumen are required.

(f) Bituminous Mastics are generally laid on horizontal surfaces and also trowelled 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 manufactures' instructions.

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) A pound of washing soda dissolved in three gallons of mixing water will make a cement mortar water-proof.

### **Cold Weather Concreting**

As has been discussed earlier, the rate of hardening of concrete is very much retarded when the temperature falls. 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.

Many specifications require that concreting be discontinued when the temperature falls below 38° F. (Use immersion thermometer inserted in concrete near forms or surface for recording temperature.) If however, the work is of urgency or importance that it must be continued, it can be carried out with complete success provided certain precautions are taken.

Fresh concrete must not be allowed to freeze before it has fully hardened. Most convenient method to adopt for protecting exterior concrete work is to use hot water for mixing. The temperature of the water should be about 140° F. On no account should the hot water be added to the cement alone. Aggregates can also be heated to 70° F. before mixing. The temperature of wet concrete as mixed should however, be not more than 100° F. when normal or rapid hardening cement is used or 85° F. when high alumina cement is used. Mixer drum may also be warmed. Cement should not be heated.

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.

*Calcium chloride* is the most commonly used material to accelerate hardening of the concrete and is perhaps the most reliable. The usual proportion is not more than 2 lbs. of calcium chloride per cwt. of cement; more is harmful. This will give satisfactory results provided the temperature does not fall below 23° F. In no circumstances should this chemical be added to high alumina cement. Calcium chloride is a white deliquescent and hygroscopic salt commercially available (and may be obtained from The Imperial Chemical Industries) at low cost in a flake or granular form and delivered in moisture-proof bags or airtight drums, and should be stored in a dry place. It is dissolved 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.

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 by about 1.5° F. for each per cent salt added to the water. For temperature below 32° F. dissolve 1 lb. of salt in 18 gallons of water; add 3 oz. salt for each 3 deg. below 32° F. Larger percentages of salt appear to weaken the concrete. Salt over 5 p. c. 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. Salts should be thoroughly

dissolved or the results will not be satisfactory.

It has been shown earlier that heat is generated during the hydration and setting of cement and the rate of hardening is accelerated by increased heat generation. Whether or not steps are taken to increase heat generated (by the use of extra cement, use of rapid hardening or high alumina cement or accelerators), action must be taken to prevent concrete freezing for at least two days, and to minimize heat loss. Timber formwork is a valuable insulating agent and should be used in cold weather. The concrete must be kept warm 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 sacking, straw blankets, old paper cement bags, tarpaulins or a 6-in. 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. Precautions 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

**Effects of 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 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 of cement. The prevention of such loss of water from concrete during its early life is known as curing. Concrete must be kept moist until it has reached a required strength; this is of particular importance during the early life of the concrete.

The strength of 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 curing is continued for about a month.

*Strength of Ordinary Portland Cement Concrete at Various Ages :—(Approx.)*

3 days	20%	3 months	85%
7 days	45%	6 months	95%
28 days	60%	1 year	100%

*Strength of Rapid-Hardening Cement Concrete :—(1:2:4 mix.)*

Age	24 hours	2000 lbs./sq. in.
	2 days	3000 "
	3 days	3800 "
	7 days	5300 "
	28 days	6810 "
	12 months	7500 "

*Strength of High Alumina Cement Concrete :—(1:2:4 mix.)*

Age	7 hours	3700 lbs./sq. in.
	12 hours	7000 "
	24 hours	7400 "
	3 days	7580 "

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 tempera-



tures 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 occurred and before it has hardened, and should be continued for a minimum period of 7 to 10 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. If concrete is frozen, setting and hardening ceases. 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.** Evaporation of mixing water is largely prevented by 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 must be taken that water is not lost by absorption from the concrete itself. This can be done by either painting the inside of forms with mould oil or by well wetting them before concreting. Sub-grades (of roads) and foundations should also be wetted. Where forms have to be stripped before the curing period is complete, other methods of curing, as detailed below, will have to be adopted. If surface finish is important then the method adopted for curing must be such as will not cause it to be damaged. Soon after the concrete is placed it should be protected from the direct rays of the sun by covering with hanging curtains on vertical surfaces or by screens mounted on light frames on flat surfaces. Screens should be fitted with side and end flaps so as to prevent wind blowing under them and must be secured against blowing away. When the concrete has hardened sufficiently, a few hours after placing, to resist damage, any of the following methods may be used :—

(i) Covering with gunny bags, hessian, coir matting, bundles of straw, 2 to 3-inch layer of saw-dust, sand or earth, or other moist retaining materials and keepig such materials constantly moist by watering. It should be noted that many earths and loamy sands will cause discolouration of concrete.

(ii) Ponding.—Small earthen mounds are made in squares over the entire area, and standing waterto a depth of one to two inches. This is probably the most common method for flat surfaces.

(iii) Spraying with water. Continuous saturation by sprays or wet fabric is preferred to intermittent sprinkling by hand. Spraying water (under pressure) within the first few hours after concreting should be avoided.

(iv) Vertical surfaces present some difficulties. In some cases it may be possible to spray the concrete periodically with water. Where this cannot be done, large 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. When curing in done in a tank (of precast members) water should be changed frequently.

There are a number of proprietary membrane curing compounds on 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 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  $2\frac{1}{2}$ " to 3" (and 4" 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. Dirty and flaky aggregates and oversanded mixes must be avoided. The mix should be at least of 1 : 2 : 4 in the case of plain concrete and of  $1 : 1\frac{1}{2} : 3$  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 effect of using different cements has not been fully established. The sulphate-resisting cements and high-alumina cements have a greater resistance to sea water than ordinary Portland cement. Portland blast-furnace cement is considered highly resistant to attack by sea water and it has been used at places abroad.

Special care has to be given to construction joints; no construction joints should be allowed within two feet below low-water level or within two feet of the upper and lower planes of wave or within two feet of the upper and filling in the joint spaces. Care should also be taken to protect the reinforcements from exposure to salty atmosphere during storage and preparation.

Resistance to attack can be increased by the addition of *surkhi* or pozzuolana (ground fine as cement) about 10 to 30 per cent of cement (cement will be proportionately less) in a concrete. "Trass" is used in England: 1 part trass, 2 parts cement and  $2\frac{1}{2}$  to 3 parts of well graded aggregate.

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 of jointing material can be of acid-resisting cement. Bituminous paints or emulsions applied over concrete or mortar joints become 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 concentrations 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 : 1½ : 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.

The presence of sulphate 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 laboratory 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 2" min., and for foundations and footings, a covering



of 4" should be given. Construction joints should also be reduced to a minimum.

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 construction joints.

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. A fine sand would not permit this and so will not be bound together so firmly. 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 No. 52 mesh (B. S. sieve) tends to give harsh-working mixes prone to segregation and to local defects. For concrete placed by hand, the sand should contain not less than 15 p.c. passing the No. 52 mesh, and values up to 20 or 30 p.c. are to be preferred. Lower values are suitable for concrete placed by vibration.

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. Give adequate cover of concrete over reinforcement and use impermeable concrete as far as possible. A covering of cement plaster about  $\frac{1}{2}$ " to  $\frac{3}{4}$ " thick, not weaker than 1 : 2 $\frac{1}{2}$ , applied on all doubtful faces



will afford better protection to the rods than a concrete with porous ballast. In the case of R. B. roof slabs, the rods must not touch the bricks which are porous, but should have a covering of  $\frac{3}{4}$ " thick cement mortar. Rods can also be protected by applying a coat of cement bitumastic paint and dusting with neat cement. But the rods must have adequate covering of concrete.

### **Cracking in Concrete—Causes and Remedies**

(i) Settlement due to shrinkage around reinforcements or aggregate particles can be avoided by graded aggregate, low water-cement ratio and adequate compaction.

(ii) Delayed finishing and final floating of concrete up to a certain limit, avoids surface cracks.

(iii) Surface (form-work etc.) or sub-grade on which the fresh concrete is placed must be damped or it should otherwise be water-proof so that it does not absorb water from the concrete.

(iv) Formwork or sub-grades should be of adequate strength to bear the pressure of the wet concrete without swelling, spreading, or any movement.

(v) Ensure adequate moist curing.

(vi) Sufficient thickness of concrete should be given at the points where bars are bent up and anchored, to avoid minute hair cracks.

(vii) Concentration of tensile reinforcements at square openings or re-entrant angles (as in corners of doors and window openings) cause cracks. Can be avoided by suitably placed reinforcements having adequate coverings.

A defect commonly seen in concrete work is that known as "crazing", or fine superficial cracking. These "hair" cracks do not affect the strength of the concrete, but they are nevertheless very unsightly. The factors which influence crazing are (i) chemical action, (ii) expansion and contraction due to temperature or moisture changes. Hair cracks are partly due to the unequal shrinkage of the surface concrete and the mass behind it, and generally result from the use of a mortar surface dressing too rich in cement, too much water, insufficient curing, or from over-trowelling. One method of avoiding craz-

ing is to remove the surface skin of the concrete by brushing it with a stiff brush soon after setting. Moist curing for long periods decreases the possibility of cracking.

**Treatment of Cracks.** The strength of R. C. 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 part of fine sand. The crack is sealed at the surface with cement paint or grout of stiff consistency applied with a brush, leaving openings 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.

Cracks in water retaining structures can be filled with hemp or jute fibres 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 Silicate of Soda** (water-glass). 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 and the grade specially made for treatment of concrete known as P.84 is available from the Imperial Chemical Industries. 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 has



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 be used with 1 to 3 (silicate to water) for the second coat and 1 to 2 for the third coat. For average concrete, 5 galls. of the diluted solution will be sufficient for about 1000 sq. ft. 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.

Other forms of surface treatments are also applied where concrete may suffer deterioration if it is brought into contact with certain substances. 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 be used:—

**Bolled linseed oil.** Best results are obtained when the oil is applied hot. Two or three coats should be given and each 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, and page 7/12.)

#### **Staining Concrete with Copper Sulphate**

One of the methods employed for colouring concrete is that of staining by means of a copper sulphate solution and below are given some details of its application:—

The surface of the work should first be washed down and the two applications of copper sulphate solution applied with a brush whilst it is yet slightly damp. The strength of the solution used will vary in accordance with the shade required. For ordinary concrete work the percentages will vary from 10 to 25. A 10 per cent solution



requires 1 lb. of sulphate to 1 gallon of water; this will cover about 15 sq. yds. allowing for two applications. This will give a blue stain.

The copper sulphate should be mixed in an earthenware (glazed) container. If mixed in an iron container an electro-chemical action takes place causing part of the copper sulphate to turn into ferrous sulphate. The ferrous sulphate will turn the blue copper sulphate green. If it is desired to obtain a green stain, mixing in an iron container might be resorted to, but this method cannot be advised as the depth of colour cannot be controlled.

A better method of preparing the green stain is to mix two separate and distinct solutions, one of copper sulphate and one of ferrous sulphate, both being 10 per cent strength, i.e. 1 lb. of sulphate to 1 gallon of water. Small quantities of the ferrous sulphate solution should then be added to the copper sulphate solution, until the desired shade of green is obtained; the reason for adding the ferrous solution to the copper is that the former is very much the stronger of the two, and only a small quantity of the ferrous solution is therefore required to obtain a suitable green stain.

As an alternative, good effects can be obtained by first giving an application of copper sulphate and while the copper sulphate is still wet, to apply ferrous sulphate in patches. If the sulphates are then sprayed with water a cloudy or blended effect will result. Care must be exercised in not applying too much of the ferrous sulphate, as both of the solutions give a green appearance when first applied to the concrete.

If the silicate of soda treatment is employed on a surface previously treated with copper sulphate, the staining effect is lost.

### Gunite 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 of a capacity of about 200 to 250 c.ft. of air per minute at 80 lbs. pressure, spray nozzle and 2-ins. flexible hose pipe (heavy duty) which

should not be longer than 200 ft. At the end of the hose there is a nozzle to which water under pressure by a separate connection is supplied. Uniformly graded, thoroughly mixed dry materials are charged into the gun and shot under a pressure of about 35 to 40 lbs./sq. in. by compressed air. Slightly moist sand, with about 3 to 6 p.c. of moisture, 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  $\frac{3}{8}$ -in.; usual size is  $\frac{3}{16}$ -in. downwards. Hard-stones sand should be used. About 3 galls. of water per cwt. of cement, just only sufficient water necessary for the hydration of cement, is used. The quantity of water added can be regulated for correct consistency. There is usually 20 to 30 p.c. of "rebound" depending upon the wetness of the mixture. A very wet mixture will not stick on. While shooting, the nozzle should under normal conditions be held at a distance of 2 ft.-6 ins. to 3 ft. 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. If sand trowelling is obligatory, it should be done immediately after shooting as gunite hardens in a short time. The correct No. of gun should be obtained for the max: size of aggregate or sand to be used. The compressed air entering the gun must be dry and free from oil. In one operation a coating of only 1 to  $1\frac{1}{2}$  ins. thickness can be applied. For greater thickness more than one operations are required with an interval of about 30 to 60 minutes between each operation. A finishing or "flash" coat may be applied as soon as initial set of main coat has begun. The flash coat should be as thin as possible, usually about  $\frac{1}{8}$ -in. Reinforcement, usually of 3-in. sq. mesh, may be incorporated to withstand structural and temperature stresses.

This method is very useful for rehabilitating or re-conditioning old concrete, brick or masonry works which have deteriorated either due to climatic conditions or inferior work. It is also used for water-proofing exposed concrete surfaces or for resisting water pressure on pipes, cisterns, etc., where it forms a very impervious layer.



The low water/cement ratio used and the high degree of compaction obtained produce a hard and very dense concrete.

It is desirable to provide expansion and contraction joints in exposed surfaces thus treated as in the normal concrete works.

### Grouting or Colloidal Concrete

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 7 galls. of water to 1 cwt. of cement, to which sand is added afterwards to form a colloidal grout. The water content should be the minimum that is required to produce 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. 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 form, 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 1-in. The coarse aggregate should not contain any particles of less than 1-in. 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. When grouting is to be done for depths of over 3 ft., 2-ins. dia. pipes should be embedded at intervals and the grout poured through them. A dispersing agent such as "Cheecol" is generally added at the rate of 1 gall. to 2 tons of cement, or 4 fluid ounces per (cwt.) bag of cement. The function of a dispersing agent is the same as that of a "wetting" agent. (See also under "Cement Grouted Macadam" in Section 18.)



*Cheecol* is a liquid which when mixed with a cement grout, it makes the grout flow more freely round the aggregates so that all the voids are fully filled up.

### CEMENTATION OR PRESSURE GROUTING

Cementation is the forcing under pressure of cement grout into cracks, voids or fissures in structures or the ground. This system is useful for repairing structures, consolidating ground and forming water cut-offs, etc. Holes are drilled at selected points and cement grout, which is sufficiently thin to ensure complete penetration, is pumped in. This normally restores stability in a structure which has otherwise become unstable due to cracks or voids.

Ground of fairly hard nature but loose texture, *e.g.*, certain types of made-up ground, 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 pipes 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.

## 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 located at point of minimum shear, that is, midway between supports, with a vertical plane at right angles to the direction of the beam. 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 plane 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.

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 one inch of the underside 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.

Normally the construction should be planned so that the day'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.

(b) **Columns:** These should be finished with a level surface a few inches below junction with beams, this is particularly important in the case of the column heads in flat-slab construction. Two hours should elapse after the depositing of concrete in columns or walls before the depositing of 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 more mature *i.e.*, partially hardened, the surface should be scrubbed and wire-brushed, removing any laitance, care being taken that pieces of aggregate are not dislodged, moistened with water and neat cement slurry applied followed by the application of a thin layer of rich cement mortar (1 : 2).

(c) Where the concrete has hardened, similar treatment to the above is given. The concrete is hacked or chiselled and watered.

Fresh concrete should be well rammed against old, particular attention being paid to edges and corners.

Some engineers prefer a rebated or grooved joint; the surface of the previously laid hardened concrete is roughened, cleaned and covered with half-inch layer of 1 : 2 cement mortar.

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 explained 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) Temperature changes, and (iii) Changes in moisture content. These are called functional joints. In the initial stages large stresses are built up in the concrete and these stresses must be relieved by the incorporation of joints. Subsequently, when the concrete has hardened, expansion and contraction occurs due to the seasonal variations of temperature and moisture content. If after drying, concrete is subsequently wetted it will expand but not sufficiently to regain its original volume. 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.

The amount of shrinkage during setting and drying 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 per 100-ft. length of 1 : 2 : 4 concrete, based on water/cement ratio, is approximately :—

(i)	With water/cement ratio of	0.3	0.17 in.
(ii)	—ditto.—	0.5	0.50 in.
(iii)	—ditto.—	0.7	0.94 in.

In actual practice because of incomplete drying due to curing and the restraint offered by reinforcement the shrinkage that will occur is less than determined by laboratory experiments. In a normal concrete the amount of shrinkage may be  $\frac{1}{4}$  in. to  $\frac{1}{2}$  in. per 100 ft. length. Contraction of neat cement is roughly three times as great as of ordinary mortar 1 : 3 or 1 : 2 : 4 concrete.

The combined effect of shrinkage and temperature 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 surfaces of large continuous areas of

plain and reinforced concrete should be subdivided 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 occur. 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.

Co-efficient of Linear Expansion for 1 deg. F. :—

Steel	.0000067
Plain concrete	.0000060
Reinforced concrete	.0000065

Total expansion of a structure in inches = co-efficient of linear expansion  $\times$  length of the structure in inches  $\times$  change of temperature in deg. F.

For a rise in temperature of 50 degrees F. a concrete structure will expand  $50 \times .0000065 = 0.000325$  inch for every inch length. If this expansion is not allowed to occur and if the modulus of elasticity of the concrete is 3,000,000 lbs. per sq. in. the compressive stress which will develop will be  $0.000325 \times 3000000 = 975$  lbs. per sq. in.

A structure 100 ft. long will expand about  $\frac{3}{8}$ -in. through a temperature change of 50 deg. F.

Observation of buildings in service indicate, the total movement is somewhat less than half that which might be anticipated by combining the contraction due to temperature drop with shrinkage. It is considered that the max. movement at joints located 200 ft. apart will not exceed 1-in. under most unfavourable conditions. Some of the American experts consider that buildings under



200 ft. length need not be provided with expansion joints, but it is safer to provide joints at frequent intervals according to temperature variations. Minimum space left for a joint should be  $\frac{1}{4}$ -in. and maximum 1 in. to  $1\frac{1}{2}$  ins. If a building is constructed in summer, there need not be more than  $\frac{1}{2}$  in. between the two surfaces of the joint.

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 to crack inspite of the joint in it. Reinforcement must never pass through an expansion joint. 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 instance 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. 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 strip of 16 oz. metal is used for this purpose.  $\frac{1}{4}$ -in. diameter holes at 8-ins. 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 strip can be painted at the surface to match the wall. The figures shown on the following pages illustrate general principles and should be modified according to situation.

*Fig. A* : Vertical joint in walls. Where large allowance is desirable for expansion and shrinkage, a space of  $\frac{1}{4}$ -in. can be left in between. Where dealing with water leakage,



a copper strip, 16 oz. metal, can be put in.

In unreinforced walls, joints at intervals of about 15 ft. may be necessary for walls exposed to weather and 30 ft. for walls protected from weather.

*Fig. B :* Vertical joint in reservoir wall.

*Fig. C :* Joint in roof slab.

*Fig. D :* Joint for external wall of buildings.

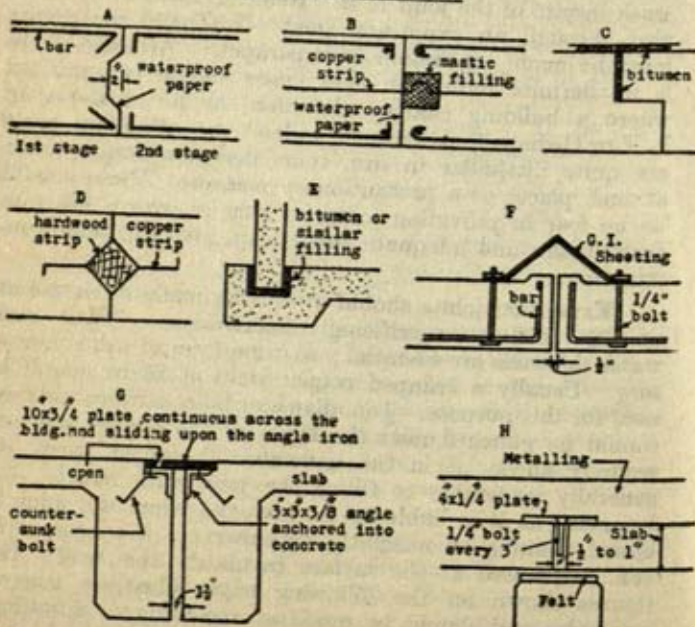
*Fig. E :* Base of wall of cylindrical tank, where wall and floor are not monolithic.

*Fig. F :* Roof expansion joint.

*Fig. G :* Bridge slab without metalling.

*Fig. H :* Bridge slab with metalling over it.

#### JOINTS FOR R.C.C. WORKS



#### Removal of Centerings or Formwork

All formwork should be stripped or removed with care so as not to damage the concrete; wedges etc. should be

slackened gradually to avoid imposing sudden loads on the structure. Under no circumstances should the forms be removed until the concrete has hardened sufficiently and may be left in place for as long as possible up to a maximum in normal weather of about 28 days. A very rough guide is to strike the concrete with a light hammer; a hard metallic sound indicates that the concrete has hardened sufficiently for the forms to be removed. With high temperatures the period should be less and with low temperatures more as cold weather will slow down the rate of hardening. In accessing the minimum periods, any day during which the air temperature remains below 40 °F. should be disregarded and each day that the temperature is between 50° F. and 40° F. should be counted as half a day. Minimum periods for the removal of centerings may be taken as follows :—

(i) For structures not carrying loads :

Sides of beams, walls and columns 2 to 6 days

Bottom forms of beams and slabs 7 to 14 "

Props or supports to bottoms of slabs  
and beams (more days for greater  
spans and cold weather) ; to prevent  
sagging of the members. 14 to 28 "

With rapid hardening cement 3/7 of the above time with min. of 24 hours, should be taken. With high-alumina cement a further reduction may be made.

(ii) For structures carrying loads :

Side timbers should not be removed before 7 days and supporting timbers 28 days. No loads should be allowed on the works before 28 days.

Construction loads on centering should be taken as 50 lbs. to 75 lbs. per sq. foot. (Load for wet concrete.) See under "Formwork".

## 10. SPECIAL TYPES OF CONCRETE

### 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 surface of a beam and when the design load is applied, tension is produced on the lower surface 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 either encased in pipes or sheaths, or holes are left in the concrete through 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. Pre-tensioned 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 100 lbs. per c. ft. are generally termed as lightweight concretes. Such concretes are usually produced by using lightweight aggregates such as breeze or clinker, foamed slag, pumice. Such concretes have good insulating qualities but are porous and absorbent 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 panels, flooring and for fixing bricks for joinery.

### No-Fines Concrete

No-fines concrete is made with coarse aggregate and cement only without using fine aggregate (sand). The coarse aggregate is graded to pass  $\frac{3}{4}$ -in. sieve with not more than 5 per cent passing through a  $\frac{3}{8}$ -in. sieve. Any normal aggregate may be used though a natural gravel is best. Mix proportions are 1 : 8 by volume, or 1 cwt. of cement to 10 c.ft. 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 interstices.

This type of concrete is light in weight, about 100 lbs. per c.ft.; thermal conductivity is less than normal concrete; crushing strength at 28 days is 500 to 800 lbs. per sq. in.; 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 way 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 1-in., 1½-in. or 2-in. thick layers over 3 ins. to 4 ins. 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 ¾ in. down, 1 : 2 : 4 mix., or graded from ½ in. to ¾ in., the matrix being 1 part of cement to 2½ 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 1½ ins. 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 at once well beaten and consolidated with 5-lb. 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 6 in. × 6 in. mesh placed ¾ in. 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  $\frac{3}{8}$ -in. down to No. 100 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  $\frac{3}{8}$  in. (or  $\frac{1}{4}$  in.) down to No. 7 sieve and fine aggregate from  $\frac{3}{16}$  in. down. The all-in aggregate specified above may consist of 3 parts of  $\frac{3}{8}$ -in. screenings and 1 part of  $\frac{1}{4}$ -in. 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  $\frac{1}{2}$  in. 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 1-in. and preferably  $1\frac{1}{2}$ -ins. An adequate key between the base concrete and the top finish is essential where the base concrete has set.

**Joints in concrete floors.** Floors should be laid in a series of panels not exceeding 15 ft. in length or 200 sq. ft.



in area, which may be reduced in exposed positions. Cement concrete floors are generally laid in 4 ft. to 8 ft. panels to avoid cracking due to contraction during setting. Thin ( $\frac{1}{8}$ " ) jointing strips of iron, teakwood, brass or ebonite 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 or 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)  $1\frac{1}{2}$  in. to  $\frac{3}{4}$  in. layer of concrete is laid wherein crushed marbles are used as aggregate.

(ii) The top course is made with a mixture of 1 cement and 2 marble chips  $\frac{1}{8}$  in. size, laid  $\frac{1}{4}$  in. thick.

(iii) A layer of 1 : 3 cement mortar  $\frac{1}{2}$  in. thick is laid over the concrete base next day the base has been laid. A terrazo topping  $\frac{1}{2}$  in. to  $\frac{3}{4}$  in. thick consisting of 1 part cement and 2 to  $2\frac{1}{2}$  parts marble chips ( $\frac{1}{8}$  in. to  $\frac{1}{2}$  in.), well 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  $\frac{3}{4}$  in. to 1 in. Small approximately cubes of marble of various colours or pieces of terracotta are pressed into the mortar which may be arranged into various patterns. 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 pig-

ment to cement). Weight of colouring pigment used in excess of 12 per cent of the weight of cement 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 the 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 1 c.ft. of cement and 17 lbs. of red oxide or ivory black laid  $\frac{1}{4}$  in. 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  $\frac{1}{2}$  in. thick cement plaster 1 : 3. Top layer to be same as for the flooring.

**Trowelling Floor Surfaces.** 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: At this stage no more work should be done than is necessary to smooth and thoroughly compact the surface and final finishing should not be attempted. 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 the advantage of not exerting much suction but a steel trowel is necessary to form a hard and smooth metallic surface. Exterior surfaces which are not to have a very smooth finish should not be trowelled to the same extent as the floor surfaces and 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.

**Grinding and Polishing.** When the 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 blocks 4" to 5" in dia. and 2" to 2½" thick) and fine sand sprinkled over the surface, using water freely. All pores and holes are filled with cement mortar of the same materials 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 ( $\frac{3}{4}$  lb. per 100 sq. ft.) 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 thoroughly 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.

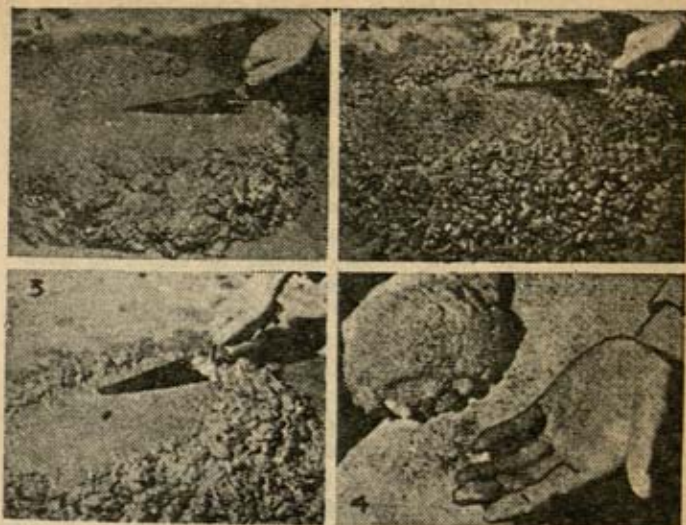
Oils used on dusting concrete floors have not been very successful.

Floor paints in various colours are available and give a hard, wear-resisting surface and withstand the action of water. They form a hard film on the top. Cement paints have been described elsewhere.

Cement rendering ½-in. 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-sand 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.

## 12. REINFORCED BRICKWORK

The design of reinforced brickwork structures shall be based on the same general principles of design and analysis as are adopted for the design of similar R. C. C. 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. Strength 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 bricks for roof slabs as shown in the illustrations is a much 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 soluble salts or other deleterious material (efflorescence). The concrete should consist of well graded fine aggregate all passing  $\frac{3}{16}$  in. B.S. sieve. Use the Table for reinforced concrete slabs (page 8/28) and place reinforcement rods in joints. Concrete at the top of the bricks should not be less than  $\frac{1}{3}$ rd the effective depth of the slab.

Reinforcement rods of greater diameter than  $\frac{1}{2}$  in. should not be used. 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 rods should be straight and free from kinks. All bars should be in one plane only. The thickness of the joint should not be less than  $1\frac{1}{2}$  ins. and not less than three times the diameter of the reinforcement rods. A line of bricks should be first laid 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 R. C. C. 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 cement concrete and steel.

The following working stresses may be taken generally for R. B. works :—

Safe compressive (in bending) stress for bricks in slabs when compression is limited to the thickness of one brick only ... 350 to 400 lbs./sq. in.

Ditto. for beams or slabs when compression is not limited to one brick ... 250 lbs./sq. in.

Direct compression ... 200 lbs./sq. in.

Ditto. with loop binders ... 300 lbs./sq. in.

Brickwork in shear or diagonal tension 25 lbs./sq. in.

Ratio of modulus of elasticity for

steel to that of brick ... 40 to 60

Adhesion between steel and mortar 80 to 90 lbs./sq. in.

Shear in brickwork ... 60 lbs./sq. in.

(If bricks are tested, those giving a breaking stress of 1200 lbs./sq. in. in compression and 20 lbs./sq. in. in tension may be accepted.)

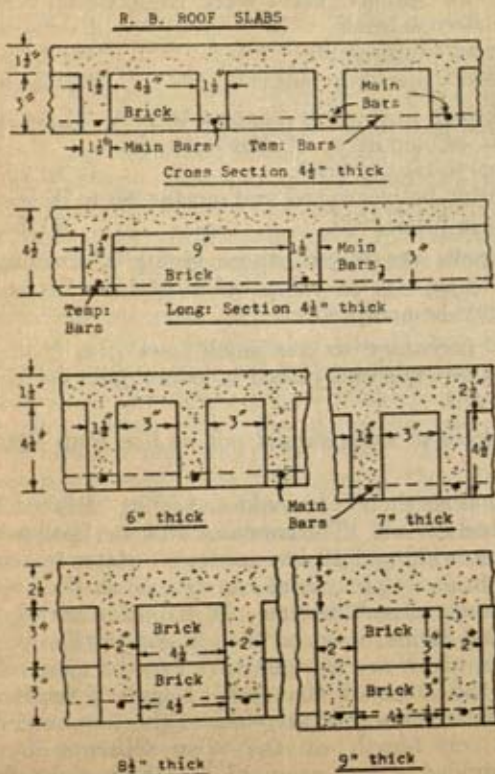
It is necessary to use small bars (less than  $\frac{3}{8}$  in.) hooked on all ends and placed in every joint for the main reinforcement.

Thickness of slabs should not be less than  $\frac{1}{30}$  of the span.

Another method is to make slabs like "Ribbed Floors" as described earlier. The concrete ribs or flanges can be made 3 ins. wide and 12 ins. centre to centre to accommodate 9-in. bricks.  $1\frac{1}{2}$  ins. thick concrete is on the top. The ribs are designed as T-beams. Distribution bars  $\frac{3}{8}$  in. dia. are provided 12 ins. apart on the top slab portion or flanges of the tees, which are at right angles to the main reinforcement in the stem of the 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, 6 inches 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.



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



## REINFORCED BRICKWORK SLABS

Based on: Stress in steel 18000 lbs./sq. in.; Stress in bricks (comp.) 350 lbs./sq. in.;  
Modular ratio 60; Percentage of steel 0.5 per cent; BM=65 bd<sup>2</sup>. (Approx. values).

Total depth of slab ins.	Safe load including weight of slab in lbs./sq. ft. on span of :										Steel area per ft. width	Depth below centre of steel
	4'	5'	6'	7'	8'	9'	10'	12'	14'	16'	sq. in.	in.
3	150	100	70	50	40						.11	$\frac{3}{4}$
4½	420	270	190	140	110	80	70				.19	$\frac{3}{4}$
6	750	480	330	250	190	150	120	90			.25	1
7½	1270	810	560	410	320	250	200	140	100		.32	1
9	1800	1150	800	590	450	360	290	200	150	110	.39	1½

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 spans 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; WL/2 (Cantilevers) by 0.50

This table is applicable where the joints are filled with cement mortar or fine cement concrete and rods are 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 and should have a tensile strength of at least 160 lbs./sq. in. in 7 days.

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  $\frac{3}{4}$  in. 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.

**Reinforced Brick Panelled Walls.** (See also Panelled Walls and Partition Walls in Section 7-pages 7/21 and 7/52.) Reinforcement may be of wire netting or hoop iron in every second or third joint fully embedded in cement mortar 1 : 3. The reinforcement may consist of two strips (of hoop iron) placed near to either edge of the wall. The reinforcement must be continuous between supports. Panels  $4\frac{1}{2}$  ins. thick should not be more than 20 ft. long, and 9 ins. thick not more than 30 ft. long. Where the panelling has 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  $4\frac{1}{2}$  ins. panelled walls.

Where panels are not reinforced, pillars may be provided 6 ft. apart  $13\frac{1}{2}$  ins. thick by 9 ins. broad, for half-brick thick panels.

### Reinforced Brickwork Water Tanks

(For unreinforced water tanks see Section 7.)

$t = hD/22$ .  $t$  is in inches (thickness of brick wall)  $h$  and  $D$  are in feet (height and diameter).

Hooped reinforcement (as explained under R.C. tanks), and vertical rods, to be fixed outside the wall. Outside to be cement plastered and inside rendered water-proof. Water lock (metal strip) to be provided at the junction of the wall and floor.

## 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.

**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 parti-



cles larger than  $\frac{3}{8}$ " in which case "coarse aggregate" is more correct. (See also page 18/9.)

**Bar :** The term is applied to simple sections both round or square, usually above  $\frac{1}{2}$ " size. (Also defined under "Steel Structures" in Section 21.)

**Batch :** The quantity of concrete mixed at one time.

**Bleeding :** The discharge or 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.

**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 continuous 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 formwork 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 sufficient 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 found by dividing the total of the percentages of materials retained on specified sieves (B. S. sieves Nos. 100, 52, 25, 14, 7,  $\frac{3}{16}$ ",  $\frac{1}{4}$ ",  $\frac{3}{8}$ ",  $1\frac{1}{4}$ ", 3") by 100. It gives an idea of the fineness or coarseness of an aggregate; 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 mix** : 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.

**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.

**Plums** : Hard, clean natural stones used in mass concrete or foundations. The plums should not be larger than one-third of the cross-section of the concrete.

**Pozzolanic** materials, such as surkhi also 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 Section 12.)

**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 of cement, thus increasing the time during which concrete may be worked. Retarders decrease the rate of development of strength and may reduce the ultimate strength by as much as 50 per cent ; they are used only in special circumstances.

*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 rounds generally under  $\frac{1}{2}$ " 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 or different materials in a concrete mix.

*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.

*Tilting mixer* : A mixer the drum of which can be tilted. The materials are fed in when the opening of the drum is raised and the mixture is discharged by tilting the drum.

*Trough mixer* : A mixer having one or two horizontal shafts fitted with a number of tines or blades.

*Trowelling* : Smoothing over the surface of concrete or cement mortar rendering with a flat steel trowel.

*Truck mixer* : A mixer mounted on a self-propelled chassis, capable of mixing materials during transit from a batching plant to the point of placing.

*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").

*Winch* : A small hoist.

*Wire* : Round iron under 3/16" diameter.

*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.

*Workability agents*. Finely divided materials such as fine sand, clay, hydrated lime, talc, and pulverized chalk, etc. are used to improve the workability of mixes deficient in fines and help to reduce bleeding. Only a small quantity, say about 5 per cent of cement, should be used.

*Yield (of concrete)*: The factor obtained by dividing the volume of the mixed concrete as laid and measured in situ by the volume of the coarse aggregate used. Sometimes expressed as the volume of concrete per unit quantity of binder.



## AREA, CIRCUMFERENCE &amp; WEIGHT OF ROUND BARS

Dia. in.	Areas for given number of Bars.														
	3/16"	1/4"	5/16"	3/8"	7/16"	1/2"	5/8"	3/4"	7/8"	1"	1 1/8"	1 1/4"	1 1/2"	1 3/4"	1 7/8"
1	.03	.05	.08	.11	.15	.20	.31	.44	.60	.79	.99	1.23	1.48	1.77	1.77
2	.06	.10	.15	.22	.30	.39	.61	.88	1.20	1.57	1.99	2.45	2.97	3.53	3.53
3	.08	.15	.23	.33	.45	.59	.92	1.33	1.80	2.36	2.98	3.68	4.45	5.30	5.30
4	.11	.20	.31	.44	.60	.79	1.23	1.77	2.41	3.14	3.98	4.91	5.94	7.07	7.07
5	.14	.25	.38	.55	.75	.98	1.53	2.21	3.01	3.93	4.97	6.14	7.42	8.84	8.84
6	.17	.30	.46	.66	.90	1.18	1.84	2.65	3.61	4.71	5.96	7.36	8.91	10.60	10.60
7	.19	.34	.54	.77	1.05	1.37	2.15	3.09	4.21	5.50	6.96	8.59	10.39	12.37	12.37
8	.22	.39	.61	.88	1.20	1.57	2.45	3.53	4.81	6.28	7.95	9.82	11.88	14.14	14.14
9	.25	.44	.69	.99	1.35	1.77	2.76	3.98	5.41	7.07	8.95	11.04	13.36	15.90	15.90
10	.28	.49	.77	1.10	1.50	1.96	3.07	4.42	6.01	7.85	9.94	12.27	14.85	17.67	17.67
Circum- ference	.589	.785	.982	1.178	1.374	1.571	1.964	2.356	2.749	3.142	3.534	3.927	4.320	4.712	4.712
Wt. per ft. lbs.	.094	.167	.261	.376	.511	.668	1.043	1.502	2.044	2.670	3.380	4.172	5.049	6.008	6.008

## SPACING OF ROUND BARS IN REINFORCED SLABS

Sectional area of steel per foot width of slab when spaced as follows

Dia. in.	2"	2½"	3"	3½"	4"	4½"	5"	5½"	6"	7"	8"	9"	10"	12"
1/4	.29	.23	.20	.17	.15	.13	.12	.11	.10	.08	.07	.07	.06	.05
5/16	.46	.36	.31	.26	.23	.20	.18	.17	.15	.13	.11	.10	.09	.08
3/8	.66	.53	.44	.38	.33	.29	.26	.24	.22	.19	.17	.15	.13	.11
7/16	.90	.72	.60	.51	.45	.40	.36	.33	.30	.26	.23	.20	.18	.15
1/2	1.18	.94	.78	.67	.59	.52	.47	.43	.39	.34	.29	.26	.24	.20
5/8	1.84	1.47	1.23	1.05	.92	.82	.74	.67	.61	.53	.46	.41	.37	.31
3/4	2.65	2.12	1.77	1.51	1.32	1.18	1.06	.96	.88	.76	.66	.59	.53	.44
7/8	3.61	2.88	2.40	2.06	1.80	1.60	1.44	1.31	1.20	1.03	.90	.80	.72	.60
1	4.71	3.77	3.14	2.69	2.36	2.09	1.88	1.71	1.57	1.35	1.18	1.05	.94	.78
1½	5.96	4.77	3.98	3.41	2.98	2.65	2.39	2.17	1.99	1.70	1.49	1.33	1.19	.99
1¾	7.36	5.89	4.91	4.21	3.68	3.27	2.95	2.68	2.45	2.10	1.84	1.64	1.47	1.23
1½	8.91	7.12	5.94	5.09	4.45	3.96	3.56	3.24	2.97	2.55	2.23	1.98	1.78	1.48
1½	10.60	8.48	7.07	6.06	5.30	4.71	4.24	3.86	3.53	3.03	2.65	2.36	2.12	1.77

## SECTION 9

### TIMBER STRUCTURES

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## 1. GLOSSARY OF TERMS

A *log* is the trunk of a tree with braches lopped off.

A *balk* or *baulk* is obtained by nearly squaring a log.

*Planking*—When the thickness does not exceed 2" and at the same time the width exceeds twice the thickness.

Timber terms such as, Boards, Battens, Planks, Deals, Scantlings, Laths, Strips, have varying definitions (sizes), therefore while ordering timber scantlings sizes should be quoted.

*Dovetailing*—In carpentry and joinery, the method of fastening boards or other timbers together, by letting one piece into another in the form of 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 timbers or the pieces jointed.

*Fox-tail wedging*—Is a particular mode of morticing in which the end of the tenon is notched beyond the mortise.

*Rebate*—A groove on the edge of a board.

*Sash*—The framework which holds the glass in a glazed window or door.

*Stud* or *Studding*—The small timbers used in partitions and outside wooden walls, to which the laths and boards are nailed:

## 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 9/28 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.

$$\text{Approx. contents of a log in c.ft.} = \text{length} \times \left\{ \frac{\text{mean girth}}{4} \right\}^2$$

$$\text{or } \left\{ \frac{\text{sum of the three girths}}{4} \right\}^2 \times \frac{\text{length}}{7}$$

All dimensions are in feet. Girths are measured at ends and centre.

### Tests for Softwoods or Hardwoods

*Softwoods* have long and narrow pointed leaves and are characterised by distinct annual rings; have straight grains, more uniform texture and light colour. *Softwoods* are very strong for direct pull but weak in resisting thrust or shear. *Hardwoods* have broad leaves, dark colour, are generally dense and have narrow and well defined annual rings; are heavy, strong and hard. *Hardwoods* are capable of resisting all stresses equally well. Colour is a variable feature and no sure guide to its properties of strength and durability but a heavy wood is generally strong and hard.

*Softwoods* are indented across the grain with a pressure of 1000 lbs./sq. in. to a depth of 1/20th inch or more, while *hardwoods* need a pressure of over 1400 lbs./sq. in. for the same indent or less.



**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.

### 3. SEASONING OF TIMBER

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 a foot above floor level with longitudinal and cross pieces arranged one upon another leaving a space of about 2 inches 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.

"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. 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 absorptio of moisture. Well seasoned wood lasts longer under all conditions.

A timber is considered fit for carpenter's work when it has lost  $1/5$ th of its original weight and fit for joiner's work when  $1/3$ rd of its weight has been lost.



## 4. 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 un-ventilated 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 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 negate 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 ground moisture by providing damp-roof 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.

Timber is liable to deteriorate from the time it is felled, and it is useless to treat timber which is intended for structural purposes after it has become infected.

### 5. PRESERVATION OF TIMBER

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 30 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. It is mixed with fuel oil to the extent of 50 per cent by weight which ensures stability to creosote against evaporation and leaching from the treated timber. *Heavy creosote* is a product of coal tar distillation and is generally applied without admixture of oil for brush painting. 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. Since creosote varies widely



in composition, the creosote used for wood preservation under Indian conditions shall conform to IS: 218—1952. 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 thoroughly incorporated in the proportion of 2 lbs. of lime to 1 gall. 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  $\frac{1}{4}$  in. 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, naphtha, kerosine 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 handling 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, minimized 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 in 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 borax and boric acid, ammonium phosphate, sulphate and chloride. The solution used is generally of 2 per cent strength. 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 wood-work of any kind should be laid within 2 ft. of a fire-place or a flue.

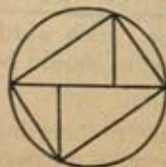
## 6. DESIGN OF BEAMS AND BATTENS

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 page 9/29 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, by using timber packing blocks, or by  $\times$  braces, (called "bridging") at intervals. Roughly, size of a beam is (which may be used for supporting a ceiling as well):—

Depth in inches	= half the span in ft.	For a well proportioned beam $b = 0.6$ to $0.75d$ .
Width	= 2" or more	
Spacing	= 12"	

The strength of a circular beam is only  $3/5$ th 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  $7/10$ th 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.



The width of a beam should not be less than 2" ( $1\frac{1}{2}$ " min.). 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.

**Timber Beams**

Distributed safe loads in lbs. on beams 1 inch wide.:

Table I—Simply Supported—Designed for Deflection with formula:  $150 bd^3/L^2$ 

d in.	Clear span in feet										
	4	5	6	7	8	9	10	11	12	13	14
3	253	162	112	82	63	50	40	33	28	23	20
4	600	384	266	195	150	118	96	79	66	56	48
5	1171	750	520	382	292	231	187	154	130	110	95
6	2025	1296	900	661	506	400	324	267	225	191	165
7	3215	2058	1429	1050	803	635	514	427	357	304	262
8	4800	3072	2133	1567	1200	948	768	634	533	454	392
9	6834	4374	3037	2261	1708	1350	1093	903	759	647	558
10	9375	6000	4166	3061	2343	1852	1500	1239	1041	887	765

Table II—Simply Supported—Designed for Strength with formula:  $180 bd^2/L$ 

d in.	Clear Span in feet										
	4	5	6	7	8	9	10	11	12	13	14
3	405	324	270	231	202	180	162	147	135	124	115
4	720	576	480	411	360	320	288	261	240	221	205
5	1125	900	750	642	562	500	450	409	375	346	321
6	1620	1296	1080	925	810	720	648	589	540	498	462
7	2205	1731	1370	1260	1103	980	882	802	735	680	630
8	2880	2304	1920	1646	1440	1280	1152	1047	960	886	831
9	3645	2916	2430	2083	1822	1620	1458	1325	1215	1122	1041
10	4500	3600	3000	2571	2250	2000	1800	1636	1500	1384	1285

(a) The above tables are worked out for 1 inch width of beam. Width of the actual beam in inches should be multiplied by these figures for full load.

(b) The tables will do for most of the timbers but where the formula varies (for the value given on page 9/28) the same can be multiplied by the table loads and divided by the table co-efficient to obtain correct loads.

(c) 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  $5/4$ .



## 7. DESIGNS OF COLUMNS

The safe working stresses are given at page 9/28 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,  $\frac{3}{4}$  th of the compressive strength should be taken and beyond 15, with the following approximate formula:—

$$\text{Safe compressive stress in lbs} = f \times \left(1 - \frac{L}{60d}\right) = f \times A$$

where:  $f$  = safe compressive stress per sq. in. for short lengths.

Ratios of  $A = \left(1 - \frac{L}{60d}\right)$  for various values of  $\frac{L}{d}$ :—

$\frac{L}{d}$	A	$\frac{L}{d}$	A	$\frac{L}{d}$	A	$\frac{L}{d}$	A
1 to 10	1.00	19	.683	28	.533	38	.366
10 to 15	.750	20	.666	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

The values are always approximate for timber. No wooden column should have  $L/d$  exceeding 45. Some engineers recommend that in no case shall the smallest dimension be less than  $1/30$ th of the height of the post.

Take only  $7/8$ th load where corners are chamfered. Round columns take about 0.77 load of a square column of the same side as diameter.

The following safe loads in lbs. may be allowed on verandah posts, for common timbers:—

Section of Post	Height of Post				
	6'	7'	8'	10'	12'
4" × 4"	7100	6500	6000	4900	3800
5" × 5"	12800	12000	11300	9900	8500
6" × 6"	20000	19200	18200	16500	14700

The following sizes of posts are recommended by the Bombay Municipality for timber-framed buildings:—

	Height of Post	Size
(i) Where the building consists of not more than one storey :—	Not exceeding 11 ft.	5" × 5" or 6" dia.
(ii) Where the building consists of not more than two storeys :—		
Lowest or 1st storey	Not exceeding 11 ft.	6" × 6" or 7" dia.
2nd storey ..	"	5" × 5" or 6" dia.
(iii) Where the building consists of not more than three storeys :—		
Lowest or 1st storey	Not exceeding 11 ft.	7" × 7" or 8½" dia.
2nd storey ..	"	6" × 6" or 7" dia.
3rd storey ..	"	5" × 5" or 6" dia.
(iv) Where the building consists of not more than four storeys :—		
Lowest or 1st storey	Not exceeding 11 ft.	8" × 8" or 9½" dia.
2nd storey ..	"	7" × 7" or 8½" dia.
3rd storey ..	"	6" × 6" or 7" dia.
4th storey ..	"	5" × 5" or 6" dia.

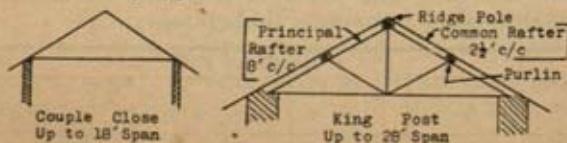
The above sizes are for hardwood, if softwood is used, increase sectional area by 25 per cent. Where the height of posts exceeds 11 ft., the sizes shall be increased by 3/16 of an inch for each additional foot of height. The posts should not be more than 10 ft. apart. Where the distance apart of posts is less or more than 10 ft., 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 5 inches.

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 3 ft. from the corner post in each direction, or alternatively, the corner post should be struted at this distance into the top beam.

## 8. ROOF TRUSSES

(For the design of roofs in general, see Section on "Roofs")

The following types of timber roofs are generally used



A *couple roof* has only two rafters halved and nailed at the top without any tie. This type of roof is suitable

up to 10 ft. 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 16 ft. with 18 ft. 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 14 ft. spans. 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 are recommended :

Span	Brickwork in lime		Stone walling
	Thickness	Height of wall not to exceed	
10 ft.	9"	7 ft.	16" Height must not exceed 15 ft.
	13½"	14 ft.	
18 ft.	9"	5 ft.	
	13½"	10 ft.	

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. These types of roofs should not be spaced more than 3 ft. apart for 30 lbs. and 4 ft. apart for 20 lbs. per sq. ft. roof load.

In constructing trusses a full sized drawing of the truss is first made on a level platform from which templates of all tenons, mortices and scarfs, etc., are made as a guide to ensure all the trusses being of the same size.

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. Min. size of battens is 1" × 1½" and spacing 12" centres. If the battens are of softwood, take min. size 1½" × 1½". Fix max. span for the battens (or boardings) according to the roof load (as simply supported beams).

Spacing of rafters =  $\text{Span}/4 + 6''$ .

Check with span for battens.



Coupled rafters should be spaced the max. distance apart over which battens or boardings will carry the load, generally 2 to  $3\frac{1}{2}$  ft.

Design rafter for deflection as a simple beam with load normal to roof. Span of rafter is  $=L/2 \sec.\phi$ .  $\phi$  is roof angle. For rafters a span of 8 ft. 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 semicontinuity 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.

$$\text{Design tie for tension} = \frac{WL}{4h}$$

$W = w \text{ LD sec.}\phi$ ,

$w = \text{wt. per sq. ft. of roof load (normal to roof),}$

$L = \text{span in ft.,}$

$D = \text{spacing of rafters,}$

$h = \text{height of truss.}$

Section available for tension is the net area at the connection to the rafter, or at the joint if the tie beam is in two pieces.

Min. size of a tie is  $1'' \times 2\frac{1}{2}''$

A tie should have a min. bearing of 8'' on walls or piers.

Ties should not be used to support a ceiling for spans of more than 12 ft. without supporting the ties. Where a ceiling is supported, ties should be calculated for a deflection of  $L/480$ .

Tie rods may be of steel or wrought iron and are preferable to wooden ties. Diameter is  $1/16''$  per ft. 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 (concent-

trated or distributed) as a beam, find out the section required, keeping a constant width, due to this beam 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—20 lbs./sq. ft. of roof surface. Spacing—4 ft. Timber—Sal or Teak		Load—30 lbs./sq. ft. Spacing—2'-9" Timber—Sal or Teak		
Load—20 lbs./sq. ft. Spacing—2'-9" Timber—Deodar or Kail		Load—30 lbs./sq. ft. Spacing—2'-0" Timber—Deodar or Kail.		
Effective span in feet	Rafter	Tie or Collar	Ridge* Pole	Remarks
8	2"×3"	1"×3"	3"×7"	*Rafters are nailed to ridge pole.
10	2"×3½"	1½"×3" or 1"×4"	"	Size of ridge pole is suggested only for common cases. This should be calculated for the load on the roof span as a simple beam
12	2"×4"	"	"	
14	2"×4½"	1½"×3½"	3"×7½"	
16	2"×5"	1½"×4"	3½"×8"	
18	2"×5½"	2"×4½"	"	

### King-Post Truss

King-post trusses are suitable for spans up to 28 ft. with 30 ft. max. King-rod of mild steel or wrought iron is preferable to king-post dia is about  $1/24$  in. per ft. of span. The king-rod has a long thread and can be tightened up, thus bracing the whole truss. It is difficult to get suitable timber for trusses beyond 28 ft. span.

*Ridge Pole* is provided over the king post truss to support the common rafters. Usual size is 3"×6" which should be fixed according to the roof load as a simple beam.

Table on the next page gives sizes of scantlings for different king-post trusses. Cross-sectional areas of scantlings are nearly proportional to the spacing of trusses, viz., if the spacing is made 5 ft. instead of 10 ft., the size of the rafter for 16 ft. span will be about  $2'' \times 5''$  instead of  $4'' \times 5''$  and *vice versa*. If softwood is used instead of hardwood, 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 strap are tightened.

Steel trusses are preferred for spans beyond 25 ft.

Table giving sizes of scantlings for King-Post Trusses, for Hardwoods (Teak, Sal, etc.), Spacing 10 ft., Slope of Roof 1 in 2 :

Span ft.	Principal Rafter in.	Struts in.	Tie Beam		*King Post in.	King Rod dia. in.
			With Ceiling in.	Without Ceiling in.		

**Roof Load 45 lbs./sq. ft.**

14	4×5	4×3	4×5	4×4	4×3	
16	4×5	4×3	4×5	4×4	4×3	
18	4×6	4×3	4×5	4×4	4×3	$\frac{3}{4}$
20	4×6	4×3	4×5	4×4	4×3	
22	4×7	4×4	4×6	4×5	4×4	1
24	4×8	4×4	4×6	4×5	4×4	1
28	4×8	4×4	4×7	4×6	4×5	$1\frac{1}{4}$
30	4×8	4×5	4×8	4×7	4×5	$1\frac{1}{2}$

**Roof Load 35 lbs./sq. ft.**

14	4×4	3×3	4×5	3×4	4×3	
16	4×5	4×3	4×5	4×4	4×3	$\frac{5}{8}$
18	4×5	4×3	4×5	4×4	4×3	
20	4×6	4×3	4×5	4×4	4×3	
22	4×6	4×3	4×5	4×4	4×3	$\frac{3}{4}$
24	4×7	4×3	4×5	4×4	4×4	
28	4×7	4×4	4×6	4×5	4×4	1
30	4×7	4×4	4×7	4×6	4×5	$1\frac{1}{4}$



## Roof Load 25 lbs./sq. ft.

14	4×3	3×3	4×4	3×4	3×3	1
16	4×4	3×3	4×4	3×4	3×3	
18	4×4	4×3	4×4	3×4	4×3	
20	4×4	4×3	4×4	3×4	4×3	
22	4×5	4×3	4×5	3×4	4×3	
24	4×5	4×3	4×5	4×4	4×4	
26	4×5	4×3	4×6	4×4	4×4	
28	4×6	4×4	4×6	4×5	4×4	

Variations in the span, spacing or rise of trusses  $\pm 25$  per cent will make no appreciable difference in the sections of the members.

\*A wooden king-post of the old orthodox practice is now no longer considered to be economical or structurally sound for a roof truss, and is a wasteful design. Where an iron rod is not to be used it is preferable to provide two scantlings of timber, about  $1\frac{1}{2}$ " 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.

*Spacings of King-post Trusses:*

For ordinary loads of 30 to 50 lbs./sq. ft. the minimum spacing may be about:  $\frac{1}{2}(\text{span} + 4)$ .

Spacing is not generally kept more than 10 ft., with 12 ft. max.

For roofs with ceiling (heavy type) about  $\frac{2}{3}$ th of the above spacings may be adopted.

**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 8 to 10 feet apart when the width of the verandah exceeds 8 ft. For verandah widths above 10 feet, it is preferable to strut the principal and hip rafters to the extent of  $\frac{1}{3}$ rd of their length from the wall. The struts to spring from the wall at  $6\frac{1}{2}$  feet from the floor level. In verandahs of widths less than 8 feet,

common rafters are used 15 to 18 inches apart, (with max: up to 3 ft.), 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 be taken for the roof coverings. These should be multiplied by the cosine of the roof angle to obtain weights normal to the roof surface.

	lbs./sq. ft.
Asbestos sheets $\frac{1}{4}$ " thick (corrugated)	.. 3.3
—do— .. (flat) ..	.. 2.3
C.G.I. sheets 24 gauge .. ..	.. 1.5
C.G.I. sheets 22 gauge .. ..	.. 1.75
C.G.I. sheets 18 gauge .. ..	.. 2.75
Bituminous roofing felt .. ..	.. 1.5
Glazed roofing (with $\frac{1}{4}$ " glass with lead covered steel bars) .. ..	.. 6
Boarding 1" thick .. ..	.. 3
Boarding $\frac{3}{4}$ " thick .. ..	.. 2
Eternit sheets or tiles .. ..	.. 2.5
Slates on battens .. ..	.. 7
Slates on 1" boarding .. ..	.. 12
Thatch with frame, 9" .. ..	.. 10
Thatch with frame, 6" .. ..	.. 6.5
Timber trusses light roofs .. ..	.. 2
Timber trusses heavy roofs .. ..	.. 3
Rafters .. ..	.. 1
Battens .. ..	.. 0.5
Single Allahabad tiles including battens .. ..	.. 17
Double Allahabad tiles including battens .. ..	.. 34
Mangalore Tiles .. ..	.. 11
Mangalore tiles with flat tiles .. ..	.. 16
Mangalore tiles bedded in mortar over flat tiles .. ..	.. 22
Mangalore tiles with battens .. ..	.. 14
Flat and pan tiles .. ..	.. 30
Plain pan tiles .. ..	.. 22
Country tiles with battens, single .. ..	.. 14
Country tiles with battens, double .. ..	.. 24

Allowance for weight of truss, purlins, rafters, etc., may be taken at 5 lbs./sq. ft. for roofs and 6 lbs./sq. ft. for heavy roofs, of covered area.

Or, the weight of truss may be taken :—

$$\frac{1}{2} \left( 1 + \frac{\text{span}}{10} \right) \text{—lbs./sq. ft. of covered area.}$$

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 1½
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 1½
Slates .. .. .	1 in 2	1 in 2½
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  $\frac{1}{3}$ rd the depth of the timber.

*Fish Joints* are made to resist tension. For lengthening ties a plain fixed joint is most economical. The two ends of the beam are butted 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, and any hard stopping should be done with tight driven plugs.

### Iron Stirrups and Straps

Stirrups and straps may be made out of  $2'' \times \frac{3}{4}''$  bars for trusses of spans 20 ft. and over, and of  $2'' \times \frac{1}{2}''$  bars for trusses below 20 ft. span. Three-way (T) straps may either be cut from solid plate or welded; they are made of wrought iron and fixed on both sides. Bolts are generally of  $\frac{3}{4}''$  dia. and fixed about 3 to 4 inches centres. Stirrup ends are forged out of the heel strap which will give a size of  $\frac{3}{4}''$  dia. and  $\frac{7}{8}''$  dia. from  $2'' \times \frac{1}{2}''$  and  $2'' \times \frac{3}{4}''$  bars respectively. A long washer is provided at the top and the strap tightened with nuts.

Bolts in timber members should not be placed nearer the edges of members than 2d, nearer the ends of ties than 6d, closer together in the line of stress than 6d, or closer together transversely to the stress than  $4\frac{1}{2}$ d. All bolts must be provided with two washers. Bolt holes in timber and straps should be driven fit and not loose.

For estimating purposes, the weights of stirrups, straps, bolts, nuts and washers, etc., may be taken  $\frac{3}{8}$  cwt. per truss for spans of 20 ft. and above, and  $\frac{1}{8}$  cwt. for spans under 20 ft. Weight of the king-rod is extra.

**Wall Plates** should be securely fastened to the wall by  $\frac{3}{4}''$  dia. anchor bolts with 1'-6" lengths embedded in the wall with plate washers at the ends, at intervals of 5 ft. Size of the wall plates may be  $4'' \times 3''$  for spans up to 12 ft. and  $6'' \times 4''$  above 12 ft. spans. Bed plates should be provided under heavy trusses.

Heavy trusses must be anchored to the walls at each end with about  $\frac{3}{4}''$  dia. bolts 3 ft. long, with  $4'' \times 4'' \times \frac{1}{2}''$  washer plates at the ends.

## 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  $\frac{3}{8}$ -in. 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.

## 9. PARTITIONS

Sizes of the principal members of the frames may be taken as follows :—

for spans not exceeding	20 ft.....	4"×3"
" " " " "	30 ft.....	5"×3½"
" " " " "	40 ft.....	6"×4"

The filling in pieces (called "studs" or "quarterings") need not be thicker than 2", 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 3 ft. or 4 ft. 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 12" to 18" centres ; in a brick nogged partition they should be 18"×2'-3" to 3'-0" apart.

**Trellis Work.** Wooden battens  $1\frac{1}{2}$ " to  $2\frac{1}{2}$ " wide by  $\frac{1}{2}$ " to 1" thick are crossed diagonally in opposite directions or at right angles, leaving 2" openings in between them, and nailed to frames.

## 10. FLOORS (Hardwood)

Floor boards should be  $1\frac{1}{4}$ " to  $1\frac{1}{2}$ " thick, 4" to 6" wide and 6' to 12' long ; will rest on joists, usually 15" apart.

For deodar, kail or chir wood, the boards should not be less than  $1\frac{1}{4}$ " thick and not more than 6" wide and not more than 10 ft. long. For teak wood size may be, 1"



thick and 4" wide. More usual size is 12"  $\times$  4" by 6 ft. to 10 ft.

**Joints.** Sides to be tongued and grooved or tongued and ploughed. Ends will rest on the joist below and will break joint with one another. Heads of all screws to be countersunk ; two 3-inch screws should be used for each 6-inch plank wherever it crosses or ends on joist.

### Single Floors :

Span for bridging joists	12 to 15 ft.	} Breadth is usually .25 to .33 of depth.
Spacing centre to centre	12 to 15 ins.	
Width .. .. .	2 to 2½ ins.	
Depth in inches: $\frac{\text{span in ft.}}{2} + 2$ ins.		
Bearing on wall plates ..	4½ ins.	

When bridging joists exceed 8 ft. in length, they should be strutted apart at intervals of 4 to 6 ft. by struts 1" wide and of full depth of the joist, fixed at right angles to the joists, or 1½"  $\times$  3" fixed by herring-boning. (The struts are fixed between the joists.)

### Double Floors :

Binders .. .. .	6 to 8 ft. c. to c.
Bearings on templates ..	6 ins.

Bridging joists rest on binders to which they are notched.

**Framed Floors.** Girders are placed 6 to 10 ft. c. to c., binders are tusk tenoned into them ; as this method weakens the girders it is preferable to use iron stirrups. To obtain max. stiffness breadth should be 5/7th of the depth. Sizes should be calculated as simple beams. Bearings on wall plates to be 9" to 12".

Floors may be made high up in the centre by about ¾" per 20 ft. 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 twenty ; or designed with reduced fibre stress as follows :



Percentage of reduction :—

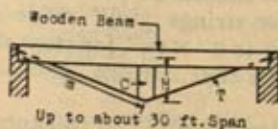
Ratio of length to width	20 to 30	30 to 40	40 to 50	50 to 100
Percentage reduction	25	35	40	50

The ends of all timbers set in masonry should have a space of  $\frac{1}{4}$ " 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 1'-6" to 2'-0" 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.

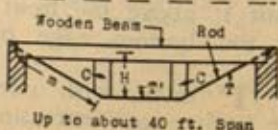
## II. TRUSSED BEAMS



$$\text{B.M. in top beam} = \frac{WL}{32}$$

$$\text{Tension } T = \frac{Ww}{4H}$$

$$\text{Compression } C = \frac{5}{8} W$$



If the load is concentrated  $W$  in the centre, the stresses will be half.

$$\text{B.M. in top beam} = \frac{WL}{16}$$

$$\text{Tension } T = \frac{3Ww}{8H}$$

$$T' = \frac{WL}{8H} \quad C = \frac{3}{8} W$$

$W$  is uniformly distributed load over the girder.

$$H = \frac{L}{8} \text{ to } \frac{L}{12} \quad L \text{ is Span.}$$

The beams may be composed of a single timber or of two or more timbers placed side by side.

(Some books give slightly different values.)

## 12. STAIRCASES

Hardwood should be used for staircases, especially for 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 4 ft. 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 :

3'-6" width of stairs 10 or 11 step flight, one carriage piece 2" × 4", increasing for 4'-6" wide stairs, 14 or 15 step flight to 2" × 7". When the width of staircase exceeds 8 ft. two carriage pieces should be provided.

*Strings* : 2" × 13" for close strings 3'-6" wide stairs 10 or 11 step flight, increasing to 2" × 15" for close strings (wall string) and 3" × 15" for cut strings 4'-6" wide stairs 14 or 15 step flight.

The strings should be not less than 1½" in thickness and of such breadth as will permit 1" above the front edge of the tread and 1" below the bottom edge of the riser.

*Treads and Risers*: Treads 1½" thick, with risers 1" thick for 3'-6" and 4' wide stairs each increasing by ¼" up to 1¾" and 1½" respectively maximum for every 6" extra width of stairs over 4 ft. The tread is generally made to project about ¾" to 1" 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".

**13. DOORS AND WINDOWS**  
Table showing Size of Frames and Parts for  
Doors and Windows

Particulars (a)	Size of frames (chowkats)	Thickness of leaves	Width of rails, styles, ledges or braces
<b>Doors :</b>			
<i>Glazed, framed, panelled or battened</i>			
Double, up to 4' x 7'	3" x 4½"	1½"	3½"
" exceeding 4' x 7' up to 5' x 8' ..	3½" x 4½"	2"	4"
" exceeding 5' x 8' ..	3½" x 5"	2"	4½"
Single, up to 3' x 6½' ..	3" x 4½"	1½"	4"
" exceeding 3' x 6½' ..	3" x 4½"	2"	4½"
<i>Ledged and Braced</i>			
Double, up to 4' x 7' ..	3" x 4½"	2½"(b)	4"
" exceeding 4' x 7' ..	3½" x 4½"	2½"(b)	4½"
Single, up to 3' x 6½' ..	3" x 4½"	2½"(b)	4"
" exceeding 3' x 6½' ..	3" x 4½"	2½"(b)	4½"
<i>Ledged and Country doors</i>			
Double or single sizes ..	3½" x 4½"	2½"(b)	4"
<i>Wire gauzed*</i>			
Double, up to 4' x 7' ..	3" x 5" (c)	1½"	4"
" exceeding 4' x 7' ..	3½" x 5½" (c)	1½"	4½"
Single, up to 3' x 6½' ..	3" x 5" (c)	1½"	4"
" exceeding 3' x 6½' ..	3½" x 5½" (c)	1½"	4½"
Iron-barred ..	6" x 4½"		
Garage doors ..	..	2½"(d)	6"
<b>Windows :</b>			
Glazed double, up to 3' x 5'	3" x 3½"	1½"	2½"
" up to 4' x 5'	3" x 4"	1½"	2½"
" exceeding 4' x 5'	3" x 4½"	2"	3"
Glazed single, up to 2' x 5'	3" x 3½"	1½"	3"
" exceeding 2' x 5'	3" x 4"	1½"	3"
Iron-barred up to 2½' x 3½'	4" x 3"	..	..
" up to 4' x 5' ..	5" x 3"	..	..
" with plank shutters	5" x 5"	..	..
Fanlights .. ..	(e)	1½"	2½"
Clerestory widows ..	3" x 3"	1½"	3"
Battened double, all sizes	3" x 4"	2½"(b)	3"
" single all sizes	3" x 4"	2½"(b)	3"
Wire gauzed up to 2' x 5'	3" x 4½" (c)	1½"	3"
" exceeding 2' x 5'	3" x 5"	1½"	3½"
" shutters to clerestory windows. ..	2" x 2½"	1½"	2½"

\*If X.P.M. is used it may be of size ½" S.W.M. ½" x ½".



For the size of frames given in the table the first dimension is the 'depth' the dimension at right angle to the wall plane, and the second dimension is the 'breadth' the dimension in the plane of the wall.

Sizes considered are for Deodar wood generally.

*Remarks :*

(a) Dimensions given are out to out of the frame containing the leaves.

(b)  $1\frac{1}{2}$ " ledges and braces and 1" battens.

(c) Dimension given is for a frame carrying two sets of leaves.

(d)  $1\frac{1}{2}$ " ledges and braces and 1" battens.

(e) Size of the frame to be the same as for the main door or window.

Iron-barred shutters: styles, top bottom and lock-rails to be  $2\frac{1}{2}$ " thick, framed together and filled in with round iron bars  $\frac{3}{4}$ " to 1" diameter or  $\frac{3}{4}$ " square bars fixed at 4 to 6 inches central distances apart. The bars to be housed into the top and bottom rails to a depth of 1 inch and to be passed through the lock-rail.

Internal doors should not, as a rule, be less than 2'-9" (min. 2'-3") wide by 6'-6" high. A common rule for the size of doors is: height=width+ $3\frac{1}{2}$  to 3 ft. Doors of greater width than 3 ft. are generally made in two leaves. The size of doors named is the size of the clear opening between the door frames, the size of the opening through the wall being greater.

Middle of lock rail is 2'-6" above floor level. Where mortice locks are used, the lock rail will be not less than 8 inches wide.

All styles and rails should be properly and accurately mortised and tenoned. 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. All rails over 7 inches in depth should have double tenons. Framed joints should be coated with white lead before the frames are put together.

Jambs for doors and windows may be splayed 2 or 3 inches to the foot on the inner sides in case where a door

or window opens inward. Splayed jambs are not suitable unless there is a good interval between the windows or doors as they are weaker than square jambs.

### Venetian Doors and Windows

Blades are generally  $3\frac{1}{2}$ " wide by  $\frac{3}{8}$ " thick and overlap about half their width ; they are secured to a moulded stanchion by 1" 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  $\frac{3}{8}$ " diameter by  $\frac{3}{4}$ " long, and fit into holes in the rebated portion of the frame.

### Windows

Normal height of windows above floor is 2'-6".

In *clerestory windows* the leaves are hung 1" off-centre to make them self-closing.

**Hold Fast**s can be made from  $1\frac{1}{2}$ "  $\times$   $1\frac{1}{2}$ " steel bars bent over at both ends leaving  $13\frac{3}{4}$ " clear length between bends. One end should have a screwed hole to which the frame is secured by a bolt  $\frac{1}{2}$ " in diameter. Give an additional hold-fast on each side where no sill has been provided.

## Safe Working Stresses for Different Kinds of Timber in

Trade Name	Weight lbs./c.ft.	Tensile stress in Bend- ing	Shear		Comp-
			Hori- zontal	Along grains	Parallel to grain
1	2	3	4	5	6
Fir, Partal ..	29	1100	85	120	850
Babul, Kikar ..	52	2600	220	315	1600
Haldu .. ..	42	1950	135	190	1200
Toon .. ..	33	1450	105	150	900
Deodar .. ..	35	1450	100	145	1100
Walnut, Akhrot ..	36	1650	120	170	950
Benteak .. ..	42	1950	130	185	1250
Lendi, Venteak ..	46	2050	155	220	1250
Spruce .. ..	30	1100	185	120	800
Kail, Blue Pine ..	32	950	80	115	750
Chir .. ..	36	1200	90	130	900
Burma Paudak ..	54	3200	190	275	2050
Bijasal .. ..	50	2100	135	190	1300
Sal (M.P.) .. ..	50	2400	135	190	1500
Sal (Bengal, U.P.)	55	2850	180	260	1900
Teak (S. India) ..	42	2300	140	200	1500
Teak-M.P. Bombay	39	2000	140	200	1250
Arjun .. ..	50	1750	160	230	1050
Sain .. ..	55	2200	145	210	1350

- (a) The strength of the timbers given in the table provide Where the timber is of exceptionally good quality or accordingly.
- (b) The recommended safe working stresses are for dry or protected from weather. For works built in out-wetting and quick drying, the safe tensil stresses (Col. by 8/9. Where the timbers are in continuously wet respectively.

\* "India Forest Records"—The Forest Research Institute, Dehra Dun



## lbs. per sq. inch and Co-efficients for Design of Beams

Perpendi- cular to grain	Modulus of Elasticity in 1000 lbs./sq. in.	Design for		EXPLANATION: Col.
		Strength with $\frac{bd^2}{x}$	Deflection with $\frac{WL^3}{y}$	
7	8	9	10	
230	1340	120	160	3. Is max. fibre stress for design of beams and other tension members. 4. For design of beams. 5. For design of fastenings, bolts and dowel joints. 6. For design of short columns. Bending moment values have been taken for simply supported beams with $\frac{WL}{8}$
930	1540	290	190	
520	1300	210	160	
370	1080	160	130	Where W is in lbs., L is in ft., b and d are in inches. Deflection is taken $L/360$ . For round beams multiply the strength and deflection equations by 1.7 which will give the vales for $d^3$ and $d^4$ .
380	1350	160	160	
330	1300	180	160	
590	1570	210	190	Where beams are subject to continuous heavy loading the Modulus of Elasticity should be reduced by 75 to 50 p.c. of the tabulated values or the beams should be calculated for lesser deflection than $L/360$ .
530	1560	240	190	
240	1310	120	160	
240	970	105	120	
320	1390	130	170	
1130	1900	360	230	
580	1460	240	180	
650	1800	260	220	
1160	2040	310	250	
630	1600	260	200	
570	1360	220	160	
740	1100	190	130	
780	1580	250	240	

for ordinary good class timbers having only minor defects. bad quality, the strength should be increased or decreased

inside locations, where the timbers will remain continuously side locations or where timbers are occasionally subjected to 3) should be multiplied by  $5/6$  and safe compression (Col. 6) position, the stresses should be multiplied by  $2/3$  and  $8/11$ .

Values of  $bd^2$ 

d"	b"														
	1	1½	1½	2	2½	2½	3	3½	4	4½	5	5½	6		
1	0.7	1.2	1.5	1.8	2.0	2.3	2.5	2.8	3.0	3.5	4.0	4.5	5.0	5.5	6.0
1½	1.2	2.0	2.3	2.7	3.1	3.5	3.9	4.3	4.7	5.5	6.2	7.0	7.8	8.6	9.4
2	1.7	2.8	3.4	3.9	4.5	5.1	5.6	6.2	6.7	7.9	9.0	10.1	11.2	12.4	13.5
2½	2.3	3.8	4.6	5.4	6.1	6.9	7.7	8.4	9.2	10.7	12.3	13.8	15.3	16.8	18.5
3	3.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	14.0	16.0	18.0	20	22	24
3½	3.8	6.3	7.6	8.9	10.1	11.4	12.7	13.9	15.2	17.7	20	23	25	28	30
4	4.6	7.8	9.4	10.9	12.5	14.1	15.6	17.2	18.7	22	25	28	31	34	37
4½	5.7	9.5	11.3	13.2	15.1	17	18.9	21	23	26	30	34	38	42	45
5	6.7	11.2	13.5	15.7	18	20	22	25	27	31	36	40	45	49	54
5½	9.2	15.3	18.4	21	24	28	31	34	37	43	49	55	61	67	73
6	12.0	20	24	28	32	36	40	44	48	56	64	72	80	88	96
6½	15.2	25	30	35	40	46	51	56	61	71	81	91	101	111	121
7	18.7	25	31	37	44	50	56	62	69	87	100	112	125	137	150
7½	23	30	38	45	53	60	68	76	83	106	121	136	151	166	181
8	27	36	45	54	63	72	81	90	99	126	144	162	180	198	216
8½	32	42	53	63	74	84	95	106	116	148	169	190	211	232	253
9	37	49	61	73	86	98	110	122	135	171	196	220	245	269	294
9½	43	56	70	84	98	112	127	141	155	197	225	253	281	309	337
10	48	64	80	96	112	128	144	160	176	224	256	288	320	352	384
10½	61	81	101	121	142	162	182	202	223	283	324	364	405	445	486
11	75	100	125	150	175	200	225	250	275	350	400	450	500	550	600
12	108	144	180	216	252	288	324	360	396	504	576	648	720	792	864





**Hinges.** 2" screws are used with 5-in. and 6-in. hinges and 1" screws with smaller sizes.

### Rendering Panes of Glass Opaque :

White lead	1 lb.	}	Mix the whole till becomes plastic.
Linseed oil	4 oz.		
Varnish	1 oz.		

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.)

### Wire-Gauze

*Fly-proof gauze*—9 or 10 mesh to the linear inch with 20 or 22 gauge wire.

*Mosquito-proof gauze*—14 or 16 mesh to the linear inch with 28 to 30 gauge wire.

Fly-proof wire gauze admits only 50 to 56 per cent of light and mosquito proof 64 to 68 per cent.

Wire gauze 12×12 meshes to the sq. inch made from 22 gauge wire is used for general purposes.

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## SECTION 10

### COLUMNS & STRUTS

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(For Masonry, R.C.C. and Timber Columns see under  
    respective Sections)

### General Definitions of Terms

*Column* is a general term applied to vertical members supporting a load the length of which exceeds three times the least lateral dimensions. It is also defined as "an isolated vertical load bearing member."

*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.

*Strut* is a pillar, stanchion, column or other vertical or inclined compression member.

*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 crippling due to an axial load.

*Sectional areas.* The 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, etc., where specified to be deducted. The diameter of rivet and bolt holes for deduction shall be assumed to be 1/16 in. in excess of the normal diameter of the rivet or bolt.

### 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.

Slenderness ratio

$$= \frac{\text{effective length or unsupported length}}{\text{least radius of gyration}} = \frac{l}{r}$$

$$\text{Radius of gyration} = \sqrt{\frac{I}{A}} \text{ or } I = A r^2$$



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 axes 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 single angles 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  $\frac{1}{3}$ rd 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.

#### **Maximum Slenderness Ratio of Steel Struts :**

The ratio of effective length to the least radius of gyration of any strut should 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 of  $l/r$  more than 150 for main members and 200 for secondary members in compression.

*Slenderness Ratio for Bridge Structures :*

$l/r$  should not exceed 100 for main members and 125 for bracings.

**Euler's and Rankine's** formulae are not now much in use for practical works, therefore, have been omitted.

Safe working stresses in tons/sq. in. of gross sectional area for axial compression loads on struts:—

$l/r$	New steel	Old steel	$l/r$	New steel	Old steel	$l/r$	New steel	Old steel
0	9.00	8.00	70	5.60	5.41	150	2.30	2.02
10	8.51	7.40	75	5.36	5.15	160	2.06	1.81
15	8.27	7.29	80	5.12	4.88	170	1.86	1.62
20	8.03	7.17	85	4.87	4.60	180	1.68	1.46
25	7.78	7.05	90	4.62	4.33	190	1.52	1.33
30	7.54	6.92	95	4.37	4.06	200	1.39	1.21
35	7.30	6.79	100	4.13	3.81	210	1.27	1.10
40	7.06	6.64	105	3.90	3.57	220	1.17	1.07
45	6.81	6.48	110	3.67	3.34	230	1.08	0.93
50	6.57	6.30	115	3.46	3.13	240	0.99	0.86
55	6.33	6.10	120	3.26	2.93	250	0.92	..
60	6.09	5.89	130	2.89	2.58	300	0.65	..
65	5.84	5.66	140	2.57	2.28	350	0.49	..

$l$  is the effective length in inches and  $r$  the least radius of gyration in inches. Intermediate values may be determined by interpolation.

**New Steel**—Structural Mild steel manufactured according to B.S. Specification 15 : 1948.

**Old Steel**—Structural Mild steel manufactured according to B.S. Specification 15 : 1936.

**Cast Iron Columns.** No cast iron column should have an unsupported length greater than 80 times its least radius of gyration. The following working stresses may be taken for cast iron columns for various ratios of  $l/r$ , working stresses usually used in compression free from flexure are  $\frac{1}{4}$  to  $\frac{1}{8}$  for dead weights, and  $\frac{1}{8}$  for columns free from vibrations, of the ultimate strength.



$L/r$ ratio	15	20	25	30	35	40	45
Working Stress	4.30	4.16	3.86	3.75	3.54	3.33	3.12
$L/r$ ratio	50	55	60	65	70	75	80
Working Stress	2.92	2.70	2.50	2.29	2.08	1.87	1.66

$L$  is the actual length of column in inches, and  $r$  the least radius of gyration in inches.

### I-Section Steel Columns Encased in Concrete

If a steel strut of I-section is solidly encased in cement concrete of 1 : 2 : 4 mix with a cover of at least 2 inches all round its surfaces and edges (so as to increase the width of the flange by 4 inches) and the casing is reinforced horizontally with binding wires of at least  $\frac{3}{16}$  in. dia. at 6-in. pitch in the form of stirrups, the least radius of gyration may be taken equal to  $\frac{1}{3}$ th of the width of the encased column in inches, i.e.,  $\frac{1}{3}$  (width of the steel flange + 4 ins.) The load on the encased section shall not exceed 50 per cent of the load permitted on the uncased steel section nor the ratio  $l/r$  of the steel section exceed 250.

The min. thickness of flange or web plates of a built-up column in a multi-storey building should be not less than  $\frac{3}{8}$  in. In all other cases, steel should be not less than  $\frac{1}{4}$  in. thick except in the case of standard rolled steel sections.

### Effective Length of Struts

Type of Strut	Effective length $l$ of strut
Both ends held in position and restrained in direction (fixed ends).	0.7 $L$
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 $L$



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.

### 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 in inches of a steel rectangular slab can be found from the formula :—

$$t = \sqrt{\frac{3p}{f} \left( Y^2 - \frac{y^2}{4} \right)}$$

$t$ =slab thickness in inches;  $p$ =pressure or loading on the base in tons per sq. inch ;  $f$ =permissible bearing in steel (12 tons/sq. in.)  $Y$ =greater projection of the plate beyond the stanchion in inches;  $y$ =lesser projection of the plate beyond the stanchion in inches.

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  $1\frac{1}{2}$  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 moment 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

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  $1\frac{1}{2}$  ft.) at any storey level. The ends of the sections to be jointed 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 12 ins. min. : The thickness of the stone cap or template should not be less than  $\frac{1}{3}$ th of the length of its side, or less than  $1\frac{1}{2}$  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 (per sq. inch). The sum of these stresses at the extreme fibre shall not exceed  $T$  :

$$\text{where } T = d + c \left\{ 1 - \frac{d}{p} \right\} \left\{ 1 - .002 \frac{l}{r} \right\}$$

$d$  = total axial load on column in tons divided by the gross cross sectional area of the column in sq. inches;  
 $c = 7.5$  for mild steel,  $p$  = working stress for axial



compression load per sq. in., specified in the table at page 10/4.

The stress can be worked out as usual from the equation :

$$f = \frac{W}{A} \pm \frac{M}{Z}$$

The allowable stress specified in the table at page 10/4 for the factor of  $l/r$  can be increased to  $T$  owing to part of the total stress  $W/A + M/Z$  being due to bending. It should also be checked for the allowable working stress under direct load as the section should not be smaller than for under direct load.

Or alternatively, members subject to both axial compression and bending stresses shall be so proportioned that the quantity

$\frac{f_1}{F_1} + \frac{f_2}{F_2}$  does not exceed unity: where— $f_1$  = the axial compressive stress;  $F_1$  = the permissible compressive stress in axially loaded struts (see table at page 10/4);  $f_2$  = the sum of the compressive stresses due to bending about both rectangular axes;  $F_2$  = the min. permissible compressive stress for members subject to bending (10 tons/sq. in.):

If the bending stress is induced due to wind, the working stress may be increased by 33 per cent.

Example of an eccentrically loaded column:

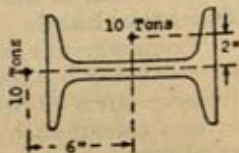
A 10" × 8" R.S.J. of height 20 ft. is to be loaded as indicated with ends adequately restrained as both ends in position but not in direction.

Area of R.S.J. = 16.18 sq. in.

Least radius of gyration = 1.84

Modulus XX = 57.74

Modulus YY = 13.69



Stress due to direct load =  $d = \frac{10 + 10}{16.18} = 1.24$  tons/sq. in.

Stress due to bending =  $\frac{10 \times 6}{57.74} + \frac{10 \times 2}{13.69} = 2.50$  tons/sq. in.

Total stress = 3.74 tons/sq. in.

$$\frac{l}{r} = \frac{20 \times 12}{1.84} = 130$$



Allowable axial stress with  $l/r$  130 is 2.58 tons/sq. in. (for old steel)

This stress can be increased to  $T$  owing to part of the total stress of 3.74 tons/sq. in. being due to bending.

$$T = 1.24 + 7.5 \left\{ 1 - \frac{1.24}{2.58} \right\} \left\{ 1 - .002 \times 130 \right\} = 4.09$$

The actual stress of 3.74 is less than this, therefore, safe.

**Lattice Bars** are provided to reduce the slenderness ratio of a long strut, and for connecting together two or more sections. The thickness of flat lacing bars shall be not less than one-fortieth of their length for single lacing, and one-sixtieth of their length for double lacing. Single lattice bars must not be longer than fifty times their thickness, and double lattice bars seventy five times their thickness. 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 45 deg. to the axis of the member.

Where a column or strut is built up of separate members laced together the max. spacing of lacing bars shall be such that the slenderness ratio,  $l/r$  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 the lesser, where  $l$  is the distance between the centres of the pairs of connections of the lattice bars to each component.

The lacing system, as far as is practicable, shall not be varied through the length of the strut. Where welded lacing bars overlap the main members, the amount of lap shall be not less than 4 times the thickness of the bar or mean thickness of the flange of the member to which the bars are attached, whichever is the lesser.

Where laced compression members are provided with *tie plates* at the ends of the lacing systems, the thickness of the tie plates shall be not less than one-fiftieth of their length.



## SECTION 11

### ROOFS

(For Timber Roofs, see under "Timber Structures.")

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## 1. LOADINGS FOR DESIGN OF ROOFS

The following loads should be taken into account for the design of roofs :—

(i) **Superimposed loads:** Comprise all loads other than dead loads such as live loads, snow loads and wind loads.

(ii) **Dead loads** due to the weight of the roofing materials such as trusses, purlins, battens, sheets or slabs, tiles, ceilings or any other fixtures.

**Imposed Loads other than Wind Load***Live Load*

(a) For a roof to which access is provided other than that necessary for cleaning and repair, (i.e., a roof which is to be used for sitting or sleeping), the minimum imposed load shall be 30 lbs. sq. ft. measured on plan, subject to a minimum load of \*240 lbs. per ft. width of the roof slab or covering uniformly distributed over the span, and 1920 lbs. uniformly distributed over the span on any beam or truss.

(b) For a roof to which no access is provided other than that necessary for cleaning and repair, the minimum imposed load where the slope of the roof does not exceed 30 deg. shall be 15 lbs./sq. ft. measured on plan, and for a roof with a slope greater than 30 deg. the minimum imposed load shall be 15 lbs. less 1 lb. for every 3 deg. by which the slope exceeds 30 deg. No imposed load is necessary for a roof with a slope greater than 75 deg., or a flat sheeted roof.

*Snow Loads*

Snow weighs 5 to 12 lbs. per c. ft. when fresh and 15 to 50 lbs. per c. ft. when compacted. Snow loads where applicable shall be taken at 10 lbs./sq. ft. per foot depth of snow anticipated.

Where snow is likely to occur, a min. value of 25 lbs./sq. ft. of horizontal covered surface should be taken for all slopes up to 20 deg.; this load may be reduced 1 lb. for each degree of slope above 20 deg. No allowance

---

\*That is, min: load should be 240 lbs. per ft. width of slab for all spans up to 8 ft. and 1920 lbs. for all beams (supporting slabs or coverings) up to an area of 64 sq. ft. The load of 30 lbs./sq. ft. becomes operative at spans of 8 ft. and on areas of 64 sq. ft.

for snow need be made for roofs sloping 1 to 1 or steeper. Make allowance for snow on both sides of a sloping roof.

### WIND PRESSURE

Description of wind	Velocity in miles per hour
Strong breeze	27
Moderate gale	35
Strong gale	50
Whole gale (trees uprooted, structures damaged)	59
Storm (wide spread damage)	68
Hurricane	over 75
Violent „	100

Total pressure of the wind on a surface normal to the current of the wind is :—

$$P = 0.00256kV^2$$

where:

$P$  = pressure in lbs./sq. ft.,

$k$  = co-efficient depending on the shape and dimension of the surface,

$V$  = velocity of the wind in miles per hour at the height of the surface.

V	50	55	60	65	70	75	77	80	85	90	95	100	105	109	113	116
P	8	10	12	14	17	19	20	22	25	28	30	34	38	41	43	46

$V$  = velocity in miles per hour,  $P$  = horizontal pressure due to wind in lbs./sq. ft. of surface exposed.

Average Wind Pressures According to Height on a Vertical Surface :—

H	0	10	20	30	40	50	60	70	80	90	100	125	150	175	200	250	300	350	400
P	8	12	15	18	20	22	23	25	26	28	29	31	34	36	38	41	43	46	48

$H$  = average height in feet of the exposed surface above the mean retarding surface,  $P$  = total horizontal effect (pressure) of wind in lbs./sq.ft.

The above wind pressures should be doubled for works along a 50 mile wide coastal belt areas subject to heavy storms, and increased by 50 per cent for works on hills according to the height and locality, and halved for the central planes.

When calculating wind pressure on tall structures, the total pressure to be adopted should be worked out in successive slabs of 10 ft. 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 coastal 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 :

$$P_n = P \times \frac{2 \sin \theta}{1 + \sin^2 \theta}; P_v = P_n \times \cos \theta; P_h = P_n \times \sin \theta.$$

$P$  = wind pressure in lbs./sq. ft. of vertical surface,

$P_n$  = corresponding normal pressure per sq./ft. of roof surface,  $P_v$  = vertical component of  $P_n$ ,  $P_h$  = horizontal component of  $P_n$ ,

$\theta$  = angle of roof slope with the horizontal.

Pressure of wind on roofs: (based on the above formulae).

Slope of Roof $\theta$	5°	10°	1 in 3 18°-26'	20°	1 in 2 21°-48'	1 in 1 26°-34'	30°	1 in 1 33°-41'	40°	1 in 1 45°	60°	75°
$P_n = P \times$	.173	.337	.575	.612	.653	.745	.800	.848	.910	.943	.990	.999
$P_v = P \times$	.172	.332	.545	.575	.606	.667	.693	.706	.697	.667	.495	.259
$P_h = P \times$	.015	.059	.182	.209	.242	.333	.400	.471	.585	.667	.857	.965

Pressure of wind on roofs for 1 : 2 slope :—

Wind pressure in lbs./sq.ft.									
	5	10	15	20	25	30	40	50	60
$P_n$	3.73	7.45	11.2	14.9	18.6	22.4	29.8	37.3	44.7
$P_v$	3.33	6.67	10.0	13.3	16.7	20.0	26.7	33.3	40.0
$P_h$	1.67	3.33	5.00	6.67	8.33	10.0	13.3	16.7	20.0

Although the above formulae are not much in use now and have been replaced by "Codes", but trusses designed on these principles have stood well, and they give results on the safe side



The following rule is in accordance with IS : 456—1953 and B S : 449—1948 :—

For the vertical surfaces of buildings, the unit wind pressure  $P$  in the table at page 11/3 shall be taken as a pressure of  $0.5 P$  on the windward surface and a suction of  $0.5 P$  on the leeward surface. A surface inclined at an angle of  $70^\circ$  or over with the horizontal shall be deemed to be vertical for calculating the pressure and suction.

The wind pressure and suction normal to the roof surface shall be found by multiplying the unit pressure  $P$  given in the table at page 11/3 by the factors given below, a negative factor denoting suction :

Slope of roof	Windward slope of pitched roof (or half of flat roof)	Leeward slope of pitched roof (or half of flat roof)
$0^\circ$	-1.0	-.050
$22\frac{1}{2}^\circ$	-0.25	(Wind pressure on windward side of a sloping roof of less than $30^\circ$ is now taken as a "suction".)
$30^\circ$	0	
$45^\circ$	+0.25	
$70^\circ$ & above	+0.50	

The wind pressure for intermediate roof slopes may be interpolated linearly.

It is recommended in the IS : 456—1953 that in no case should the wind pressure or suction be taken as less than 10 lbs. / sq. ft., notwithstanding the values specified in the pressure table .

A maximum wind pressure of 40 lbs./sq. ft. perpendicular to the direction of the wind is considered for violent storms along the Indian coasts and about 20 lbs./sq. ft. in the planes. Therefore, a wind load of 30 lbs./sq. ft. for coastal areas, 15 lbs./sq. ft. for planes and  $22\frac{1}{2}$  lbs./sq. ft. for hills, normal to the roof surface, will be sufficient for most of the roofs; it is considered that an allowance for wind load of 30 lbs. will cover for the weight of workmen on roof and 2 ft. 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 maximum, 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 bars 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 of the steel may be increased by  $33\frac{1}{3}$  p.c. 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 25 lbs/sq. ft.

### Dead Loads

Weights of roofing materials are given in Section 9.

**Weight of steel roof trusses** for slopes of 1 in 2.

$$(i) W = \frac{1}{2} L \sqrt{wLS}$$

$$(ii) W = \frac{1}{2} SL \left( 1 + \frac{L}{10} \right)$$

$$(iii) W = \frac{P}{45} SL \left( 1 + \frac{L}{5\sqrt{S}} \right)$$

W = wt. of truss in lbs.,  
w = wt. of roof surface in lbs./sq. ft.,  
L = span in ft.,  
S = spacing of truss in ft.,  
P = capacity of truss in lbs./sq. ft. of horizontal projection of roof.

The following weights may be taken approximately for estimating purposes :—

Span of truss in ft.	25	30	40	50	60	80	
Approx. wt. in lbs./sq. ft. of horizontal projection (area)	2	2½	2½	3	4½	5	Spacing 10 ft.
	1½	1½	1½	2½	3	3½	Spacing 15 ft.

To the above weights add weights of bearing plates, anchor bolts, anchor plates and purlins, etc.



A saving in weight of about 10 to 13 per cent can be obtained by the use of welding instead of riveting but there may not be a corresponding saving in cost.

Weights due to purlins, rafters, reebers, etc., may be taken at 4 lbs./sq. ft. of covered area for light roofs and 5 lbs./sq. ft. for heavy roofs.

*Purlins* : Steel purlins will weigh from  $1\frac{1}{2}$  to 4 lbs./sq. ft. of area covered, depending upon the spacing and the capacity of the trusses. The *bracings* in the planes of the upper and lower chords will vary from  $\frac{1}{2}$  to 1 lb./sq. ft. of area.

## 2. GENERAL PRACTICAL CONSIDERATIONS FOR MAKING AND FIXING TRUSSES

(a) Principal rafters should not be longer than 10 ft. between struts.

(b) Purlins must be fixed at the joints (node points of the truss) otherwise bending will occur and the rafters will have to be designed as beams necessitating bigger sections for the rafters. But sometimes such an arrangement is unavoidable where roofing sheets of some particular size have to be fixed.

(c) For fixing spacings between purlins—The max. span for C. G. I. sheets supported at their ends should be 5 ft. for 22 gauge and  $4\frac{1}{2}$  ft. for 24 gauge sheetings. If adjoining sheets are lapped 9 inches and connected by a double row of rivets these spans may be increased to 6 ft. and  $5\frac{1}{2}$  ft. respectively, and which may be 8 ft. for 18 gauge and  $6\frac{1}{4}$  ft. for 20 gauge sheets.

(d) Nos. of Panels is generally taken = Span/5.

(e) M.S. bed plates of size  $1' \times 1' \times \frac{1}{2}"$  thick are fixed over the stone templates under the truss ends with shoe angles of size  $2\frac{1}{2}" \times 2\frac{1}{2}" \times \frac{1}{2}"$ . Two angles are on each end, fixed with the gusset plate.

(f) Lead sheet  $\frac{1}{8}"$  thick is provided under the plate on the free end. Slotted holes 2" long and of width  $1/16"$  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.

(g) Minimum Size of Truss Members :

Angle	..	..	$2'' \times 1\frac{1}{2}'' \times \frac{1}{4}''$	} Angles should be preferred to Tees
Flat	..	..	$2'' \times \frac{1}{4}''$	
Rod	..	..	$\frac{5}{8}''$ dia.	

(h) *Ties* : Should not have slenderness values of more than 250 (max. 350). Deduction is made for the rivet holes from the cross sectional area  $= t(d + \frac{1}{8}'')$  for each rivet hole. It is thickness of tie,  $d$  is diameter of rivet.

Angles should be preferred to flats or rounds for ties and other tension members to ensure stiffness and rigidity of framework and to minimize bending and distortion during handling and transport. For very 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 )  $\times$  the working stress (8 tons/sq.in.) equals the total stress carried ; and a strut being given such an area that the area of the strut  $\times$  the working stress obtained from buckling formula (Section 10) is equal to the total stress carried. Use tables given in Section 4.

(i) Proper Method of Fixing Purlins is shown in the illustration.

Proper Way of  
Fixing  
Purlins  
(they are  
stronger)



(j) *Rivets*. The rivets should be in the standard gauge lines of angles, as has been illustrated and tabulated in Section 4. The rivets are generally  $\frac{3}{4}''$  and  $\frac{5}{8}''$  dia., at 3'' centres, and all distances from centre of rivet to edge of plates or end of angles should not be less than  $1\frac{1}{2}''$  except when governed by standard gauge line of angles.

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.

(k) Thickness of walls for supporting steel trusses should be as follows :—

Spans 12 ft. to 24 ft. . .	18 ins.	(Also see under Timber Trusses).
Spans 25 ft. to 40 ft. . .	24 ins.	

(l) Prefer 18 gauge sheets for roofing as well as ridges, gutters and valleys, etc.

A *Queen-post* truss is rather unsuitable for steel work as it is "deficient" and the tie must be designed to carry the resulting bending moment.

*Transverse Strength of Corrugated Iron*

$$W = \frac{44 \cdot 6 \times t \times b \times d}{L}$$

where: W=breaking weight in tons (distributed), t=thickness of sheet in inches; b=breadth of sheet in inches; d=depth of corrugation in inches; L=unsupported length of sheet in inches.

Also see Table at page 4/30

**Proportions of Roof Inclinations**

Proportion of height to half span		Proportion of length of Rafter	
		to height	to half span
1/1	45°—0'	1.41	1.41
1/1½	33°—41'	1.80	1.20
1/√3	30°—0'	2.00	1.15
1/2	26°—34'	2.24	1.12
1/2½	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  $\frac{1}{4}$ th to  $\frac{1}{3}$ th 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 Structures" under Roof Slopes). Pitch of roof on West Coast is usually  $\frac{1}{4}$ .

**Spacing of Trusses.** The spacing of steel trusses is generally 10 ft. up to 50 ft. span and beyond that  $\frac{1}{3}$ th of the span. Spacing may be increased for light roofs with C.G.I. sheets up to 15 ft.



**Camber.** Where tie is to be cambered, it is  $1/30$ th to  $1/40$ th 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 30 ft., of same section as struts.

**Bearings.** Ends which are to be fixed in masonry should be properly anchored down with bolts of lengths varying from 2'-6" to 4'-0" and minimum dia. of  $\frac{5}{8}$ ". For open sheds, anchorage has to be more secure and may be doubled. Two anchor bolts are provided on each end for ordinary trusses.

Provision should be made for expansion and contraction of the trusses due to temperature changes. Up to about 25 ft. span both ends may be fixed but beyond this span, either one of the ends is free to move and the other is fixed or both the ends are made free according to the span and the temperature variations. For free ends, slotted holes are made in the truss sole plates and the ends are made free to slide. For big spans of say 80 ft. or above, ends may be built on rollers. Slotted holes should also be made in the joints of the purlins, say at about 50 ft. intervals and arrangements made for their free movement. 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.

Reactions at the ends of the trusses are vertical on free ends and inclined on fixed ends.

### 3. DESIGN OF STEEL ROOF TRUSSES BY STRESS CO-EFFICIENT TABLES

To find stress in any member:

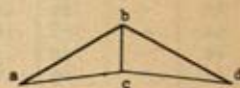
- (a) Multiply the total dead load carried by the truss including its own weight by the co-efficient for dead load.
- (b) Multiply the total wind load pressure (or the live load to be taken in lieu of the wind load) acting normal to its surface on one side of the roof by the co-efficient for wind pressure.

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.



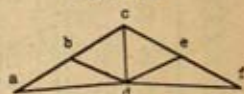
Member of Truss	Kind of Stress	Stress co-efficient			
		$R = \frac{S}{4} \quad C = \frac{S}{30}$		$R = \frac{S}{3} \quad C = \frac{S}{24}$	
		Dead load (vertical)	Wind load (normal to roof surface)	Dead load (vertical)	Wind load (normal to roof surface)
ab	+	.65	.70	.52	.58
bc	-	.08	.07	.07	.05
ac	-	.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

S=Span, R=rise, C=camber.



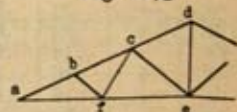
About 10 Ft.

Fig. 11/2



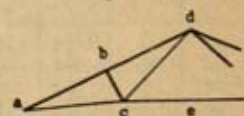
About 15 Ft.

Fig. 11/3



About 25 Ft.

Fig. 11/4

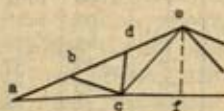


About 20 Ft.

Fig. 11/5

Member of Truss	Kind of Stress	Stress co-efficient			
		$R = \frac{S}{3}$	$\frac{S}{R} 30^\circ$	$R = \frac{S}{4}$	$R = \frac{S}{5}$
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	1.43
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

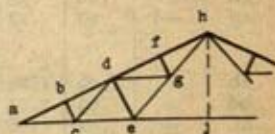
Fig. 11/5 truss without camber

Ditto.  
with camber = Rise/6

About 30 Ft.

without camber  
Fig. 11/6above truss  
with camber = Rise/6

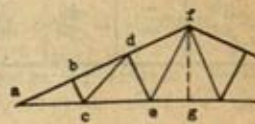
Member of Truss	Kind of Stress	Stress co-efficient			
		$R = \frac{S}{4}$ $C = \frac{S}{30}$		$R = \frac{S}{3}$ $C = \frac{S}{24}$	
		Dead load (vertical)	Wind load (normal to roof surface)	Dead load (vertical)	Wind load (normal to roof surface)
ab	+	1.13	1.34	.90	.93
bd	+	1.07	1.34	.83	.93
df	+	1.02	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



About 50 Ft.

Fig. 11/7

(For further details about this truss see next page)

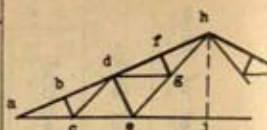


About 30 Ft.

Fig. 11/8



Member of Truss	Kind of Stress	Stress co-efficients				
		$R = \frac{S}{3}$	$R = 30^\circ$	$R = \frac{S}{4}$	$R = \frac{S}{5}$	$R = \frac{S}{6}$
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	.94	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	.23	.26
gh	-	.49	.57	.67	.86	.95
eg	-	.36	.42	.49	.63	.70



About 50 Ft.

Truss same as  
Fig. No. 11/7  
without camber

Ditto.  
with camber =  $R/6$

Member of Truss	Kind of Stress	Stress co-efficient		Length co-efficient
		Dead Load (vertical)	Wind load (normal to roof surface)	
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
fl	-	.50	.56	.188
ce	-	.17	.37	.156
eh	-	.17	.37	.156
hk	-	.42	.93	.156
fh	-	.25	.56	.156

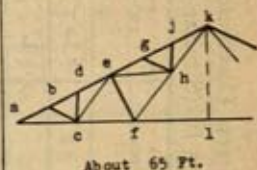


Fig. 11/9

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.

(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 members have been worked on the assumption that the roof purlins occur over the points of intersection of the various members with the rafter; where such is not the case, allowance should be made for the bending of the rafter and rafters designed as beams.

(The different tables might give slightly varying results but it will not make any appreciable difference in the final choice of a suitable section).

# Size of Members for Steel Trusses of Various Spans

For figures of trusses see Stress Co-efficient Tables

Span 16 ft. Clear—4 Panels at 4.75 ft. (Similar to Fig. No. 11/5.)

		10 ft. centres	12'-6" centres	15 ft. centres
Type A Roofing (load 22 lbs./sq. ft. of roof surface acting vertical)	ab	2 Ls 2" x 1 1/4" x 1/4"	2 Ls 2" x 1 1/4" x 1/4"	2 Ls 2" x 1 1/4" x 1/4"
	dc	1 L do	1 L do	1 L do
	bc	1 L do	1 L do	1 L do
	ce	1 L do	1 L do	1 L do
	ac	1 L do	1 L do	1 L do
Type B Roofing (load 31 lbs./sq. ft. of roof surface acting vertical)	ab	2 Ls 2" x 1 1/4" x 1/4"	2 Ls 2" x 1 1/4" x 1/4"	2 Ls 2" x 1 1/4" x 1/4"
	dc	1 L do	1 L do	1 L do
	bc	1 L do	1 L do	1 L do
	ce	1 L do	1 L do	1 L do
	ac	1 L do	1 L do	1 L 3" x 2" x 1/4"

Type A Roofing (load 22 lbs./sq. ft.  
of roof surface acting vertical)



Type B Roofing (load 31 lbs./sq. ft.  
of roof surface acting vertical)



Span 20 ft. Clear—4 Panels at 5.9 ft. (Similar to Fig. No. 11/5.)

	10 ft. centres	12'-6" centres	15 ft. centres
ab	2 Ls 2" x 1 1/2" x 1/4"	2 Ls 2" x 1 1/2" x 1/4"	2 Ls 2" x 1 1/2" x 1/4"
dc	1 L 2" x 1 1/2" x 1/4"	1 L 2" x 1 1/2" x 1/4"	1 L 2" x 2" x 1/4"
bc	1 L do	1 L do	1 L 2" x 1 1/2" x 1/4"
ce	1 L do	1 L do	1 L 2" x 2" x 1/4"
ac	1 L do	1 L do	1 L do
ab	2 Ls 2" x 1 1/2" x 1/4"	2 Ls 2" x 1 1/2" x 1/4"	2 Ls 2" x 2" x 1/4"
dc	1 L do	1 L do	1 L 2" x 1 1/2" x 1/4"
bc	1 L do	1 L do	1 L do
ce	1 L do	1 L do	1 L do
ac	1 L 2" x 2" x 1/4"	1 L 2 1/2" x 2" x 1/4"	1 L 2 1/2" x 2" x 5/16"

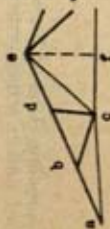
Type A roofing (load 22 lbs./sq. ft.  
of roof surface acting vertical)



Type B Roofing (load 31 lbs./sq. ft.  
of roof surface acting vertical)

Span 24 ft. Clear—6 Panels at 4.7 ft. (Similar to Fig. No. 11/6.)

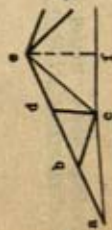
	10 ft. centres	12'-6" centres	15 ft. centres
ab	2 Ls 2" x 1 1/4" x 1/4"	2 Ls 2" x 1 1/4" x 1/4"	2 Ls 2" x 1 1/4" x 1/4"
ec	1 L do	1 L do	1 L do
dc	1 L do	1 L do	1 L do
bc	1 L do	1 L do	1 L do
cf	1 L do	1 L do	1 L do
ac	1 L do	1 L 2 1/2" x 2" x 1/4"	1 L 3" x 2" x 1/4"
ab	2 Ls 2" x 1 1/4" x 1/4"	2 Ls 2" x 1 1/4" x 1/4"	2 Ls 2" x 2" x 1/4"
ec	1 L do	1 L do	1 L do
dc	1 L do	1 L do	1 L do
bc	1 L do	1 L do	1 L do
cf	1 L do	1 L do	1 L do
ac	2 Ls do	2 Ls do	2 Ls 2" x 1 1/4" x 1/4"

Type A Roofing (load 22 lbs./sq. ft.  
of roof surface acting vertical)Type B Roofing (load 31 lbs./sq. ft.  
of roof surface acting vertical)

Span 30 ft. Clear—6 Panels at 5.8 ft. (Similar to Fig. No. 11/6.)

	10 ft. centres	12'-6" centres	15 ft. centres
ab	2 Ls 2" x 1 1/2" x 1/2"	2 Ls 2" x 2" x 1/2"	2 Ls 2 1/2" x 2" x 1/2"
cc	1 L do	1 L 2" x 1 1/2" x 1/2"	1 L 2" x 1 1/2" x 1/2"
dc	1 L do	1 L do	1 L do
bc	1 L do	1 L do	1 L do
cf	1 L do	1 L do	1 L dy
ac	1 L 2 1/2" x 2" x 1/2"	1 L 3" x 2 1/2" x 1/2"	2 Ls 2" x 1 1/2" x 1/2"
ab	2 Ls 2 1/2" x 2" x 1/2"	2 Ls 2 1/2" x 2" x 1/2"	2 Ls 3" x 2" x 1/2"
cc	1 L 2" x 1 1/2" x 1/2"	1 L do	1 L do
dc	1 L do	1 L do	1 L dy
bc	1 L do	1 L do	1 L do
cf	1 L do	1 L do	1 L do
ac	2 Ls 2" x 1 1/2" x 1/2"	2 Ls 2" x 1 1/2" x 1/2"	2 Ls 2" x 1 1/2" x 1/2"

Type A Roofing (load 22 lbs./sq. ft.  
roof surface acting vertical)



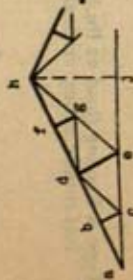
Type B Roofing (load 31 lbs./sq. ft.  
of roof surface acting vertical)



Span 40 ft. Clear — 8 Panels at 5'75 ft. (Similar to Fig. No. 11/7.)

	10 ft. centres	12'-6" centres	15 ft. centres
ab	2 Ls 2" x 1 1/2" x 1/4"	2 Ls 2 1/2" x 2" x 1/4"	2 Ls 3" x 2" x 1/4"
bc	1 L do	1 L 2" x 2" x 1/4"	1 L 2 1/2" x 2" x 1/4"
cd	2 Ls do	2 Ls 2" x 1 1/2" x 1/4"	2 Ls 2" x 1 1/2" x 1/4"
bd	1 L do	1 L do	1 L do
ej	1 L do	1 L 2" x 2" x 1/4"	1 L 2 1/2" x 2" x 1/4"
ac	2 Ls do	2 Ls 2" x 1 1/2" x 1/4"	2 Ls 2" x 1 1/2" x 1/4"
dg	1 L do	1 L do	1 L do
ab	2 Ls 3" x 2" x 1/4"	2 Ls 3" x 2" x 1/4"	2 Ls 3 1/2" x 2 1/2" x 1/4"
bc	1 L do	1 L 3 1/2" x 2 1/2" x 1/4"	2 Ls 2" x 1 1/2" x 1/4"
cd	2 Ls 2" x 1 1/2" x 1/4"	2 Ls 2" x 1 1/2" x 1/4"	2 Ls 2" x 1 1/2" x 1/4"
bd	1 L do	1 L do	1 L do
ej	1 L 2 1/2" x 2" x 1/4"	1 L 3" x 2" x 1/4"	1 L 3 1/2" x 2 1/2" x 1/4"
ac	2 Ls 2" x 1 1/2" x 1/4"	2 Ls 2" x 2" x 1/4"	2 Ls 3" x 2" x 1/4"
dg	1 L do	1 L 2" x 1 1/2" x 1/4"	1 L 2" x 1 1/2" x 1/4"

Type A Roofing (load 22 lbs./sq. ft. of roof surface acting vertical)



Type B Roofing (load 31 lbs./sq. ft. of roof surface acting vertical)

Span 50 ft. Clear—10 Panels at 6.25 ft. (Similar to Fig. No. 11/7.)

	10 ft. centres	12'-6" centres	15 ft. centres
ab	2 Ls 2½" × 2" × ½"	2 Ls 3" × 2" × ½"	2 Ls 3½" × 2½" × ½"
he	1 L 3" × 2" × ½"	1 L 3" × 2" × ½"	1 L do
ed	2 Ls 2" × 1½" × ½"	2 Ls 2" × 1½" × ½"	2 Ls 2" × 1½" × ½"
bc	1 L do	1 L do	1 L do
ej	1 L 3" × 2" × ½"	1 L 3" × 2" × ½"	1 L 3½" × 2½" × ½"
ac	2 Ls 2" × 1½" × ½"	2 Ls 2" × 1½" × ½"	2 Ls 2" × 2" × ½"
dg	1 L do	1 L do	1 L 2" × 1½" × ½"
ab	2 Ls 3" × 2½" × ½"	2 Ls 3½" × 2½" × ½"	2 Ls 3½" × 2½" × ½"
he	2 Ls 2" × 1½" × ½"	2 Ls 2" × 1½" × ½"	2 Ls 2" × 2" × ½"
ed	2 Ls 2" × 1½" × ½"	2 Ls do	2 Ls 2" × 1½" × ½"
bc	1 L do	1 L do	1 L do
ej	2 Ls do	2 Ls do	2 Ls do
ac	2 Ls 2½" × 2" × ½"	2 Ls 3" × 2" × ½"	2 Ls 3½" × 2½" × ½"
dg	1 L 2" × 1½" × ½"	1 L 2" × 1½" × ½"	1 L 2" × 1½" × ½"

Type A Roofing (load 22 lbs./sq. ft.  
of roof surface acting vertical)

Type B Roofing (load 31 lbs./sq. ft.  
of surface acting vertical)

Truss members for various spans and spacings given in the tables have been worked out with the following assumptions based on a working stress of 8 tons/sq. in.

Type A: Roof covered with C.G.I. sheets and steel purlins, at 7 lbs./sq. ft. of ground area. Wind load at 30 lbs./sq. ft. or  $22\frac{1}{2}$  lbs. normal to the roof surface.

Total load for design is taken at 22 lbs./sq. ft. normal for roof coverings, vertical for trusses. (Total weights divided by 1.33).

Type B: Roof covered with single tiles and steel purlins with ceiling, at 18 lbs.+6 lbs. Wind load same as for Type A above.

Total load for design is taken at 31 lbs./sq. ft.

Roof slopes	..	..	1 in 2
Camber	..	..	span/28

Panels equal and equally loaded; spans centre to centre of bearings will be about 1 ft. more.

Gusset plates at the shoes, ridge and junctions of heel and tie and upper tie will be  $\frac{3}{8}$ " thick and for other joints  $\frac{1}{4}$ " thick. For big spans say 50 ft., thickness of the gusset will be increased to  $\frac{1}{2}$ ". All gussets are single. Rivets  $\frac{3}{8}$ " dia.

The following variations will require no alterations in truss sections:

- (a) Variations of  $\pm 10$  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 after 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.



#### 4. WORKSHOP AND FACTORY ROOFS

Where it is desirable to provide north light without direct sunlight a "saw-toothed" roof is suitable. 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 less 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 40 ft. For larger spans a truss comprising lattice girders is used.

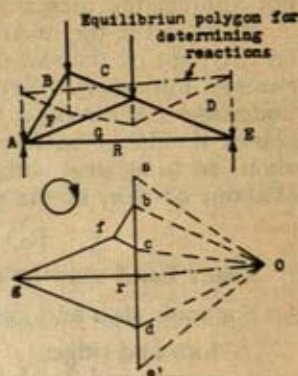
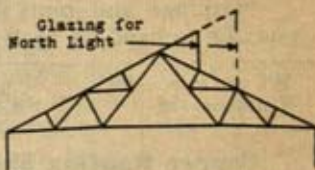


Fig. 11/10



ing 40 ft. For larger spans a composite form of construction comprising lattice girders and north light trusses is used.

Fig. 11/10 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.

## 5. ROOF FIXTURES

**Sheet Lead** may be of the following thicknesses:—

For roofs, flats and gutters 7 to 8 lbs/superficial ft.

„ hips and ridges	6 to 7	„	„
„ aprons and flashings	5	„	„
„ bottoms of cisterns	7	„	„
„ sides of cisterns	6	„	„

Flashings and joints should have a lap of 4". Lead must be so fixed on roofs as to allow free expansion.

Wt. per sq. ft.—lbs.	5	6	7	8	10
Thickness—in.	.084	.101	.118	.135	.169

**Copper Roofing Sheets** generally used for flat roofs are of gauge :

Gauge	22	23	24
Weight in lbs./sq. ft.	1.302	1.116	1.023

Iron nails should not be in contact with copper as it will corrode both the metals.

Copper roofing sheets 24 gauge weigh 1.33 lbs./sq. ft. as laid.

**Eaves Gutters.** May be made of galv. iron sheet 18 to 22 gauge, bent semi-cylindrical with edges rounded and twice the diameter of the down pipe. Gutters can also be made of milled lead sheets of weight 6 lbs./sq. ft or of zinc sheetings (for important buildings), or of cast iron. Zinc gutters, of which the most widely used are ogee and half-round types, require no painting. They are not so strong as those of steel but satisfactory service can be had if the thickness of metal is not less than No. 14 zinc gauge. Weights of zinc sheets are given in Section 4.



Gutters and flashings should be so arranged as to give free play for expansion or contraction in any direction. Slope of gutters should be about 1 in 100 in straight lengths and steeper for portions which are not straight.

Not less than one wrought iron bracket should be fixed to each 6 ft. length, and one bracket to each angle, nozzle piece, drop end, etc. Where cast iron gutters are fixed, fix one bracket to each length. Gutter lengths should overlap 6" and rounded edges should be fixed with 3" dowel pins. Bracket size:  $1" \times \frac{1}{4}"$ .

**Valleys.** May be of lead, zinc or galvanized iron sheeting. Should not be less than 6" wide at the bottom and turned up under the roofing 15" on each side. Same size of sheets may be used as for eaves gutters but prefer 18 gauge. Overlapping is 9".

**Ridges and Hips.** Are made from 20 gauge G.I. sheets, with 9" lap on either side and at each joint between two lengths.

**Flashings.** May be of milled lead as specified above or of 18 B.W.G. zinc or galvanized iron (lead is specified for important buildings). Where lead is used, copper nails will be employed to fasten it to the woodwork. Only one edge of flashing is fixed and the other end is left free for expansion. Against a wall, the flashing will be slightly titled and turned up 6 inches, the upper edge being left free and covered by an apron overlapping it by about 4 inches, the upper edge of which should be tucked 2 inches deep into a joint of the masonry and filled with mastic or cement mortar.

**Rain-water Pipes.** If of G.I. sheeting, may be of 18, 20 or 22 gauge. If of zinc, these should be of No. 12 zinc gauge (23 I.S.W.G.). Usual size is 3 to 6 inches dia. Brackets are fixed about 6 ft. apart. Joints between successive lengths of pipes will be by collars at least 4 inches deep; the collars should be first riveted together and then shrunk on.

The bore of the rain water pipes should be 1 sq. in. for each 50 sq. ft. of roof area drained and may be provided about 20 ft. apart.



### Definitions of Roofing Terms

*Pent-roof*—A roof with slope on one side only.

*Flashings*—Pieces of lead or copper sheets let into the joints of a wall or worked in the roof slates or tiles around dormers, chimneys, etc., to lap over gutters to prevent leakage.

*Verge*—The edge of the tiling or slates projecting over the gable of a roof, that on the horizontal portion being called *eaves*.

*Verge Board*—Often corrupted into Barg Board; the board under the verge of gables.

*Weather Boarding*—Boards 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.

## 6. CURVED ROOFS

Curved roofs can be made up to 30 ft. span with 20 ft. radius of 18 S.W.G. corrugated iron without trusses with :—

Tie rods— $2'' \times \frac{3}{8}''$  or  $\frac{3}{4}''$  dia., 6'-4" apart.

King rods— $\frac{3}{8}''$  dia.

Eaves stiffened by L irons  $2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{1}{4}''$  and top strengthened by an L iron  $1\frac{1}{2}'' \times 1\frac{1}{2}'' \times \frac{1}{2}''$  running along the under surface of corrugated iron on each side of the king-rods.

## 7. FLAT ROOFS

For **Superimposed Loads** on flat roofs see Section 6 and page 11/2.

### Ferro Concrete Battens for Flat Roofs

Clear span	6'	7'	8'	9'	10'
Full section of batten	$2\frac{1}{2}'' \times 5\frac{1}{2}''$	$2\frac{1}{2}'' \times 6''$	$2\frac{1}{2}'' \times 6\frac{1}{2}''$	$3'' \times 7\frac{1}{2}''$	$3\frac{1}{2}'' \times 7\frac{1}{2}''$
Tensile reinforcement rod of diameter	$\frac{3}{8}''$	$\frac{3}{8}''$	$7/16''$	$\frac{1}{2}''$	$\frac{1}{2}''$

The above table is based on a roof load of 100 lbs./sq. ft. plus weight of the batten and is worked out with stresses: 18000 and 750 lbs./sq. in. Battens are 12" centre to centre. One  $\frac{1}{4}$ " dia. bar is provided on the compression side of every batten as shown in the illustration. This provides for common roofs. These battens require special care while fixing on roofs to see that the tensile reinforcement side is placed downside.



### Safe Loads on Nail Planking

Span	Safe superimposed load in lbs./sq. ft.					Apply to good class ordinary timber.
ft.	1"	1½"	2"	2½"	3"	
1½	420	..				For loads on other kinds of timber multiply by the following co-efficients:—
2	180	600				
2½	90	300	730	..		
	50	180	420	825		
3½	..	110	260	520	900	
4		75	180	350	600	
4½		50	125	230	420	Deodar 1.33
5		..	90	180	300	Sal 1.83
5½			70	130	230	Chir 1.42
6			50	100	180	

### Diameter and Spacing of Ballies for 100 lbs./sq. ft. Roof Load

Span ft.	Deodar Ballies		Sal Ballies	
	Spacing 1 ft. c/c	Spacing 1½ ft. c/c	Spacing 1 ft. c/c	Spacing 1½ ft. c/c
4	3" dia.	3½" dia.	2½" dia.	3" dia.
5	3½" "	4" "	3½" "	3½" "
6	4" "	4½" "	3½" "	4" "
7	4½" "	5" "	4½" "	4½" "
8	4½" "	5½" "	4½" "	5" "
9	5½" "	6" "	5" "	5½" "
10	5½" "	6½" "	5½" "	5½" "
11	6½" "	6½" "	6" "	6½" "
12	6½" "	7½" "	6½" "	7" "

## JACK ARCH ROOFING

Size of Rolled Steel Beams and old Rails for Different Spans :—

Size of Beam	Max. clear span up to which the beam may be used with a spacing of:				
	4'-0"	4'-6"	5'-0"	5'-6"	6'-0"
5" x 2½"	8'-4"	7'-10"	7'-5"	7'-0"	6'-8"
5" x 3"	9'-4"	8'-10"	8'-4"	7'-11"	7'-7"
6" x 3"	10'-9"	10'-1"	9'-7"	9'-1"	8'-8"
7" x 3½"	13'-1"	12'-4"	11'-8"	11'-1"	10'-7"
7" x 4"	13'-7"	13'-0"	12'-3"	11'-8"	11'-2"
8" x 4"	15'-4"	14'-5"	13'-8"	13'-0"	12'-6"
9" x 4"	17'-5"	16'-6"	15'-8"	14'-11"	14'-3"
10" x 4½"	19'-10"	19'-0"	18'-1"	17'-3"	16'-6"
10" x 5"	21'-0"	20'-3"	19'-6"	18'-10"	18'-1"
12" x 5"	24'-2"	22'-9"	21'-7"	20'-7"	19'-8"
12" x 5"	24'-3"	23'-4"	22'-4"	21'-3"	20'-5"
1-75 lbs. Rail	10'-0"	9'-9"	9'-6"	9'-0"	8'-6"

The above spans are calculated for a dead load of 130 lbs./sq. ft. and live load of 25 lbs./sq. ft. Max. deflection =  $1/925$  of effective span.

Area of concrete filling below the crown of jack arches, for estimating, is taken = span × depth from top of crown to top of springing ×  $\frac{1}{4}$ .

Brick-arches and tie rods have been described under Arches in the Section on "Masonry Structures". For Jack arch roofs, R.C.C. pre-cast beams can also be used instead of R.S.Js. Such beams can be made of a shape somewhat like the top of a pillar of an arched bridge for the ends of the arches to rest on the sides of the beam.



### Sections of Wooden Battens for Flat Roofs

Spacing 12 inches centre to centre

Span ft.	100 lbs. sq. ft.			70 lbs. sq. ft.	
	Sal	Deodar or Chil	Kail	Sal	Kail
	in.	in.	in.	in.	in.
3	$1\frac{1}{2} \times 2\frac{1}{2}$	$1\frac{1}{2} \times 2\frac{1}{2}$	$1\frac{1}{2} \times 2\frac{1}{2}$	$1\frac{1}{2} \times 2$	$1\frac{1}{2} \times 2\frac{1}{2}$
3½	$1\frac{1}{2} \times 2\frac{1}{2}$	$1\frac{1}{2} \times 2\frac{1}{2}$	$1\frac{1}{2} \times 2\frac{1}{2}$	$1\frac{1}{2} \times 2$	$1\frac{1}{2} \times 2\frac{1}{2}$
4	$2 \times 2\frac{1}{2}$	$1\frac{1}{2} \times 3$	$1\frac{1}{2} \times 3\frac{1}{2}$	$1\frac{1}{2} \times 2\frac{1}{2}$	$1\frac{1}{2} \times 3$
4½	$2 \times 3$	$1\frac{1}{2} \times 3\frac{1}{2}$	$2 \times 3\frac{1}{2}$	$1\frac{1}{2} \times 3$	$1\frac{1}{2} \times 3\frac{1}{2}$
5	$2 \times 3\frac{1}{2}$	$2 \times 3\frac{1}{2}$	$2 \times 3\frac{1}{2}$	$1\frac{1}{2} \times 3$	$2 \times 3\frac{1}{2}$
5½	$2 \times 3\frac{1}{2}$	$2 \times 3\frac{1}{2}$	$2\frac{1}{2} \times 4\frac{1}{2}$	$2 \times 3\frac{1}{2}$	$2 \times 3\frac{1}{2}$
6	$2 \times 4$	$2\frac{1}{2} \times 4$	$2\frac{1}{2} \times 4\frac{1}{2}$	$2 \times 3\frac{1}{2}$	$2\frac{1}{2} \times 4$
6½	$2\frac{1}{2} \times 4$	$2\frac{1}{2} \times 4\frac{1}{2}$	$2\frac{1}{2} \times 4\frac{1}{2}$	$2\frac{1}{2} \times 3\frac{1}{2}$	$2\frac{1}{2} \times 4\frac{1}{2}$
7	$2\frac{1}{2} \times 4$	$2\frac{1}{2} \times 4\frac{1}{2}$	$2\frac{1}{2} \times 4\frac{1}{2}$	$2\frac{1}{2} \times 4$	$2\frac{1}{2} \times 4\frac{1}{2}$
7½	$2\frac{1}{2} \times 4\frac{1}{2}$	$2\frac{1}{2} \times 4\frac{1}{2}$	$2\frac{1}{2} \times 5$	$2\frac{1}{2} \times 4\frac{1}{2}$	$2\frac{1}{2} \times 4\frac{1}{2}$
8	$2\frac{3}{4} \times 4\frac{1}{2}$	$2\frac{3}{4} \times 5$	$3 \times 5\frac{1}{2}$	$2\frac{1}{2} \times 4\frac{1}{2}$	$2\frac{3}{4} \times 5$
8½	$2\frac{3}{4} \times 5$	$2\frac{3}{4} \times 5\frac{1}{2}$	$3 \times 5\frac{1}{2}$	$2\frac{1}{2} \times 4\frac{1}{2}$	$2\frac{3}{4} \times 5\frac{1}{2}$
9	$2\frac{3}{4} \times 5$	$2\frac{3}{4} \times 5\frac{1}{2}$	$3\frac{1}{2} \times 5\frac{1}{2}$	$2\frac{1}{2} \times 5$	$2\frac{3}{4} \times 5\frac{1}{2}$
9½	$2\frac{3}{4} \times 5\frac{1}{2}$	$3 \times 5\frac{1}{2}$	$3\frac{1}{2} \times 6$	$2\frac{1}{2} \times 5\frac{1}{2}$	$3 \times 5\frac{1}{2}$
10	$3 \times 5\frac{1}{2}$	$3 \times 6$	$3\frac{1}{2} \times 6\frac{1}{2}$	$2\frac{1}{2} \times 5\frac{1}{2}$	$3 \times 6$

### 8. SPECIFICATIONS FOR DIFFERENT TYPES OF ROOFS

**First Class Mud Roofing** consists of:—

Two layers of tiles,  $12'' \times 6'' \times 1\frac{1}{4}''$  each on battens, covered with 1'' layer of mud plaster, 4'' layer of earth and another layer of 1'' mud plaster on top and finished with cowdung plaster.

The tiles are laid in lime mortar. The second layer of tiles is laid over  $\frac{1}{2}''$  bed of lime mortar over the first layer and breaking joints in both directions with the layer underneath. Battens are spaced 12'' centre to centre. The necessary slope in the roofs should be formed by sloping the beams or the battens.

### **Second Class Mud Roofing**

This includes the same items of works as the First Class except that only one layer of tiles  $12'' \times 6'' \times 2''$  is laid instead of two layers of  $12'' \times 6'' \times 1\frac{1}{4}''$ .

Where tiles are laid over top of flat roofs, they should be pointed with cement mortar or grouted with cement slurry.

*Weights in lbs. per sq. ft. of Dead Load*

First class mud roofing with		
$\frac{1}{2}$ " boarding and single tiles	70	Add for superimposed load as detailed at page 11/2.
—ditto— with a double layer of $1\frac{1}{4}$ " tiles . . . . .	80	
Second class mud roofing with a single layer of 2" tiles.	70	

**Tarraced Roofing.** Consists of a layer of flat tiles set in mortar over 3" of fine concrete, which is placed on two layers of flat tiles also set in mortar, and supported on wooden battens placed one foot apart. Where tiles are laid in double layers, they should be of 1" thick size, upper layer to be embedded in mortar on the lower layer breaking joints. The tiles should be soaked in thick whitewash before use.

Lime concrete with  $\frac{3}{4}$ " brick aggregate is generally used for such roofs. For water-proofing,  $\frac{3}{4}$  lb. of bar soap and 1 lb. of alum to each cubic foot of concrete are dissolved in the water used for mixing the mortar. While the beating of the concrete is going on, the surface must be liberally sprinkled with fresh lime wash. A small proportion of 'gur' added to the lime water will improve the water-proofing qualities of the roof.

Where the tiles are supported on steel  $\perp$ s, the tees may be of size:  $2" \times 2" \times \frac{1}{4}"$ , laid 12" apart on a span of 4 feet, between joists.

### **Painting Terraced Roofs with Bitumen**

After laying the tiles the works should be kept wet for the mortar to set. After the mortar has set the surface should be allowed to dry before applying the bitumen. The bitumen is heated to about 400 degrees F. (or according to the specifications of the manufacturers), poured on the surface to be treated and pulled out so that the average thickness is  $1/16$  inch. This can be obtained by evenly spreading 34 lbs. of bitumen over 100 sq. ft. to be covered. The coat of bitumen should be continued



along the parapet walls up to the drip course. Mud filling or plastering will be done in the customary manner when the bitumen has cooled down and is no longer tacky.

The "grade" of bitumen used should be with a high melting point say of 20 to 30 penetration. In high rainfall regions, two coats may be given. The bitumen coat should be blended with sand.

### **Water proofing 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	} Or	1 part red lead	} mixed	
4 parts ground lime		5 parts whiting		with
5 parts sharp sand		10 parts sharp sand		boiled oil

#### *R.C. Roof Slabs:*

For water-proofing, 2 coats of hot maxphalte or coal tar blinded with sand are given over the slab after the concrete has been cured and dried. A 3" thick layer of good compacted earth is laid over the bitumen treatment and then tiles or mud plaster, etc., over it. Each coat of asphalt should be blinded with sand at the rate of 1 c.ft. per 100 sq. ft. of surface.

### **Repairs to Porous Roofs**

$\frac{1}{4}$  lb. bar soap,  $\frac{1}{4}$  lb. alum, each dissolved separately in one gallon of boiling water, laid on the roof alternatively with whitewash, 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.

Apply a coat of "Colas"—(Bituminous compound—see under "Roads") about 40 lbs. per 100 sq. ft. of surface and blind with coarse sand, sweep off extra sand. This will last for about 3 years.

**Roof Slopes and Drainage.** 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 R.C. or R.B. slabs duly water-proofed or with a layer of bitumen or tar with 3" 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  $1\frac{1}{2}$ " should be taken up by the height of masonry walls.

For drainage, a 3" down pipe will serve a roof area of about 400 sq. ft., and a 4" down pipe about 700 sq. ft. in dry regions and about 450 sq. ft. in heavy rainfall regions. Where no down-pipe is fixed, the outlet pipe or channel should project 1 foot outside the wall.

### Slate Roofs

The pitch of the roof is kept according to the size of the slates as follows:

24"  $\times$  12" ..... 22°

20"  $\times$  10" ..... 27°

16"  $\times$  8" ..... 33°

The lap should not be less than 3". Only slating nails of copper or of non-rusting compositions should be used. The battens should not be less than  $1\frac{1}{2}$ "  $\times$   $1\frac{1}{2}$ ".

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 the apex formed by the edge of the slates a roll not less than 3" 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, 2" wide and 15" long will be nailed across the ridge at 2 ft. centres. A 3" 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 6" 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  $1\frac{1}{2}'' \times 1''$  nailed  $9\frac{1}{4}''$  apart centre to centre, the rafters being spaced 2 ft. apart centre to centre. The slates are of three sizes, the main slate is  $15\frac{3}{4}'' \times 15\frac{3}{4}''$  out of which any size can be cut. Overlap is about  $2\frac{3}{4}''$ . 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 copper 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 82 slates cover 100 sq. ft. of area. These slates are generally made in three colours—grey, black and red. Eternit slate roof covering is not affected by weather, is fire resisting, light and cool.

### Allahabad Tiling

This is double tiling or single tiling. The lower flat tiles are laid on battens. The lowest eaves battens should be deeper than the battens above so that the line of the tiles from ridge to the eaves is continuous. The battens are placed at 12'' intervals; size is  $1'' \times 1\frac{1}{4}''$ .

The three lowest tiles in each course of each layer as well as all ridge and hip tiles should be set in mortar. Unless special eaves tiles with closed ends are used the ends of each row of semi-hexagonal and semi-circular tiles should be stopped with mortar. In the case of double tiling, the space between the two rows of flat tiles should also be so filled. Tiles in contact with mortar must be soaked in water mixed with cowdung for at least 6 hours before laying.

A dry tile should not absorb more than  $\frac{1}{8}$ th of its weight of water when immersed for one hour.

The roof should not be pitched at a slope less than 1 in 3 or more than 1 in 2.

#### *Eaves and Barge Boards:*

The barge board should be about 2'' to  $2\frac{1}{2}''$  higher than the eaves board for a single layer and about 5'' to 6'' higher for a double layer of tiles so as to cover the end row of tiling.



### Mangalore Tiling

The roof should not be pitched at a slope of less than 1 in 3 or more than 1 in 2. Each tile of size about,  $16'' \times 9\frac{1}{2}''$  covers  $12\frac{1}{2}'' \times 8\frac{1}{4}''$ , and allowing for laps, 150 tiles are required for each 100 sq. ft. of roofing. 1000 tiles weigh about  $2\frac{1}{2}$  tons. (Sizes of tiles differ; exact size should be fixed locally).

Each flat tile weighs about  $5\frac{1}{8}$  lbs. when dry. Each ridge tile is about 16'' in length and weighs when dry  $7\frac{1}{2}$  lbs.

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,  $2'' \times 1''$  or  $1\frac{1}{4}'' \times 1''$  fixed  $12\frac{1}{2}''$  centre to centre to the upper surface of the rafters, or when plank-ing is used, battens are  $1'' \times 1''$  and exactly parallel to the eaves. The lowest batten, that nearest to the eaves, should be fixed about 10'' 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 not less than 2 cwts. applied at centre of span when supported on battens at  $13\frac{1}{2}''$  centres.

### Corrugated Iron Roofing

Ordinarily 22 gauge for corrugated and 24 gauge for plain sheets are used for roofs. These roofs are not generally laid at a slope flatter than 1 in 4. Laps should be 6'' length-wise and 2 corrugations side laps. In ridge and hips where plain sheets are used, laps will be 9''.



Sheets should be fastened down just above the eaves by continuous lengths of  $1\frac{1}{2}'' \times \frac{1}{4}''$  flat irons (wind ties), bolted down every 4 ft. by  $\frac{1}{2}''$  bolts, built  $1\frac{1}{2}$  ft. into the walls and secured at the lower ends by 6-in. square washers. Slot holes are cut in the wind ties to allow of expansion and contraction due to temperature changes.

Holes in the C.G.I. sheets should preferably be made on the ground; the sheets should be placed on trestles and the holes punched in the ridges (prefer drilling) of the corrugations from the outside inwards so that a proper seating for the limpet washer is obtained about 9" apart on the sides and at every second corrugation at the ends, care being taken that all holes should occur in the ridge of the sheet as laid. When four sheets overlap at corners, the holes should be drilled and not punched. Each sheet must be fastened at the corners, at every third corrugation along the ends, and evenly at the sides at intervals not exceeding 2 ft. Galvanized bolts, screws and washers should be used. Sheets should be secured to wood framing by means of galv. cone headed screws, drive screws, or jagged nails,  $2\frac{1}{2}''$  to 3" long, at intervals not exceeding 1 ft. on every bearer. Sheets are secured to steel framing by means of  $5/16''$  galv. hook-bolts at intervals not exceeding 1 ft. on every bearer. Length of the hook-bolt will be in accordance with the size of the steel section forming the bearers. The sheets should be riveted together with galv. iron rivets and washers or bolted together with  $\frac{1}{4}''$  dia. galv. bolts. In each case a "limpet" dome washer should be used at the top. All rivets, bolts, screws, etc., should be set in white lead.

For permanent work, before laying the sheets on the roof, they may be riveted on the ground in sets. Generally three sheets in length or three or four in breadth can be completed on the ground before they are hoisted on to the roof. For fixing, the rivets are passed through from below and a lead washer put on it and head made. Rivets are of galv. iron.

For temporary structures where it is undesirable to make holes in the C.G.I. sheets, Windle's Clips can be made use of.

Spans which C.G.I. sheets can support are given in the table at page 4/30.

Projection of eaves is generally made from 1.25 ft. to 2 ft. (horizontal).

Ridges, Hips, Valleys, etc., have been described under "Roof Fixtures" at page 11/24.

For **Nanital Pattern Roofing** the roof coverings are made from 24 gauge, and rolls and ridges from 22 gauge galv. iron plain sheeting.

Deodar wood should not be used in contact with galvanized sheets as it has destructive effect on zinc. Galvanizing also quickly disappears near sea and chemical works.

For painting galv. iron roofs, see under "Paints and Painting." G.I. roofs are found 10 to 20 degrees F. cooler when painted white than when left of their natural colour.

**Asbestos Cement Corrugated Sheets.** These sheets are laid in the same manner as C.G.I. sheets. The weight of laid roofing per 100 sq. ft. varies from 325 to 350 lbs., while C.G.I. roofing (20 gauge) weighs about 265 lbs., per 100 sq. ft.

Pitch of corrugations	Width of sheets ins	End lap ins.	Side lap	Wt. of roofing 100 sq. ft.	Spacing of purlins
2½	30	6	1½ corr.	324	3'-0"
3½	41½	6	½ corr.	350	4'-6"

Sheets are available in 6 to 10 ft. lengths, varying by 6 inches. Weight is about 3.5 lbs./sq. ft.

The roofing sheets should be capable of withstanding a distributed test load of 80 lbs. per sq. ft. with purlins up to 5'-6" centres. A.C. sheets are secured with 5/16" dia. galv. hook-bolts, crank bolts, or 4½" galv. cone headed roofing screws or drive screws, fitted with bituminous washers under the G.I. washers (lead Limpet washers are not used). Holes for fixing sheets should be drilled 7/16" dia. (½" larger than screws), and never punched. To



avoid fracture, sheets must not be rigidly fixed and screws must not be too tight. A hammer must never be used; Asbestos 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 about every 150 ft. length of the roof. Roof ladders or duck-boards should invariably be used so that no damage is done to the sheets when the workmen walk over it.

Asbestos cement roof cannot take excessive stresses, therefore, care should be taken while fixing purlins not to exceed 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.

Where it is proposed to use Asbestos sheets, particulars and specifications should be obtained from the manufacturing firms and work designed and executed accordingly.

### **Insulation Against Hot Weather of Corrugated Iron or Asbestos Roofs**

(Extracts from notes published by Dr. S.P. Raju in the Institution of Engineers (India) Journal Vol. 35—No. I, Sept. 1954.)

Put on the top of the sheets a layer of about 3 to 4 inches of paddy husk mixed in lime mortar (1 lime,  $2\frac{1}{2}$  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 tamper gently with a trowel, leaving about 3 inches of the roof sheets from the edge free. Finish with about  $\frac{3}{8}$ " to  $\frac{1}{2}$ " lime plaster. Where paddy husk is not available, groundnut husk or any other suitable vegetable husk may be used.



### **Fixing Mangalore or Country Tiles Over C.G.I. Sheet Roofing**

Pitch of the roof should be about 2 : 9

Sloping wooden battens  $1\frac{1}{2}'' \times 1\frac{1}{2}''$  are laid over the ridges of the corrugations 18'' to 21'' 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 3-in. screws. The horizontal battens  $2'' \times \frac{1}{2}''$  for country tiles and  $1'' \times \frac{3}{4}''$  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  $1\frac{1}{2}'' \times \frac{1}{8}''$  secured by  $\frac{3}{8}''$  bolts to the corrugated sheets below.

Ridges and hips can be of the tiles laid in mortar or of G.I. sheets or zinc sheets laid as detailed before. Battens may not be used in the case of country round tiles.

### **Thatched Roofing**

The bamboo framing consists of vertical bamboos  $1\frac{1}{2}''$  in diameter and 1 ft. apart to which split bamboos are securely fastened at right angles, 6'' apart. Thatch is from 6'' to 12'' thick and laid in 3'' layers. The first layer is generally attached to the bamboo framing before it is placed on the roof. The pitch of the roof is usually about 35 degrees.

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## 1. BRICKS

Standard sizes for 9-inch bricks as prescribed by various Public Works Departments in India :—

$9\frac{1}{2}" \times 4\frac{3}{4}" \times 2\frac{1}{4}"$	$9" \times 4\frac{1}{2}" \times 2\frac{1}{4}"$
$9" \times 4\frac{3}{8}" \times 2\frac{1}{4}"$	$8\frac{7}{8}" \times 4\frac{1}{2}" \times 2\frac{1}{4}"$
$9" \times 4\frac{1}{4}" \times 2\frac{1}{4}"$	$8\frac{3}{4}" \times 4\frac{1}{2}" \times 2\frac{1}{4}"$
$9" \times 4\frac{1}{4}" \times 2\frac{1}{2}"$	$8\frac{7}{8}" \times 4\frac{1}{2}" \times 3"$

Standard sizes for 10-inch bricks :

$10" \times 4\frac{7}{8}" \times 3"$	$9\frac{1}{2}" \times 4\frac{3}{4}" \times 2\frac{3}{4}"$
$10" \times 5" \times 3"$	$9\frac{1}{2}" \times 4\frac{1}{2}" \times 2\frac{3}{4}"$
$10" \times 5" \times 3\frac{1}{4}"$	$9\frac{1}{2}" \times 4\frac{3}{4}" \times 2\frac{7}{8}"$

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— $8" \times 3\frac{1}{2}" \times 2\frac{1}{4}"$  ;  $8" \times 4" \times 2\frac{3}{8}"$ .

**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  $\frac{1}{8}"$  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 3 to 4 ft. 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  $\frac{1}{4}$ th of its weight when dry, and a second class brick not

more than  $\frac{1}{4}$ th, after immersion 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 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  $\frac{1}{4}$  in. 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 brick 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, gravee 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 crack, 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 strength of burnt bricks (in flat position) varies from about 40 to 180 tons per sq. ft. (Punjab bricks are on the high side with about 100 tons.) The strength decreases when soaked in water by about 25 per cent. Strength of unburnt (sun-dried) bricks is from 15 to 35 tons per sq. ft. A brick of about 2500 lbs. per sq. in. crushing strength has 960 lbs. per sq. in. transverse strength, 680 lbs. per sq. in. tensile strength, and 250 lbs. per sq. in. shearing strength.

Crushing strength of brick masonry is only about  $\frac{1}{4}$  to  $\frac{1}{2}$  (or even less) of the crushing strength of a single brick, and depends upon the mortar used and the bond. Permissible stresses in brickwork are given in Section 7. Specific gravity of bricks should not be less than 1.8.

For inspection the bricks should be stacked in stacks of 2-brick thick containing 2000 bricks each. The space left in between the two stacks (for passage) should not be less than  $2\frac{1}{2}$  to 3 ft.

Bricks absorb less heat than stones.

800 to 1000 bricks give about 100 c.ft. of brick-bats, and about 1100 to 1200 bricks make 100 c.ft. of brick ballast.

### Breeze fixing bricks :—Composition :

- |  |  |
|--|--|
| 1 Cement   | } Will readily<br>receive<br>screw or nail |
| 4 Coke breeze screened to $\frac{1}{4}$ " to $\frac{3}{4}$ "<br>or screened coal ashes |  |

### 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 refractory 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  $1600^{\circ}\text{C}$ . Crushing strength of bricks should be not less than 1800 lbs./sq. in. Fire-bricks weigh about 150 lbs per c.ft. 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 2 c.ft. ( $1\frac{1}{2}$  cwt.) 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 patterns. 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  $1\frac{1}{2}$ " gauge for foundations

and floor concrete and 1" gauge for roof concrete. No under-burnt or "jhama" bricks (over-burnt porous) should be used. About 1100 to 1200 bricks (9" size) make 100 c.ft. of brick 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  $\frac{1}{2}$  in. thick.

Hollow blocks have certain advantages over bricks: they are only about  $\frac{1}{3}$ rd 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 brick-



work needs to be finer than that for stonework. For ordinary masonry work, concrete, and first coat of plaster, the sand should pass through a sieve of  $12 \times 12$  meshes per sq. in., and for fine works, pointing, or second coat of plastering, sand should pass through No. 14 or 16 B. S. sieve but should not have too much of finer sand. (Also see under "Mortars and Concretes"). Crushed stone is not so satisfactory as natural sand since crushed stone contains a lot of fine particles like dust.

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 powder should pass through  $\frac{1}{12}$  in. sieve (about No. 8 B.S. sieve) for masonry works.

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 B.S. sieve No. 30 ( $50 \times 50$  mesh).

*Pozzuolana* (or *puzzolana*)—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 also manufactured artificially, which is *Surkhi* in India. In fact, surkhi is not an adulterant but makes cement mortars and concretes more waterproof, more resistant to alkalies and to salt solutions 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 1 to 2 inch slump is just as readily placed as a corresponding straight cement concrete of much higher slump. The proportion of surkhi 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 standardized produce and its properties are widely variable. This is still in experimental stage in India and should be used with caution for important

works. (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 increases 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 cracking 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 weighs about 66 lbs./c. ft., expands on slaking and then weighs 40 lbs./c. ft. when fresh, increasing to about 50 lbs./c. ft. 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 screen of  $12 \times 12$  meshes to the square inch, 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 non-porous platform in a 6 to 9-in. 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 about a foot deep in a drum or a tub with about three feet 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 screen of 12 meshes per lineal inch, 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 conditions, 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 its volume, in the preparation of mortar 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 category 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 25 ft. No piece larger than  $\frac{1}{4}$  in. 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, which 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 200 lbs. per sq. in.

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  $\frac{1}{2}$ -in. 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 the joint gives the adhesive strength of mortar which should not be less than 20 lbs. per sq. in. for 1 lime to 2 sand mortar. For testing tensile strength, briquettes are made (as for cement test), and cured for 24 days by immersion in water. A good hydraulic lime should have an ultimate tensile strength of at least 100 lbs. per sq. in. and a fat lime 40 lbs./sq. in., with 1 : 3 lime—sand mortar. The compressive strength should be 4 to 5 times its te-n



sile strength. Mortar consisting of 1 part good quality kanker lime and 1 part sand should develop a compressive strength of over 700 lbs. per sq. in. after 3 months and twice that after two years. A pure surkhi mortar gives a breaking strength of about 80 to 90 lbs. per sq. in. if left in dry air, and 300 to 350 lbs. per sq. in. if left immersed under water.

Compressive strength of 1 : 4 cement-sand mortar after 3 months is over 3,000 lbs. per sq. in

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 it 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  $\frac{1}{4}$  in. to  $\frac{1}{2}$  in.

### 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 2-in. 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 a few feet below the surface

in the low-lying portions of the catchments of nullas and rivulets. The nodules are of sizes varying from  $\frac{1}{2}$  in. to 4 ins. 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. (Also see 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 p.c. of pure carbon, and slightly lighter than water. One ton of average Indian coal yields approximately : 0.5 ton of coke, 12 gallons 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 con-



## 6. MIXTURES FOR MORTARS &amp; 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.

Mortars are usually made largely by guess-work, but to secure the best results (which are often the cheapest) and effect saving, it is necessary to determine the best proportions. The following proportions are suggested which are by volume of dry materials; cement is by weight or by bags.

## Mortars for Masonry Works

	Cement	Sand	White lime	Surkhi*	Masonry Strength %
General ...	1	3 to 8	—	†	100—60 %
„ and hollow blocks	1	6	1	—	96 %
„ common ...	1	9	2	—	86 %
See Section 7	1	36	9	—	59 %
Fine sand ...	—	2	1	—	45 %
Coarse sand ...	—	3	1	—	
Common ...	—	1	1	1	Strong
—	—	—	1	½ to 2	
—	—	1	—	1	
—	—	4	2	1	
—	—	—	5 +	12	
Kankar lime + ...	1	—	—	Cinders	Strong
Slow-setting mortar	—	—	1	1 to 3	
„ „	—	1½	1	1½	Hydraulic lime
Structures below ground or water-level and subject to load ...	—	1	1	—	
Structures above ground-level, subject to load ...	—	2	1	—	
Less important structures ...	—	3	1	—	
„ „	—	—	—	—	

\*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 finely ground Surkhi where salts are found, and cement proportion should be high.



cretes and is much lighter and porous. Generally has ashes mixed with it.

**Furnace Slag.** Slag is a waste product obtained 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 cementitious 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 which 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 large 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 varieties 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 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 a 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.

**Mortars for Plastering**

	Cement	Sand	Lime paste	Surkhi*
General	1	2 to 6	—	—
Water-tight mortar	1	2 to 3	$\frac{1}{2}$ to $\frac{1}{2}$	—
Under-coat	1	4 to 6	1	—
Finishing coat	1	9	2	—
			Lime (hydrated)	
For chimney breasts	1	8 to 10	3	—
Water-tight finishing	1	10 to 12	3	—
Deteriorated cement	1	—	2	—
Inside work (better)	1	12	7	—
„ first coat	—	2	1	—
„ common	—	—	1	2
—	—	14	1	$\frac{1}{2}$ to 1
„ finishing coat	—	1	—	1
			Hydraulic lime	
Outside work	—	2 ot 3	1	—

**Mortars for Pointing**

	Cement	Sand	Lime paste	Surkhi
Outside work	1	1 to 3	—	—
	1	3	1	—
	1	6	1	—
	1	10	3	—
	1	9	2	—
	$\frac{1}{2}$	—	1	1 to 2
	—	$1\frac{1}{2}$	1	—
	—	1	1	$\frac{1}{2}$ to 1
Kankar lime	—	—	1	—

**Lime Concretes**

	Sand	Lime	Surkhi
Fat lime concrete is not suitable for use in large masses	$1\frac{1}{2}$ to 2	1	—
	—	1	2
	1	1	1
Kankar lime	2	1	1
	—	1	—

For various proportions of cement concrete see Section on Estimating.

**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 screen of 40-mesh to the lineal inch or a coarse cloth. Again kept immersed in water for 7 days, when the excess water is poured off.

**Cement/Lime Mortar.** (Also see 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-cement / 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 mesh of  $12 \times 12$ .

(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 pound of washing soda is mixed in three galls. of water used for the preparation of cement/sand mortar, it will make a water-tight mortar.



(vi) 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  $\frac{1}{16}$  in. screen and not more than 60% should pass the  $\frac{1}{8}$  in. screen. (Size of sand particles is given in "Soil Mechanics".)

(vii) Fine sand is used for plaster and pointing, passing through No. 16 B.S. sieve, but not more than 10% should pass through No. 100 B.S. sieve. (This fine sand is not suitable for R.C.C. works). Very fine sand will produce cracks.

(viii) Cinders should be screened through 64 mesh to the sq. in. 100 c. ft. coarse cinders = 67 c. ft. cinder powder. More of cinders gives a leaner mix.

(ix) Cement which has deteriorated in respect of quick-setting can be used in place of lime and makes better mortar than fat lime.

(x) Chopped jute  $\frac{1}{4}$  in. to  $\frac{1}{2}$  in. long (or hair) is added to lime mortars for plastering in the proportion of about 30 lbs. to every 100 c. ft. of slaked lime.

(xi) Addition of a little powdered soap-stone increases the whiteness and polish of a lime plaster.

## 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 or solidification 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 varieties; free silica or quartz can be seen without the use of a lens. Granite is an acid rock. **Basic rock :** 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, Limestone and Shale are examples. Gravel, sand, silt, clay and peat are considered as uncemented and unconsolidated sedimentary rocks.

**Metamorphic rocks :** Are either igneous or sedimentary 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.

*Dry.* Seems 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 crust.

*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 hori-



zontal 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 radial 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 road 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** : Is 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 and consist essentially of carbonate of lime intermixed with certain impurities (silica, alumina, iron, etc.), and minerals. There are many varieties 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 varieties 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 sandstone are sharpness or fineness of grains, clear and shining 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 layers. Sandstones vary much in quality and many varieties 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  $\frac{1}{4}$ -in. 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 as road metal or railway ballast unless of the harder quality. They do not stand abrasion.

**Quartz :** A metamorphosed mineral rock composed entirely of silica.

**Quartzite** (including Hornstone or Flintstone) : Is a form of silicious sandstone composed of quartz grains cemented together with silica. It is the hardest and strongest rock with stratified structure and crystalline texture, more compact and dense than sandstone. Quartzites are hard to work and break up into irregular cubes with uneven lustrous surfaces. Colour varies from brown red to yellowish white. Used for stone walling where quarried in large blocks, also for concrete aggregate and road metaling.

**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 thin 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 to 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  $\frac{1}{2}$  inch, it may be considered as practically non-absorbent. Or alternatively, a good slate after 12 hours' soaking should not have absorbed more than  $\frac{1}{200}$ th 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 spongy 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 masonry 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  $\frac{3}{4}$  in. 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 5 ft.



**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 1500 lbs./sq. in. 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 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. Stones newly quarried contain quarry-sap and can be more easily worked in this

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\*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 finishes than from the coarser grained varieties.



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 having a dense texture receive a good polish. Granites, marbles, slates and compact varieties of limestones 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.

*Slate* : Chamba, Dalhousie, Rajasthan.  
 Slates of Southern India are of poor quality.

*Marble* : Gwalior, Jaipore, Kashmere and Southern India.

*Granite* : Ajmer, Bangalore, Dalhousie, Jhani, Jubbulpore, Mysore, Madras, Secunderabad, Behar, Orissa, Cutch.

*Basalt or Trap* : Bombay, Madhyabharat, 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, one inch broad, supported at both ends and loaded in the centre :—

Depth in inches	Clear span in feet									
	1	2	3	4	5	6	7	8	10	
	Safe centre load in pounds									
1	10	5	—	—	—	—	—	—	—	
2	40	20	13	10	—	—	—	—	—	
3	90	45	30	20	17	—	—	—	—	
4	160	80	50	40	30	25	20	—	—	
5	250	124	80	60	50	40	35	—	—	
6	360	180	120	90	70	60	50	40	30	
7	490	245	160	120	95	80	65	60	45	
8	640	320	210	160	125	105	90	75	60	
10	1000	500	330	250	195	160	140	120	95	
12	1440	720	480	360	280	240	200	175	140	

The weight of beam allowed for in the above table is 170 lbs. per c.ft. 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 load should be allowed.

For good marble or hard lime-stone on bed, the same loads as above should be allowed.

## 8. QUARRYING & 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  $\frac{1}{2}$  in. to 2 in. in diameter, 4 in. to 6 in. apart and about 6 in. to 8 in. 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.



sp. 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 6 to 10 ft. long,  $1\frac{1}{2}$  ins. diameter with chisel end of hard cast steel welded to it. (b) Tamping needle of brass or copper of slightly smaller 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 pound gelignite with 6 detonators explodes about 100 c.ft.

**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 fulminating 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. Six detonators are required for about one pound 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 2 ft. per minute for ordinary fuses and 100 ft. per second for instantaneous fuses. Usual length per charge whether used with gun-powder or with detonator, is 3 ft. 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 before they are taken to the place of use.

**Dynamite and Gelatine.** Is nitroglycerine 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 2 to 6 ft. depth — 1 in. dia.

„ 6 to 11 ft. depth —  $1\frac{1}{2}$  ins. dia.

„ 11 to 16 ft. depth — 2 to  $2\frac{1}{2}$  ins. 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 5 ft. deep and the distance apart should be from  $1\frac{1}{2}$  to 2 times their depth. The charge should always be placed in a sound piece of rock and if possible not nearer to a crack or fissure than 1 ft.

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  $\frac{1}{3}$ rd 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 8 inches 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 water-proof cartridge made either of thin plane iron sheet or water-proof paper of the necessary length so that the top of it is six inches 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. An inch or two 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 is 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 1/16 in. dia.) smeared with grease is inserted in the tamping material 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 powder required in oz. (approx.) =  $\frac{1}{2}$  line of least resistance in ft. One lb. of powder will loosen about 50 c.ft. of soft and 30 c.ft. of hard 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 1 to 3 ins. in dia. and 1 to 4 ft. deep for blasting with powder. The depth of the 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 50 lbs. of dynamite should be destroyed at a time. A clear space of ground, about 100 yards 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 inch 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 20 ft.) 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.

### **Precautions 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 be 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 one 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, and the hole marked in a distinguishing manner. Another hole should be jumped at a distance of more than 18 inches 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 passed 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 150 ft. (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 a quarter of a mile 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 500 ft., before an explosive charge is fired. (b) For making the hole in an explosive cartridge 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 frosty 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 does 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 lead 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 lb. of white lead to every gallon of oil and allowing it to settle for at least a week. 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 oil 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.

Linseed 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 odour 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 lb. of each to 1 gall. 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 excess in a 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 tests 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 quarter lb. to a gall. 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

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\*Zinc sulphate is used as a 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.



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 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). Also see "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 lighten 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 : a gall. of paint covering 10,000 to 15,000 sq. ft. 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 are used only for special purposes such as motor cars, airoplanes, 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 makes 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 2 oz. white lead,  $\frac{1}{2}$  oz. litharge, and 4 oz. boiled linseed oil is used. **Size** is made from superior glues. A pound of glue makes one gallon size; double size is of double consistency and one pound makes  $\frac{1}{2}$  gall. 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.

**Knottting.** 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 5 oz. of pure shellac dissolved in a pint of methylated spirit and when thoroughly dissolved,  $\frac{1}{4}$  oz. 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, (3 oz. linseed oil to 1 lb. 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  $\frac{1}{3}$  of white lead to  $\frac{2}{3}$  of ordinary putty. For varnishing, the wood surface can be stopped with hot weak glue size (one lb. of glue making about one gallon 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 follows: Put a cupful of common shellac in an iron pot, add a tea-spoonful of resin, a piece of bees-wax (about 1 in. dia.) and a tea-spoonful 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 (7 lbs. of red lead, or 7 lbs. of red and white lead, mixed with  $\frac{3}{4}$  gallon of oil). When dry, all cracks or holes are filled up with putty and the whole surface rubbed down with pumice stone or sand-paper, 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 lb. of pumice is required for rubbing down 1000 sq. ft. 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 their coats, thick coats take longer to dry and generally begin to flake off after some-time.

*If lead paint has been used, the dry rubbing of lead painted surfaces must invariably be prohibited.* Dry sand-papering 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 2 lbs. of white zinc, 1 lb. turpentine and  $\frac{1}{2}$  lb. 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. If the old surface is firm and sound it should be rubbed down with pumice or soap-stone and washed with dhobi's earth and water. Surfaces marked by smoke or otherwise dirty should be given a coat of 3 lbs. glue and 3 oz. unslaked lime boiled in one gallon of water.



All greasy places should be brushed over with turpentine and then washed with soap and water. Before the priming coat and also before each and every subsequent coat of paint is applied, the work should be rubbed smooth with sand-paper or fine pumice stone. If the old paint is greasy with smoke, mix the first coat with spirits of turpentine instead of oil.

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 removes are available and in absence of the same one of the following methods may be used :—

(a) With Caustic Soda : 2 lbs. caustic soda to a gallon of water. To be used with great caution ; not to be left for more than few hours on the wood and should not be touched by hand. After this treatment surface should be washed well with clean water and neutralized with a weak acid solution or vinegar. A piece of cloth is securely tied on one end of a long wooden stick, which is dipped in the soda solution and rubbed on the painted surface. Its use should be deprecated, it is dangerous to the operator, skin and eyes.

(b) Wet the paint with naphtha, repeating as often as required, and when softened rub the surface clean.

(c) Dissolve 1 lb. country soda (Sajji) in hot water and mix into 4 lbs. of stone lime reducing the whole to a creamy paste. The work should be well coated with it and left (but kept moist) for about 3 hours, when the paint will be easily removed. If unslaked lime is used and the mixture applied hot, the action is somewhat quicker.

(d) Apply a solution composed of :—Soft soap, 1 part ; Potash, 2 parts ; Quick-lime, 1 part. The soap and potash are first dissolved by boiling in water, the

lime is then added and the whole applied while still hot with a brush on the surface of the paint or varnish to be removed and left on for 12 to 24 hours, after which the paint is easily removed by washing with hot water.

(e) 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  $\frac{1}{2}$  lb. of shellac in a pint 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 5 lbs. of whiting mixed in  $\frac{1}{2}$  gall. of methylated spirit and then sandpapered when dried.

*Fillers* can also be made as follows: (i) Whiting mixed with water; (ii) Linseed oil and bee's wax (3 : 1) boiled; (iii) Plaster of Paris 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 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 bee's 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 ceilings 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 lb. of glue makes about a gall. 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. One pint of varnish will cover about 150 sq. ft. 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 positions.

**Oiling Woodwork.** One pound of bee's wax mixed with 3 lbs. of double boiled linseed oil are heated over a slow fire till the wax is melted. After the mixture has cooled, 1 lb. of turpentine is added. This will cover about 800 sq. ft. of surface. The woodwork can also be oiled with country sweet oil to which equal parts of vinegar and tur-



pentine 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 is 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 given 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. One maund is enough for about 800 sq. ft. 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: 1 gall. of oil to 33 lbs. 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: 6 lbs. of red oxide paint; 1 lb. lamp black; 1 gall. of boiled linseed oil.

The third coat can be of: 7 lbs. of red oxide paint; 1 gall. 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 wt.; Linseed oil, raw, 4 parts; Linseed oil, boiled, 1 part; Turpentine 1 part. One gallon of this paint will cover about 400 sq. ft. of surface, two coats. For further details see Section on "Estimating." One cwt. of powder and 8 gallons of thinnings will cover about 4500 sq. ft. 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 0.003 in. per year of face exposed to sea water, and 0.002 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 6 to 8 ounces of copper acetate added to a gallon of water, or 2 ounces of muriatic acid added to a mixture of 2 ounces each of copper chloride, copper nitrate and sal-ammoniac, dissolved in a gallon 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 2500 to 3000 sq. ft. 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 stated 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.

The following mixture has been found useful :—Coal-tar, 84 lbs. ; Mineral pitch, 10 lbs. ; Slaked white lime or cement, 9 lbs. ; Kerosene oil, 9 lbs.

The mixture is prepared by heating the pitch and coal-tar separately, then mixing them together over a fire, stirring well and adding the slaked lime gradually while stirring. Then withdrawing from the fire and adding kerosene oil and stirring well. Care should be taken to see that the mixture is not overheated, 350 to 450 degrees is the correct temperature ; the pitch scales off if the mixture has been burnt.

*Quantity required*—The average covering capacity of the mixture is about 2500 sq. ft. per cwt.

Two to three coats are sufficient. The mixture should be applied hot. Subsequent coats to be given only after the previous coats have dried.

Anti-corrosive black enamel paint is available for iron-works in water.

#### **Paint for Steel Water Tanks : (inside)**

(i) White lead	27 lbs.	} Not recommended for drinkingwater
Boiled linseed oil	3½ pints	
Raw linseed oil	6 "	
Turpentine	1 "	

(ii) The inside of all steel tanks can be painted with two coats of bitumastic solution.

#### **Painting Plaster : (Also see 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 5 lbs. of zinc sulphate in a gallon 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 (1 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, 2½ galls. ;

Benzoline, 2 gall.;

Pale resin, 14 lbs.

Shake them in a vessel; when completely dissolved add 24 lbs. 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 proprietary 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 paints 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 curing 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 1 gallon of mixed paint is considered sufficient for 100 sq. ft. of smooth surface and 40 sq. ft. 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) :

10 lbs. lamp black		Will cover about 800
6 lbs. dry white lead		sq. ft. of surface.
10 pints boiled linseed oil		
1 pint turpentine.		

### **Coal Tarring**

Add 2 lbs. of unslaked lime to every gallon 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  $\frac{1}{2}$  pint country spirit to 1 gallon 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. If possible, the articles to be tarred should be dipped into the tar. Not less than 10 lbs. of coal-tar should be used per 100 sq. ft. 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 painting.

### 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 be used for another colour or is no longer required, it should be cleaned at once by dipping into kerosene oil. Old brushes should be kept in water or raw linseed oil (converging 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 and is manufactured in thicknesses varying from  $1/15$  in. to  $1/5$  in. The thickness is specified by weight in oz. per sq. ft. The following sizes and weights of glasses are generally used for window panes, etc.

Ordinary quality-small sizes: 15 oz. "thirds". No glass panes below 13 oz. should be used for doors and windows.

Better quality-sizes up to :—

10" × 8"	16 oz.	"seconds" (about $1/14$ " thick)
10" × 12"	16 oz.	"
12" × 14"	16 oz.	"
18" × 18"	18 oz.	"
24" × 24"	21 oz.	" (about $1/10$ " thick)
30" × 30"	26 oz.	" (about $1/9$ " thick)
36" × 36"	32 oz.	" (about $1/7$ " thick)

For sizes above 36" × 36" use plate glass  $\frac{1}{4}$ " thick.

There should be a space of  $\frac{1}{8}$ " 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  $\frac{3}{8}$  in. to 1 in. (usual thicknesses are  $\frac{3}{8}$ ",  $\frac{1}{2}$ ",  $\frac{3}{4}$ ",  $\frac{1}{2}$ ",  $\frac{3}{4}$ " and 1"). 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) glass 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 sandwiched.

**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 proprietary brand of heat-resisting glass which is used for making cooking utensils, electric insulators and laboratory apparatus, 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 lb., linseed oil 4 oz., varnish 1 oz., mix the whole till becomes plastic. The mixture is applied by tying it up in a piece of linen into balls about 1 in. 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 are fixed 3 to 6 inches apart. Rebates should be at least  $\frac{1}{4}$  in.

Putty can be made by mixing 2 lbs. of powdered whiting, 2 oz. of white lead (dry) with 12 oz. of raw linseed oil 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. For glazing in metal sashes, 5 per cent red lead should be added. Litharge is sometimes added as a drier. (Whiting is absolutely dead stone lime or finely powdered chalk). Glass panes can be cleaned by : (i) Methyiated spirit; (ii) Painting the glass panes with lime wash and leaving it to dry and then washing with clean water; (iii) Rubbing finally powdered chalk; (iv) Rubbing damp salt for cleaning paint spots.



## SECTION 13

### ARBORICULTURE

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## 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 or 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 3 to 4 ft. high 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 K'kar 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 5 to 10 ft. long with the lower end cut obliquely and set 2 or 3 ft. 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 bark 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 seed 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 seed will germinate in a short time. The seed should be collected in dry weather and should immediately be spread out to dry in the shade. When dried the seed should be cleaned and useless outer covering removed. As far as possible, the seeds used for sowing should be fresh, and it is best to sow seed 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 3 inches 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-lying, the trees should be planted on mounds about 1 ft. high and with 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 should be at least 4 ft. wide at top and 3 ft. wide at bottom and 3 ft. deep or 4'  $\times$  4'  $\times$  4'; normally, pits 3'  $\times$  3'  $\times$  3' are sufficient. 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 1 ft. 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 4 to 5 ft. 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, two feet wider than the shadow at noon, is covered to a depth of about 3 ins. 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 "gharas" 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 poured in the tree stands on a small mound surrounded by water. It is important to keep the surface soil for a foot or two distance all round the trees loose to effect proper watering.

One man can hand-water about 300 plants a day.

### **Fencing Young Plants**

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 (6 ft. 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 complete sunlight.

### **Turfing**

Is artificially planting grass on soil banks. Turfing is generally done on banks of erodible soil. After the bank has been made to shape, the top earth is loosened to a depth of about 3 inches and if the soil is not suitable for grass growth, a better soil about 2 inches 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  $2\frac{1}{2}$  inches and cut to 12 ins. by 18 ins. pieces. The strips should be placed on the prepared bank slope parallel to 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 water over the edge and on to the top of the sod.

Turfing should be carried out immediately after the moonsoons 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

Site should be level and of sandy loam soil where water is available according to requirement. Trenches of dimensions 12" top width and 8" bottom width and 9" to 12" deep should be dug 6 ft. apart, or as necessary according to the plants. The excavated earth should be thrown 1 ft. 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.

Seedling: will grow with greater rapidity 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 seer of seeds:—

Ber	270	Kikar	4000	Shisham	6000
Farash	3000	Pipal	1500	Mango (desi)	100

#### Number of seeds in one seer 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	Khirmi, Rayan	5400
Umber ..	10000	Asok, Devidari	400
Mango ..	40	Karauj	200
Safed or Dun Siris	1000	Padouk ..	1000
Candle-nut tree	250—300	M hogany ..	1000
Jack, Phanas	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 15 to 30 ft., fruit trees 10 to 15 ft., grain crops 4 to 5 ft., and ordinary garden crops  $1\frac{1}{2}$  to 3 ft. Sandy loams or clay loams are fertile and suitable for tree growth.

*Sandy soil*—In sandy soils large sheesham stumps having 1 ft. to 2 ft. long roots can be planted with success. Kikar and Ber can also be grown. In sandy tracts with an annual rainfall of about 8 inches, the species known as "mosquito" should be tried.

*Kalar soil*—Pits 3 ft. dia. and 3 ft. 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 2 ft. dia. and 2 ft. 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.

### 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 two to three feet 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.

*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 covered 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 300 to 400 maunds of such manure is used per acre.

#### 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 the height of summer. Trees that develop straight and clean trunk up to a height of about 10 to 12 feet 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 road side 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.

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 6 to 10 ft. away from the outer edge of the side width (berm) or, min: horizontal clearance should be 22 ft. for single lane roads, increasing clearance by 10 ft. for each additional lane width. Rows on opposite side of the 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 stems free from branches for a height of 8 to 10 ft. on highways and 9 to 11 ft. in streets of towns. If the branches grow horizontally it may be necessary to shorten some of these; but the height of 11 ft. 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 5 ft. 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 5 ft. high. Most of the failures in road-side 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.

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 50 ft. away from the road.

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 30 to 50 ft. according to the species of trees; the average being taken about 130 trees in a mile. 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 30 ft. apart. In dry districts the comparatively slow growing trees can be planted at larger intervals of say, 50 ft., 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 10 to 12 ft. 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 20 ft. of any masonry work and this distance should be increased to 80 ft. minimum (prefer 200 ft. 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 Spacings:**

Banyan	40 ft.	Mango	35-40 ft.
Bahera	50 ft.	Neem	33 ft.
Indian Cork	20 ft.	Pipal	55 ft.
Jamun	40-50 ft.	Siris	40 ft.
Karanj	33 ft.	Tamarind or Imli	40 ft.
Khirni	40 ft.	Teak	40-50 ft.
Mahwa	40-50 ft.		

**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 6" x 6", at 4 ft. 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 2 to 4 miles 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 weak 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. Two chhataks of copper sulphate is dissolved in  $\frac{1}{2}$  gall. ( $2\frac{1}{2}$  seers) of boiling water. Two galls. of lime water is added afterwards at the time of spraying. Boil 4 chhataks of dried tobacco leaves in  $\frac{1}{2}$  gall. of water for about an hour. Dissolve in this 1 chhatak of hard soap; add 3 galls. of cold water for use.

Parasite plants should be completely removed, root and all. If a branch is severely 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 interesting 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 a foot from the trunk, cutting the 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 Cavities.** 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 permanganate of potash and fill it with 1 : 2 : 4 cement concrete rammed well and finish off with a little 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 2 ft. 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

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 winter, the wounds remain uncovered for a long time; if done during warm weather, or when the trees are in full growth, the wounds bleed, and tar, if used, will not adhere to them.

Pruning should begin while the tree is young 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 about 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 as has been explained before, but pruning should be done with a scissors or a saw. The proper season for *lopping* is the winter when the sap is down.

### **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 round 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 COMMON TREES**

### *Arjun*

Is a large white tree which grows very tall with smooth trunk and branches. A magnificent 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 preceeding sowing, the seeds are soaked in cowdung water. The seedling attains 2 to 3 feet height in about 3 months and grows up to 8 feet 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 rapidly take root and develop into independent stems. This tree is easily propagated either from seeds or by cuttings 8 to 10 ft. 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 should 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 the height of 15 ft. 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 in rains. Ripens in May/June. Grafting is done, which blossom in Oct. 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 Nov. Ripens in Oct. 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 star-shaped 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 1 to 1½ ft. 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 trans-planting 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 white 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 due to 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 30 ft. apart and then keeping the strongest seedlings in the nursery and transplanting 30 ft. 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 40 to 50 ft. or a greater thickness than 2 ft. 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 one ft. wide and one ft. 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  $1\frac{1}{2}$  ft. dia. 6" deep, 6 ft. apart and in rows 10 ft. 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 building. 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 4 ft. diameter and about 4 ft. 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 waterings 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 30 to 40 ft. 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.

#### *Mohva, 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 put 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 ripens 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 3 to 4 ft. high, and planted out with earth round the roots in Dec. or in the rains, and watered frequently. 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 6 ins. apart in trenches 1 ft. 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 3 ft. deep and filled in with a mixture of  $\frac{2}{3}$  manure and  $\frac{1}{3}$  earth. Requires constant moisture and heavv manuring.

*Shisham, Sis, 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 also 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 watering 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  $2\frac{1}{2}$  ft. long and  $\frac{3}{4}$  in. 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 10 to 12 ft. 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 showing 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 against browsing 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 40 or 50 ft. in height and 5 to 6 ft. 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 9 ins. and that of the shoot about 3 ins. 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 70 to 80 ft. and of a girth up to 25 ft. It is drough 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 classes 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 5 to 6 ft.

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.

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## SECTION 14

### HYDRAULICS

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## 1. HYDRAULIC DATA

1 c. ft. of water	=6.24 galls. (Imperial) =7.48 galls. (U.S.) =62.4 lbs. in weight (fresh) =64 lbs. sea water =28.32 litres
1 gallon (Imperial)	=10 lbs. in weight =4.54 kg. =0.16 c. ft =277.46 c. ins. =8 pints =160 fluid ounces =4.546 litres = 0.16051 c.ft. =1.2 U.S. galls.
1 litre	=2.2 lbs. =0.0353 c.ft. =0.22 gall.
1 cwt. of water	=1.8 c.ft. =11.2 galls.
1 c.ft. per sec.	=373.7 galls. per minute =22,424 galls. per hour =538,176 galls. per day.
1 gall. per min.	=0.00267 c.ft./sec. =1440 galls. per day.
C.ft. per min. $\times$ 9000	=galls. per 24 hours
Tons $\times$ 224	=gallons, or 1 ton =224 galls. =1000 litres
Tons $\times$ 36	=c.ft. (approx.)
Specific gravity of water is taken as 1.	
1 ft. head of water	=62.4 lbs./sq ft. =0.4335 lbs./sq. in. =0.881 in. of mercury.
2.31 head of water in ft.	=1 lb. per sq. in.
Head in feet $\times$ 0.433	=lbs./sq. in.
Pressure in lbs./sq. in. $\times$ 2.31	=head in feet of water or (Pressure in lbs./sq. in. =2.31 feet head of water).
1 inch column of mercury	=1.13 ft. head of water =0.49 lb./sq. in.
1 acre covered 1 inch deep	=22,650 galls.
1 acre-foot	=1,613 c.yds. =43,560 c.ft.

*American Measures :*

- 1 U. S. gallon = 0.833 Imperial gall.  
 (Gallons used in India are the Imperial gallons)  
 = 231 c. ins. = 3.773 litres = 8.33 lbs. = 3.78 kg.  
 1 c.ft. = 7.481 galls.  
 1 c.ft. per sec. = 449 galls. per minute  
 = 646,963 galls. per day  
 1 c.ft. per sec. is sometimes called "second foot".

- 1 c.ft. of fresh snow = 5 to 12 lbs.  
 1 c.ft. of ice = 57.4 lbs.  
 1 c.ft. of wet and compacted snow = 15 to 50 lbs.

If  $V$  is the velocity in feet per second, the delivery in gallons per minute through a pipe of  $d$  inches diameter  
 =  $2.0 V d^2$  or say  $2 V d^2$  (approx.).

Theoretical velocity with respect to head of water or pressure is taken :

$$V = \sqrt{(2gH)} = 8.025 \sqrt{H}$$

$$\frac{1}{2g} = 0.0155$$

$$H = \frac{V^2}{2g} = 0.0155 V^2$$

= the head of energy required to produce a velocity of  $V$  feet per second. It is also called "velocity head".

where :

$V$  = velocity in ft./sec.,  $H$  = head of water in feet.  
 (This is head or energy required to produce a velocity.)  
 $g$  = acceleration or gravity constant = 32.2 ft./sec<sup>2</sup>.

1 Horse-power = 33,000 lbs. raised 1 ft. high in 1 minute,  
 = 550 lbs. raised 1 ft. high in 1 second,  
 = power exerted by 8.8 c. ft of water falling  
 1 ft. in 1 second.

(The strength of 1 horse is considered equivalent to 5 men.)

*Atmospheric Pressure :*

(The atmosphere is considered to extend to a height of at least 45 miles above sea level.)



The "atmosphere" exerts a pressure of 14.7 lbs./sq. in. which is equivalent to a static head of 33.95 feet of water or 29.92 inches of mercury at mean sea level. (1 atmosphere is usually taken = 15 lbs. per sq. in. or 34 feet of water.)

#### *Compressibility of Water :*

*Pascal's Law*—The pressure per unit of area exerted anywhere on a mass of liquid is transmitted undiminished in all directions ; and any surface in contact with the liquid will be subjected to this pressure in a direction at right angles to the surface.

The compressibility of water under pressure is very slight and it recovers its original volume immediately after the pressure is removed.

#### **Properties of Fluids Compared with Solids :**

- (i) Water spreads evenly on a level surface.
- (ii) Frictional resistance to the motion of water increases with velocity, whereas in the case of solids friction is independent of velocity.
- (iii) Friction between solids increases with pressure, while loss of head incurred by water is independent of pressure.
- (iv) In solids friction is independent of area, in liquids it directly depends on the same.

#### **General Properties of Water and Ice :**

Pure water is usually assumed to boil at 212°F. or 100°C. in the open air at sea level, and at about 1° less for every about 520 feet above sea level for heights within one mile. In other words, if we ascend a mountain (air pressure diminishes) to a height of  $H$  feet, and the number of degrees  $D$  to be deducted from 212° for the actual boiling point is given by the relation :  $H = 520 D + D^2$ . In a metallic vessel it may boil at 210°F., and in a glass one, at from 212° to 220°F. It evaporates at all temperatures and has a greater capacity for heat than any other known substance. Water attains its maximum density at 39.2°F. when it weighs 62.425 lbs. The weight decreases with rise in temperature and at 100°F. it is 62 lbs.

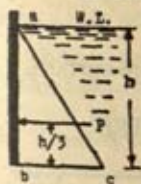
Pure water is converted into ice at  $32^{\circ}\text{F.}$  or  $0^{\circ}\text{C.}$ , when it weighs 57.4 lbs./c.ft., hence as ice, it has expanded one-twelfth of its original bulk as water. This sudden expansive force exerted at the moment of freezing is sufficiently great to split iron water pipes; being probably not less than 30,000 lbs. per sq. in. A floating piece of ice has only  $1/12$ th of its thickness above water.

**Water Pressure.** Fluid pressure acts normal to the surface and the intensity of pressure at any point is equal in all directions.

If pressure is applied to the surface of a liquid, the liquid transmits the pressure equally in all directions and with undiminished intensity.

A column of water 2.307 ft. high (or 2.036 inches of mercury) produces a pressure of 1 lb./sq. in. on the base, or the base of a column 1 ft. high will have a pressure of 0.4335 lb./sq. in.

Water standing against a wall exerts pressure equal to  $P$  in the form of a triangle  $abc$ , acting through the centre of gravity of the triangle at a distance of  $h/3$  from the base, whence :



$$P = \frac{Wh^2}{2} = 31.2h^2$$

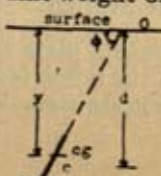
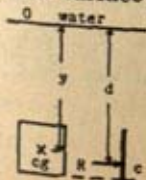
Total moment of  
water =  $10.4h^3$

$W$  = wt. of 1 c.ft. of  
water, viz., 62.4 lbs.,  
 $P$  = total pressure,  
 $h$  = depth of water.

Intensity of water pressure at any point below water surface is  $Wh +$  any pressure on the free surface of the water.

**Water pressure on submerged surfaces :—**

Total pressure on any submerged area (single face) = area  $\times$  depth of the centre of gravity of the area below the free surface  $\times$  unit weight of water.



Resultant Pressure  $R$  is acting at its centre of pressure at c.g. is the centre of gravity of the area of surface  $A$ .

$$\text{Depth of centre of pressure} = d = \frac{I_o}{A y} = \frac{I_g + A y^2}{A y}$$

$I_o$  = Moment of Inertia of the figure about 00.

$I_g$  = Ditto. about horizontal axis through centre of area. Total pressure on the plate will be =  $W a y$

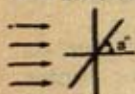
For inclined surface,  $d$  will be  $\frac{I_o \sin^2 \phi}{A y}$

$$I_o = I_g + \frac{A y^2}{\sin^2 \phi}$$

### Pressure of water in motion against a plane normal to the direction of flow

$$R = 1.8 V^2 A$$

$R$  = resistance of a plane normal to the current in lbs./sq. ft.,



$V$  = velocity of the current in ft./sec.

For a plane oblique to the current :

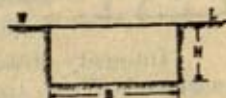
$$R^1 = R \times \frac{2 \sin^2 a}{1 + \sin^2 a}$$

$A$  = area of the surface.

### 2. DISCHARGE THROUGH NOTCHES

$$Q = \frac{2}{3} c B \sqrt{2g} H^{3/2}$$

$$= 3.33 B H^{3/2}$$



For submerged notches

$$Q = c A V$$

$Q$  = discharge in c.ft./sec.,

$c$  = co-efficient of discharge,

$A$  = area in sq. ft. =  $B \times H$

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 :

$$(i) Q = \frac{4}{15} cB \sqrt{2g} H^{3/2}$$

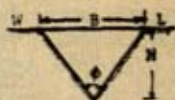
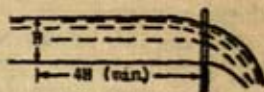
$$\text{or } = \frac{8}{15} c \sqrt{2g} \tan \frac{\phi}{2} H^{5/2}$$

(ii) *Notch with right angle :*

$$Q = \frac{8}{15} \sqrt{2g} H^{5/2} = 2.54 H^{5/2} \text{ (for sharp edged)}$$

$$= 2.54 H^2 \sqrt{H}$$

$$\text{or } Q_1 = 0.305 H^2 \sqrt{H}$$



$H$  = height above bottom of notch in inches, ft.

$Q_1$  = discharge in c. ft. per minute.

Co-efficient of discharge  $c$  varies from 0.593 for a right angle notch to 0.62 for greater angles.

The V notch is recommended for discharges up to 20,000 galls. per hour. For smaller discharges a half  $90^\circ$  V notch may be used ; the discharge of water over a half  $90^\circ$  V notch is taken half that over a  $90^\circ$  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 (3 ft. min.) The depth of the bottom of the channel below the apex of the notch should not be less than 6 ins. on the downstream side, while on the upstream side it should not be less than 12 ins. for heads up to 9 ins., nor less than 18 ins. 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 1/16-in. thick should be fixed over them to keep accuracy of form and permanent correct sharpness of edges.

Discharge over 90° Vee Notch, Sharp Crested, in Galls. per Hour.

H ins.	0	1/8	1/4	3/8	1/2	5/8	3/4	7/8
0	0	0.6	4	11	22	32	60	90
1	112	150	195	248	308	377	456	538
2	635	732	850	975	1,100	1,250	1,400	1,570
3	1,750	1,938	2,135	2,350	2,535	2,800	3,050	3,311
4	3,581	3,800	4,150	4,480	4,820	5,160	5,535	5,870
5	6,280	6,650	7,070	7,520	7,970	8,420	8,900	9,380
6	9,900	10,421	10,940	11,500	12,060	12,650	13,250	13,900
7	14,540	15,200	15,800	16,550	17,240	18,000	18,750	19,500
8	20,300	21,100	22,100	23,000	24,000	25,000	26,000	27,000

90° Vee NOTCH (Sharp Crested)

Head ins.	Discharge		Head ins.	Discharge	
	Cusecs	Galls./mt.		Cusecs	Galls./mt.
3.0	0.080	29.78	9.5	1.389	519.37
3.5	0.117	43.65	10.0	1.578	589.83
4.0	0.163	60.79	10.5	1.781	665.68
4.5	0.218	81.41	11.0	1.999	747.09
5.0	0.283	105.72	11.5	2.232	834.17
5.5	0.358	133.91	12.0	2.480	927.01
6.0	0.445	166.16	12.5	2.744	1025.8
6.5	0.542	202.65	13.0	3.025	1130.6
7.0	0.652	243.59	13.5	3.321	1241.5
7.5	0.773	288.98	14.0	3.635	1358.7
8.0	0.907	339.14	14.5	3.965	1482.2
8.5	1.054	394.16	15.0	4.313	1612.2
9.0	1.215	454.20	—	—	—

Doubling the depth increases the discharge 5.65 times ;  
trebling the depth increases the discharge 15.57 times.

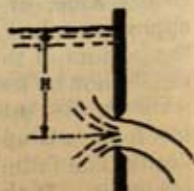
### 3. ORIFICES

**Sharp Edged Orifice :** (Small)-Free fall

$$Q = cA\sqrt{(2gH)} = 5A\sqrt{H} \text{ (approx.)}$$

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.)

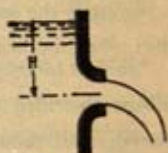
Square and rectangular orifices have slightly higher discharges than circular orifices, by about 1 to  $1\frac{1}{2}$  per cent.

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  $\frac{1}{3}$ rd 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.

$$Q = 0.97 A \sqrt{2gH}$$



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*.

\*A bell-mouthed orifice is made of the contour of the free jet issuing.



Values of Co-efficient  $c$  for various conditions of Orifices :

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 $5^\circ$ mouthpiece ... ..	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 $2\frac{1}{2}$ 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 $2\frac{1}{2}$ diameters (tube projecting inside the wall)... ..	0.73
Ditto. when the length is about 1 diameter—it is called "Borda's Mouthpiece" ... ..	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 $13^\circ-21'$ ) ... ..	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 follows :—

L	1	2	3	5	10	15	25	50
C	.62	.82	.815	.79	.77	.74	.71	.64
C <sub>1</sub>	.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 <sub>1</sub>	.47	.43	.36	.32	.29	.27		.155

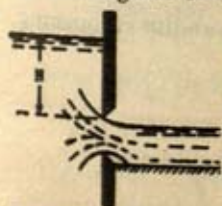
L is length in diameters ; C is co-efficient of discharge for new pipes and C<sub>1</sub> for old pipes.

### 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 discharging freely into the air. For small size sharp edged orifices and head up to 4 ft. the approximate value of c is 0.60. Average value of  $c=0.62$  for rectangular orifices.

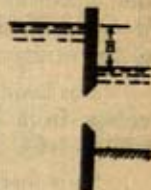


For *Partially Submerged* small orifices the same formula applies and c may also be taken the same without much error. Some engineers take H to the centre of the orifice. There are also more accurate formulae but results do not vary much.

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 6 and 13 ft. wide,  $c=0.72$ ;  
 For regulator openings above 13 ft. wide,  $c=0.82$  ;  
 For very large sluices and bridge openings,  $c=0.92$ .

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  $0.95$ .

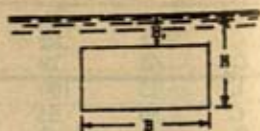


**Large Vertical Orifices with Small Heads :**

(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 0.632 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 2 ft. 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) Triangular opening :

(a) Base up :

$$Q = \frac{2}{3} cB\sqrt{2g} \left( \frac{2}{5} \frac{H^{5/2} - H_1^{5/2}}{H - H_1} - H_1^{3/2} \right)$$

(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 :

$Q_1$ —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)

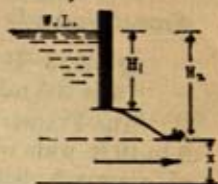
(ii) *Discharge of water into water :*

$$Q_2 = c_1 B \times \sqrt{(2gH_2)}$$

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	
	$c_1$	$c_1\sqrt{2g}$
Sluices without side walls ... ..	0.66	5.30
Canal lock and dock gates ... ..	0.70	5.62
Sluices in lock gates ... ..	0.83	6.66
Narrow bridge openings ... ..	0.90	7.22
Wide bridge openings or very large well built sluices with side walls ... ..	0.94	7.54
Wide opening the bed of which is level with the bottom of the reservoir ... ..	0.96	7.70

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 :

$$Q = c_1 \times B \left( (H_2 + x) - H_1 \right) \times \sqrt{(2gH_2)}$$

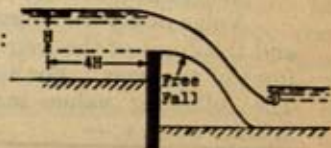
#### 4. WEIRS

The weirs are of two classes :

Sharp-crested and

Broadcrested.

*Sharp-Crested Weirs :*



*Bazin's formula* for discharge over Rectangular weirs :  
clear overfall ; without end contractions :

$$Q = \frac{2}{3} cB\sqrt{2g} H^{3/2} = 5.35cB H^{3/2} = 3.33BH^{3/2} (\text{approx.})$$

taking  $c=0.62$ .

Discharge in c. ft. per minute =  $195 B H^{3/2}$ .

*Francis formula :*

$$Q = 3.33 (B - 0.1 n H) H^{3/2}$$

$n$  is the number of end contractions (2, 1 or 0).

The Francis formulae apply accurately to weirs from 8 to 10 ft. wide with heads of from 0.6 to 1.6 ft. and velocity of approach between 0.2 and 1.0 ft. per sec. Weir lip should be between 2 ft. and 5 ft. above bottom of tank.

If end contractions are perfect, it causes at each end a shortening of the effective "breadth" by approximately  $0.1 \times H$  and if allowance is to be made for it, the Bazin's formula (for two end contractions) becomes :

$$Q = \frac{2}{3} c (B - 0.2H) \sqrt{2g} H^{3/2} = 3.32 (B - 0.2H) H^{3/2}$$

When the length of a weir  $B$  is great relative to  $H$ , end contraction does not matter.

The above equations assume no *velocity of approach*, this velocity (which tends to increase the discharge) can be taken account of by supposing the head which would be required to produce it, to be added to the actual head, viz., by adding  $V^2/2g$  to the head  $H$ , in feet. This is not generally considered the difference being negligible, the discharge is calculated with  $H$ . Where, however, the velocity of approach has to be taken into account the formula will become :

$$Q = \frac{2}{3} c B \sqrt{2g} \left\{ (H+h)^{3/2} - h^{3/2} \right\}$$

Where  $h$  is the head due to velocity of approach.

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 0.60 to 0.66. The following values may be taken in the absence of any definite figures :—

Description of weir	Value of	
	$c$	$\frac{1}{2} \sqrt{2g}$
Broad-crested or flat-topped ... ..	0.577	3.09
With narrow crests (less than 3 ft.) ...	0.623	3.33
Weir overfalls where breadth ( $B$ ) = full width of the channel (end contractions suppressed)	0.666	3.56

$c$  decreases with increase in width and increases with increase in depth up to 4 ft. beyond which it is constant, and varies from 0.62 to 0.68.

A weir for measuring discharge should have a well defined form and a fairly level crest of permanent shape and height. The point where head is measured should be at least  $4H$  from the weir crest.

#### **Water-fall from a weir :**

$$x = \frac{4}{3} c \sqrt{H} \cdot y$$

where :

$$y = D + \frac{4}{g} H$$

$x$  = 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.)

$$Q = 3.367 B H^{3/2} \text{ (sharp crested)}$$

$B$  is width of the weir at bottom ;  $H$  is height of water measured at a distance  $4 \times H$  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  $\frac{1}{4}$ rd the length of the sill. The velocity of approach must be practically nil and it should have free overfall.



**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).

$$X = H + \sqrt[3]{H \times \sqrt{D}}$$

$$Y = \frac{4}{3} \sqrt{Z} \times \sqrt{H}$$

where :

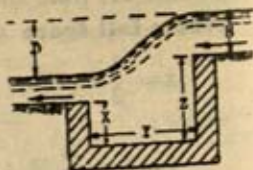
X = depth of water cushion ;

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, in galls. per hour, without end contractions :

H. ins.	0	$\frac{1}{8}$	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$
0	0	75	225	410	640	900	1,200	1,450
1	1,720	2,150	2,500	2,900	3,300	3,750	4,200	4,650
2	4,780	5,650	6,050	6,600	7,100	7,650	8,200	8,800
3	9,350	10,300	10,550	11,150	11,800	12,400	13,000	13,800
4	14,150	15,100	15,800	16,450	17,200	17,900	18,600	19,400
5	20,350	20,850	21,600	22,450	23,200	24,000	24,800	25,650
6	26,450	27,200	28,100	28,950	29,950	30,700	31,400	32,400
7	33,000	33,900	35,100	36,000	37,000	37,900	38,850	39,600
8	41,000	41,700	42,550	43,550	44,600	45,600	46,500	47,700
9	48,500	49,500	50,600	51,600	52,800	53,900	54,600	55,900
10	56,500	58,000	59,000	60,000	61,200	62,400	63,450	64,500
11	65,000	66,800	67,900	68,800	70,200	71,300	72,400	74,700
12	74,800	76,000	77,400	78,300	79,500	80,400	81,600	83,000

If the depth of water is measured at the notch instead of 3 ft. upstream, add  $\frac{1}{8}$  in. to depth if velocity over notch is 2 ft. per sec., or  $\frac{3}{8}$  in. to depth if velocity is 3 ft per sec. This is for rough calculations.

Approximate Discharge of water over a Rectangular Weir one foot wide, without end contractions :

$$Q = 3.33 BH^{3/2}$$

H in ft.	Dis. cus.	Dis. g.p.m.	H in ft.	Dis. Cus.	Dis. g.p.m.
1.00	3.33	1248	2.00	9.41	3520
1.10	3.84	1440	2.25	11.15	4170
1.20	4.37	1640	2.50	13.16	4920
1.30	4.92	1845	2.75	15.18	5677
1.40	5.50	2060	3.00	17.30	6470
1.50	6.10	2283	3.50	21.80	8153
1.60	6.71	2510	4.00	26.64	9963
1.70	7.37	2760	4.50	31.79	11890
1.80	8.03	3010	5.00	37.23	13924
1.90	8.70	3260	5.50	42.95	16063

Discharge over a Cipolletti Weir per foot of Bottom width :

H in ft.	Dis. cusecs per ft. of weir	H in ft.	Dis. cusecs per ft. of weir	H in ft.	Dis. cusecs per ft. of weir
0.1	0.107	1.1	3.884	2.1	10.245
0.2	0.301	1.2	4.426	2.2	10.986
0.3	0.553	1.3	4.990	2.3	11.743
0.4	0.852	1.4	5.577	2.4	12.517
0.5	1.190	1.5	6.185	2.5	13.308
0.6	1.565	1.6	6.814	2.6	14.114
0.7	1.972	1.7	7.462	2.7	14.936
0.8	2.409	1.8	8.130	2.8	15.774
0.9	2.875	1.9	8.817	2.9	16.626
1.0	3.367	2.0	9.522	3.0	17.494

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 submer- sion	Multi- plier	Per cent submer- sion	Multi- plier	Per cent submer- sion	Multi- plier
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	0.884	70	0.799

**Discharge over a Submerged Weir :**

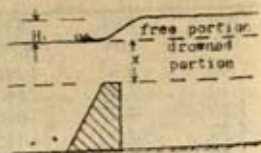
This may be divided into two portions— $Q_1$  and  $Q_2$ .

(i) *Discharge through free portion :*

$$Q_1 = \frac{2}{3} cB\sqrt{2g} H_1^{3/2}$$

$$= 5.35 cBH_1^{3/2}$$

$$c = 0.577$$



(ii) *Discharge through drowned portion :*

$$Q_2 = c_1 Bx\sqrt{(2gH_1)} \quad c = 0.80$$

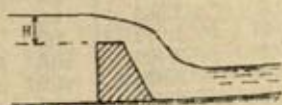
(This is without velocity of approach.) The drowned portion  $x$  on the crest is called "tail water".

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 :**

$$H = \sqrt[3]{\frac{Q^2}{11B^2}} \quad (\text{approx.})$$

$B$  = breadth of weir in ft.

**Discharge over Broad-crested Weir :**

$$H = h + \frac{V^2}{2g}$$

Simple approximate expression :

$$Q = 0.35B\sqrt{2g} H^{3/2}$$

Max. discharge occurs when  $h = \frac{2}{3} H$ . In that case

$$Q = 3.09 BH^{3/2}$$

The following formula may be used for Broad-crested weirs with crest width exceeding 2 ft. and heads from 6 inches to twice the breadth of the crest.

$$Q = 2.64 BH^{3/2}$$





Flat crests decrease the discharge until the head becomes so high (between 1.5 to 2 times the width of the crest) 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 :

$$Q = c_1 B H^{3/2}$$

$c_1 = 3$  to 4.5 depending  
upon shape of crest.



The effect of side contraction 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 4 ins. and the breadth of crest 6.56 ft.

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  $45^\circ$ . If inclination is in the opposite direction, the contraction is increased and the discharge decreased. With a  $45^\circ$  slope, the decrease may be as much as 7 per cent. Inclining the downstream face does not materially alter the discharge.

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 :

$$Q = 4.2 B H^{3/2}$$

(Due to this shape the discharging capacity is more by nearly 50 per cent than that of a rectangular broad-crested weir).



**Siphon Spill-way :**

$$Q = cA \sqrt{2gH}$$

$$\begin{aligned} c &= 0.75 \text{ for a simple type,} \\ &= 0.65 \text{ per hood type,} \\ &= 0.80 \text{ for volute type.} \end{aligned}$$

A = the total area of all the siphon spill-way opening.,

H = the difference in levels of water upstream and down-stream of the spill-way, or operating head.

**5. FLOW FORMULAE FOR OPEN CHANNELS, DRAINS & PIPES****(i) Chezy's Bazin's, and Kutter's formulae****General Equation :**

$$V = C \sqrt{RS}$$

V = velocity in ft. per sec.,

C = co-efficient of roughness,

R = hydraulic mean radius or hydraulic mean depth,  
= cross-sectional area of the liquid divided by wetted perimeter =  $d/4$  for circular pipes.

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 125 for glazed surfaces ; 100 for cemented or smooth finished concrete ; 85 for brick drains.

$$\text{Kutter's } C = \frac{41.6 + \frac{0.0028}{s} + \frac{1.811}{n}}{1 + \left( 41.6 + \frac{0.0028}{s} \right) \frac{n}{\sqrt{R}}} \quad \text{(in British system of units)}$$

n is the co-efficient of roughness.

The following values of co-efficient "n" are generally used for Kutter's or Manning's formulae :—

Surface	Condition			
	Best	Good	Fair	Bad
Uncoated cast-iron pipes	0.012	0.013	0.014	0.015
Coated cast-iron pipes ..	0.011	0.012*	0.013*	0.015
Wrought iron pipes, black	0.012	0.013	0.014	0.015
Ditto, galvanized .. ..	0.013	0.014	0.015	0.017
Riveted and spiral steel pipes	0.013	0.015	0.017	..
Vitrified sewer pipes ..	{ 0.01 } 0.01	0.013*	0.015*	0.017
Common clay drainage tiles	0.011	0.012	0.014	* 0.017
Glazed brickwork	0.011	0.012	0.013*	0.015
Brick sewers	0.012	0.013	0.015*	0.017
Neat cement surface ..	0.010	0.011	0.012	0.013
Cement mortar surface	0.011	0.012	0.013*	0.015
Concrete pipes	0.012	0.013	0.015	0.016
Concrete-lined channels ..	0.012	0.014*	0.016*	0.018
Cement-rubble surface	0.017	0.020	0.025	0.030
Dry rubble surface ..	0.025	0.030	0.033	0.035
Dressed ashlar surface ..	0.013	0.014	0.015	0.017

\*Values commonly used in designing.

#### General Values

- 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 unplanned timber ; ordinary iron pipes.
- 0.013 for well laid brickwork and ashlar ; cast iron pipes asphalted or coated, with usual bends and valves, etc.
- 0.015 for rough brickwork ; good stonework in fair order ; cast iron ; ordinary concrete.
- 0.017 for brickwork and stone in inferior condition.
- 0.020 for rubble masonry ; coarse brickwork ; earth in good order ; very fine gravel ; rough concrete ; smooth rubble, (smooth 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.

(Also see under "Irrigation")

$$\text{Bazin's } C = \frac{157.6}{1 + \frac{k}{\sqrt{R}}}$$

The value of  $k$  varies with the nature of the channel as follows :

0.109 for glazed, very smooth cement plastered surfaces.

0.29 for smooth—planks, ashlar, brickwork, concrete.

0.35 for smooth concrete surfaces and brickwork in cement.

0.83 for rough—rubble masonry, brickwork in inferior condition, gravel well rammed.

1.00 for rubble masonry and concrete surfaces not very smooth and rough brickwork.

1.54 for rougher-earth channels, or dressed surface pitched in whole or part with stone, very regular surfaces.

2.36 for very rough—ordinary earth channels.

3.17 for excessively rough—canals encumbered with weeds and boulders.

Value of  $C$  for drains and channels is given in Section 16.

Kutter's formula was derived for flow in open channels but has been used with a fair degree of accuracy for pipes and conduits also. It is only approximately correct for small circular pipes with a constant value of  $n$ .

A simpler expression for  $C$  than Kutter's is :

$$C = \frac{1.486R^{1/6}}{n}$$

It is considered that Bazin's formula gives the most accurate results, except in the case of very large channels,

when the Kutter's formula has possibly the greater accuracy. 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  $V = \sqrt{RS}$  for Clean Asphalted Cast Iron Pipes :

Dia. of pipe ins.	3	6	9	12	15	18	21	24	30	36	42	48
Value of $C$	70	86	97	105	112	117	120	123	127	131	135	138

This is a compromise value of all the empirical formulae, given in the Proc. Inst. C.E. Vol. CLIII page 297.

For very rough calculations the value of  $C$  may be taken as 100.

The discharge is calculated by multiplying the velocity by the cross-sectional area of the pipe.

(ii) **Manning's formula :**

$$V = \frac{1.486}{n} R^{\frac{2}{3}} S^{\frac{1}{2}} = \frac{0.590}{n} d^{\frac{2}{3}} S^{\frac{1}{2}}$$

" $n$ " has the same value as in Kutter's formula : may be taken 140 for glazed surfaces ; 124 for cemented or smooth finished concrete, or brickwork, or cast iron ; and 114 for ordinary good brick drains.

(iii) **Crimp and Bruges' formula for flow in brick Sewers :**

$$V = 124 R^{\frac{2}{3}} S^{\frac{1}{2}} \text{ or } 124 R^{0.67} S^{0.50}$$

$$Q = \frac{3.072D}{\sqrt{L}}$$

$Q$  = discharge in c. ft. per minute ;  $D$  = dia. in inches ;  
 $L$  = length divided by fall.

This formula holds for all good ordinary pipes and brick sewers up to 36 inches diameter, but the co-efficient 124 may have to be reduced as much as 25 per cent for old sewers and possibly increased by the same amount for large new brick culverts.

(iv) *Neville's formula for flow in open Channels and Pipes :*

$$V = 140\sqrt{RS} - 11\sqrt[3]{RS}$$

This is for new or clean cast iron pipes, coated or asphalted, laid under the ordinary conditions of water mains with valves and bends, etc.

(v) *Eytelwein's formula for Sewage Flow :*

$$V = 55\sqrt{(R \times 2F)} = 94.2\sqrt{RS}$$

F = fall in feet per mile ; V = velocity in feet per minute.

(vi) Barnes' formula or flow in "slimy sewers" of all materials :

$$V = 107 R^{0.7} S^{0.5}$$

(vii) **Hazen and Williams' Formula** for flow in Pipes and Channels :

$$V = 1.318C R^{0.63} S^{0.54} = 0.115C d^{0.63} S^{0.54}$$

Discharge in galls. per hour =  $14.06C d^{2.63} S^{0.54}$ .

This formula is not suitable for pipes when the coefficient C is appreciably below 100.

The flow chart in "Water Supply" has been worked out with C=100. For other values of C, multiply by the following co-efficients :—

C	L <sub>1</sub>	L <sub>2</sub>	Explanation
40	5.46	0.40	(a) To determine the loss of head with value of "C" other than 100 multiply the loss of head found from the chart by L <sub>1</sub> .
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	(b) To determine the quantity of flow with value of "C" other than 100 multiply the quantity of flow found from the chart by L <sub>2</sub> .
120	0.71	1.20	
130	0.62	1.30	
140	0.54	1.40	



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
Pipes and conduits 3" to 60" dia.		<b>Cement Lined Pipes</b>	
<b>Cast Iron</b>		Applied by hand	125
Very best new	140	Centrifugally applied	140
New well laid	130	<b>Wrought Iron</b> (1½" to ½")	
4 to 6 years old	120	Very smooth and straight	140
10 to 12 years old	110	Smooth and new	120-135
13 to 20 years old	100	Ordinary iron	100
26 to 35 years old (4"-10")	80	Old Iron	80
37 to 47 years old (12"-60")	80	Very rough	60
Very rough	60	<b>Masonry</b>	
Tuberculated	80-40	Very smooth	130
<b>Riveted Steel Pipes</b>		Good masonry	120
New	110	Brick sewers	100
10 years old	100	ditto-rough	90
<b>2", 2½" &amp; 3" Pipes</b>		ditto-very rough	80
Very smooth, (brass, tin, etc.)	140	Tile sewers (4" to 36")	110
Ordinary, ditto.	130	<b>Open Channels</b>	
Smooth new Iron	120	Good masonry	80-120
Ordinary iron	100	Rough ditto	65- 75
<b>Asbestos Cement</b>	145	Gravel	50- 80
		Earth very rough	65- 75
		ditto. with grass & weeds	35- 60

Co-efficient 130 in the above formula corresponds to co-efficient (n) of 0.011 in Kutter'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

(Also see under "Irrigation")

Max. velocity and discharge in a rectangular channel occurs when the depth of water is half the breadth.

Cross section of a Trapezoidal channel to have max. flow :

R to be	$=0.5 d$	A=area of flow,
A will be	$=1.828 d^2$	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  $=.81 \times$  diameter of pipe.

Depth for max. discharge  $=.95 \times$  diameter of pipe.

When depth is .25 d, 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 are :—

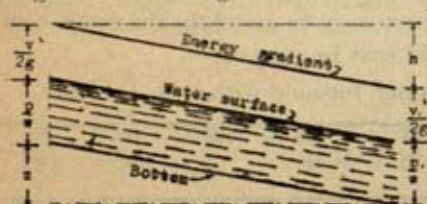
Radius of crown	...	$=R$
Radius of invert	...	$=R/2$
Radius of sides and total depth	...	$=3R$

## 7. FLOW THROUGH PIPES

### Bernoulli's Theorem :

It is a proposition advanced by Daniel Bernoulli that 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.

$$\frac{V^2}{2g} + \frac{p}{w} + z = \frac{V_1^2}{2g} + \frac{p_1}{w} + h = \text{Total energy of water per lb.}$$



$z$  = certain height above certain data,  
 $p$  = pressure in lbs/sq. ft.,  $w$  = wt. of water in c. ft.,  $h$  = friction head lost in ft.

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 :—

$$(i) \quad h = \frac{4fLV^2}{d \cdot 2g}$$

$h$  = loss of head in ft.,  $L$  is length of pipe line in ft. ;  
 $d$  = dia. of pipe in ft. ;  $V$  is velocity in ft. per second ;  $g$  is  
 acceleration due to gravity = 32.2 ft./sec.<sup>2</sup>.

$$f = 0.005 \left( 1 + \frac{1}{12d} \right) \text{ for new iron pipes}$$

$$= 0.01 \left( 1 + \frac{1}{12d} \right) \text{ for old iron pipes}$$

$$(ii) \quad h = \frac{fLV^2}{d}$$

$f = 0.000324$  for new cast iron pipes,  
 $= 0.000988$  for old cast iron pipes.

(Loss of head in old pipes is three times that in new pipes.)

$V = \left( \frac{dh}{fL} \right)^{\frac{1}{2}}$ , multiplied by section of pipe  $\pi d^2/4$  gives

discharge  $Q$  in cusecs =  $f' d^{\frac{5}{2}} \left( \frac{1}{L} \right)^{\frac{1}{2}}$

Where :  $f' = 44$  for new cast iron pipes, 25 for old cast iron pipes (mean 35), 53 for new asphalt pipes, 44 for old asphalt pipes, 37 for riveted pipes, and 43 for cement lined pipes.

(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 :

$$h = \left( 1 - \frac{A_1}{A_2} \right) \frac{V_1^2}{2g} \quad \text{or} \quad \frac{(V_1 - V_2)^2}{2g}$$

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 :

$$h = \frac{c V_2^2}{2g} \quad c \text{ varies with the ratio } \frac{A_2}{A_1}$$

Values of  $c$  (Weisbach)

$A_2/A_1$	0.10	0.20	0.30	0.40	0.50	0.60	0.70	0.80	0.90
$c$	0.362	0.338	0.308	0.276	0.221	0.164	0.105	0.053	0.015

For practical purposes this loss is taken =  $\frac{1}{2} \times \frac{V_2^2}{2g}$  and

this may be taken as the immediate loss of head experienced as water flows from a reservoir into a pipe in which it attains a velocity  $V_2$ .

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).

$$h = \left\{ \frac{A}{c(A_1 - a)} \right\}^2 \times \frac{V_1^2}{2g} \quad c \text{ is a co-efficient of contraction, may be taken} = 0.66.$$

(f) Due to bends and elbows :

$$h = \frac{k V_1^2}{2g} \quad k \text{ is a co-efficient.}$$

(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,  $V_2$  = final velocity, $a$  = cross-sectional area of the obstruction.

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 :

$$V = c \sqrt{(2gH)} = \text{Dis. / section of pipe}$$

The Pitot tube is often the most convenient means of measuring flow in pipe lines, especially those of large diameters, and in other forms of conduits and channels. In making gaugings in a pipe, it is usual to measure the centre velocity and to calculate the discharge by multiplying this velocity by a co-efficient. The best form of Pitot tube is the Pitometer and has two orifices which are set to point upstream and downstream. These two orifices are symmetrical and can be reversed to check accuracy. (Also see under "Irrigation")

#### Functions of Flow in a Circular Pipe Running Partly Full

Depth of flow	Area ( $\times d^2$ )	Proportional		Depth of flow	Area ( $\times d^2$ )	Proportional	
		V	Q			V	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	0.843	0.263	0.85	0.7115	1.137	1.030
0.40	0.2934	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.

**Hydraulic Gradient** is the line joining the pressure head of water in the beginning with the pressure head at the end and indicates loss of head due to friction in pipes. In other words, it is a line drawn through a series of points to which water would rise in piezometer tubes attached to

a pipe through which water flows. With a straight smooth pipe of uniform cross section the *Hydraulic Grade line* is a straight line extending from the reservoir water level to the end of the pipe ; the pressure for any point in the pipe being equivalent to the head represented by vertical distance from that point to the Hydraulic Grade line. 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. It is also called *Virtual Slope*. (Also see under "Irrigation.").

### Discharge through Siphons (Aqueducts) :

$$H = \left( 1 + f_1 + f_2 \cdot \frac{L}{R} \right) \frac{V^2}{2g}$$

where :

$H$  = fall or loss of head through siphon in ft. ; is the difference of water levels upstream and downstream,

$L$  = length of barrel and entrance in ft.,

$R$  = hydraulic mean radius of the barrel in ft.,

$V$  = mean velocity through the barrel in ft./sec.,

$f_1$  = co-efficient which provide for the loss of head on entry ; 0.08 for bell mounted siphons, 0.505 for cylindrical siphons with unshaped mouth of the same sectional area,

$f_2$  = co-efficient which provides for the loss of head due to friction in the barrel and  $= a(1 - b/R)$ . The values of "a" and "b" are taken as follows :

Type of Inner Surface of Barrel				a	b
Smooth iron pipe	...	...	...	0.00497	0.084
Encrusted iron pipe	...	...	...	0.00996	0.084
Smooth cement plaster	...	...	...	0.00316	0.100
Ashlar or brickwork	...	...	...	0.00401	0.230
Rubble masonry or stone	...	...	...	0.00507	0.830

Lacey's formula

$$H = \left( 1.08 + 0.008LR^{-1.05} \right) \frac{V^2}{2g}$$

Generally the velocity of approach is neglected.



**Time for Emptying Tanks :** (Also see "Water Supply.")

$$T = \frac{2A}{ca\sqrt{2g}} = \left\{ H_1^{\frac{1}{2}} - H_2^{\frac{1}{2}} \right\}$$

$T$  = time in seconds,  
 $H_1$  = head at beginning, ft.  
 $H_2$  = head at end, ft.  
 $A$  = cross sectional area of tank in sq. ins.,  
 $a$  = area of orifice, in sq. ins.,  
 $c$  = co-efficient as before  
 = .62 for steel tanks,  
 thin plates ; .92 for bell-mouth.

If the tank is completely emptied  $H_2$  will be = 0.

*In case of Hemispherical Vessels*

$$T = \frac{2\pi}{ca\sqrt{2g}} \left[ \frac{2}{3}R \left( H_1^{\frac{5}{2}} - H_2^{\frac{5}{2}} \right) - \frac{1}{5} \left( H_1^{\frac{5}{2}} - H_2^{\frac{5}{2}} \right) \right]$$

$R$  = radius of the vessel.

(Time is double that which would be required to discharge the same volume under a constant head  $H_1$ ).

When  $H_2 = 0$ , (i.e., completely emptied)

$$T \text{ will be } = \frac{14\pi R^{\frac{5}{2}}}{15 ca \sqrt{2g}}$$

**Time of flow from one vessel into another :**

$$T = \frac{2A_1 \left( H_1^{\frac{1}{2}} - H_2^{\frac{1}{2}} \right)}{ca \left( 1 + \frac{A_1}{A_2} \right) \sqrt{2g}}$$

$A_1$  = area of the vessel from which water flows to the other vessel of area  $A_2$ ,

$a$  = area of orifice.

If both the vessels have the same area :

$H_1$  = difference of head between the two vessels at the beginning,

$H_2$  = difference at the end.

$$T = \frac{A_1 \left( H_1^{\frac{1}{2}} - H_2^{\frac{1}{2}} \right)}{ca\sqrt{2g}}$$

**Time of emptying and filling a Canal Lock :**

$$T = \frac{2A\sqrt{H}}{ca\sqrt{2g}}$$

$H$  = 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 :

$$T = \frac{2A}{m\sqrt{2g}} \left( \frac{1}{\sqrt{H_2}} - \frac{1}{\sqrt{H_1}} \right)$$

$A$  = area of reservoir in plan,  
 $B$  = width of weir,  
 $H_1$  &  $H_2$  = heights above the level of the sill.  
 (water falling from height  $H_1$  to height  $H_2$ )  
 $m$  is co-efficient and  $H$  is mean of  $H_1$  and  $H_2$ .

**Nozzles :** (Also see "Water Supply")

Ratio of area of nozzle to area of supply pipe for max. transmission of power :

$$\frac{A}{a} = \sqrt{\frac{8fl}{A'}}$$

If the jet were projected vertically upwards the height the water would reach

$$\text{or } \frac{8fl}{A'} = \frac{v^2}{V^2}$$

$$\text{or } v = V \left( \frac{d_1}{d} \right)^2$$

$$= \frac{v^2}{2g} = h = \frac{H}{1 + c \frac{4L}{d_1} \left( \frac{d}{d_1} \right)^4}$$

Allowing for air resistance, the actual height of jet would be  $h(1 - 0.003 h^2)$ .

where :

$H$  = pressure head at base of nozzle,

$c = 0.005$  for new smooth pipes,

0.01 for old encrusted pipes,

0.0026 for a varnished surface,

$d$  = dia. of nozzle end,  $d_1$  = dia. of pipe,  $L$  = length of pipe.

Rate of flow thro. nozzle :

$$Q = KA\sqrt{2g(H-h)}$$

Deduct loss due to friction in pipe viz.,

$$= \frac{4fLv^2}{d2g} = h$$

$A$  = area of the cross section marked  $A$ ,

$a$  = ditto. marked  $a'$ ,

$K$  = co-efficient which may be taken from 0.95 to 0.99 for smooth conical nozzle (pipe adjutage 13°-24').

$L$  = length of nozzle pipe.

**Rate of Discharge from Fire Hydrants :**

(Also see "Water Supply")

$$Q = 24.5 d^2 \sqrt{p}$$

Q=discharge in galls./minute,  
 d=internal dia. of hydrant  
 nozzle in inches,  
 p=pressure in lbs./sq. in.

**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".)

**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. *Kinetic* 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.

**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.

**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** deals 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.

**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.

**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.

**Piezometer** : An instrument for measuring pressure 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 for observing the velocity head of flowing water.

**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.

**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 about one-half the dia. of the opening from the plane of the orifice.

**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. Piezo-meter 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.

**Venturi Meter :** A proprietary measuring device consisting of a venturi tube and a flow registering device.

**Venturi Flume :** A type of open flume with a contracted throat in which the velocity in the throat is less than the critical velocity, used for measuring flow.

**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".)

H	$\sqrt{H}$	H	$\sqrt{H}$	H	$\sqrt{H}$	H	$\sqrt{H}$
0.1	0.32	1.1	1.05	2.1	1.45	3.1	1.76
0.2	0.45	1.2	1.10	2.2	1.49	3.2	1.79
0.3	0.55	1.3	1.14	2.3	1.52	3.3	1.82
0.4	0.63	1.4	1.18	2.4	1.55	3.4	1.84
0.5	0.71	1.5	1.23	2.5	1.58	3.5	1.87
0.6	0.77	1.6	1.26	2.6	1.61	3.6	1.90
0.7	0.84	1.7	1.30	2.7	1.64	3.7	1.92
0.8	0.89	1.8	1.34	2.8	1.67	3.8	1.95
0.9	0.95	1.9	1.38	2.9	1.70	3.9	1.97
1.0	1.00	2.0	1.41	3.0	1.73	4.0	2.00



## SECTION 15

### **WATER SUPPLY** (Purification, Distribution & Pumping)

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## 1. DRINKING WATER QUALITIES

**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.

Frequently, water is treated not only to improve its quality from aesthetic and sanitary point of view, but to reduce its corrosive or scaling qualities which would affect the life and carrying capacity of the 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 sterilized.

A water for drinking should be clear and sparkling, colourless, odourless, pleasant to the taste and free from harmful bacteria. Standards of purity and quality of drinking waters are laid down by various Public Health authorities. The suitability of a water for any particular purpose can be determined only after a complete chemical and bacteriological examination, and an investigation of the source of origin of supply. Interpretation of results and standards of purity vary with the nature and source of supply and a laboratory examination of a single sample of water does not give a conclusive result.



A *potable* water is one that is safe to drink, pleasant to the 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 undesirable or unfit for drinking or domestic use.

#### **Simple Tests for Detecting Dangerous Matters in Water**

*Presence of Sewage* : Four drops of a solution of permanganate of potash (Condy's Fluid) to a small glass of water. The colour turns pale or yellow if decomposed organic matter is present.

*Presence of Lead* : Six drops of sulphuric acid in a small glass of water. White precipitate will be formed if lead is present.

*Presence of Zinc* : Six drops of ferro-cyanide of potassium to a small glass of water will turn it green if zinc is present.

*Presence of Copper* : Eight drops of ammonia in a small glass of water will turn it blue if copper is present.

#### **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 sun's rays and cause turbidity and colour (brown or green) and give taste (acidity) to the water. They often grow so densely that they clog filters. Algae growth in small reservoirs can be controlled by roofing the reservoirs in order to exclude sunlight. Taste, colour, and odour can be removed by aeration and activated carbon treatment. In large areas, copper sulphate or chlorine is used to kill algae. A dose of 5 lbs. per million gallons of copper sulphate should first be given, if not successful, after 2 or 3 days the dose may be increased with a maximum of up to 20 lbs. per million gallons which is enough to kill all plant life. A dose of about 1 to 1½ lbs./m.g. and above kills fish. Chlorine is useful for flowing water: dose is from 0.2 p.p.m. to 1.5 p.p.m. which will kill fish as well. Treatment is given prior to filtration. (This has also been dealt with in Section 17 under "Weed Growth").

*Fungi* are plants that grow in absence of sunlight in water mains. They can be controlled by treatment of the water with chlorine or copper sulphate.

**Iron** is often found in all ground waters in very minute traces which is dissolved from the earth's crust. Iron in water may also result from corrosion of the cast iron water mains. Iron in very small amounts (about 0.3 p.p.m. and above) will cause unpleasant taste and reddening of the water which may cause discolouration of clothes washed and incrustation in water mains and plumbing fixtures. Iron also stains white glazed sanitary wares. Presence of iron encourages formation of incrustations in water mains which decrease their capacity. Certain natural waters containing iron are detrimental to those with high blood pressure. Iron can be removed by "aeration" of the water and also with treatment through pressure filters and lime softening process.

Iron is often associated with manganese, found in ground waters, which has almost the same effects as iron and produces darker stains.

**Lead** is a cumulative poison and regular consumption of water with over about 0.3 to 0.5 p.p.m. of lead by weight may result in lead poisoning. Under 0.1 p.p.m. is considered safe. The waters likely to take up lead are soft or acidic, rain-waters and swamp or moorland waters. Lead is also dissolved from lead service pipes and storage tanks painted with lead paints. See under "Lead Pipes."

**Goiter** is caused by the deficiency of iodine in water, particularly from certain underground supplies derived from limestone regions; some amount of iodine in water is essential and is added artificially where goiter is endemic.

**Mottled Enamel.** Waters containing fluorides in amounts greater than about 1.5 p.p.m. cause mottling or staining of teeth of growing children, but an amount less than 0.5 to 1.0 p.p.m. is detrimental to good teeth development.

**Turbidity** in water is due to the presence of finally divided particles of clay, silt, and organic matter (living



or dead plants or other organisms). Turbidity is removed by sedimentation, with or without coagulation, and filtration.

**Colour** in water is due to colloidal organic matter and dissolved material resulting from decomposed vegetation, iron, manganese or other mineral matter. A yellowish looking water is not necessarily impure neither a clear bright sparkling water necessarily pure.

**Odour and Taste** are chiefly due to decomposing organic matter (decaying plants), mineral salts, iron oxide and manganese.

**Colour and Taste Removal.** Colour is removed to a great extent by filtration preceded by coagulation with alum to encourage sedimentation. Filtration through rapid-sand filters is more effective as regards colour removal than through slow-sand filters. Excess chlorine treatment also removes colour.

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 p.p.m. Water is also passed through the charcoal used as a filter as described later under "Activated Charcoal."

**Alkalinity or Acidity.** Alkalinity is due to the presence of mineral salts in water which cause hardness (carbonates and bicarbonates of calcium, sodium and magnesium). Acidity is caused by the decomposition of vegetable matter. Natural waters are seldom neutral and most waters are more or less alkaline because the alkaline salts are very common in the ground. All salts will cause tastes if present in sufficient amounts. Saline waters with salinity more than 15 parts in  $10^5$  are unfit for drinking and with salinity more than 60 parts in  $10^5$  are unfit for irrigation.

Acidity or alkalinity of water or other liquids is mea-



sured in "pH" value. The pH scale runs from 0 to 14 with 7 as the point of absolute neutrality; distilled water which is neutral 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. 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 pH value of 4. Most natural fresh waters have a pH value between 5.5 and 8.0. Some moorland waters may, however, have a value from 3.0 to 5.5. The value for sea water is usually between 8.0 and 8.3. The test is made by comparing the colour of the water, after phenolphthalein or other reagent has been added, with a standard chart.

pH value should be as close to 7.0 as economically feasible. Waters with pH between 7.4 and 8.4 are practically inactive. (pH value of strong caustic soda is about 13 and that of concentrated sulphuric acid between 0 and 1). Waters below 7.0 (acid waters) are corrosive and cause tuberculation. Waters above 7.0 cause incrustation of pipes.

The determination of pH value provides information concerning the corrosive character of water. Waters with pH value of below 7 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 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 testeful, but excessively acidic or alkaline waters are unfit for drinking and should be avoided.

**Hardness.** Salts of calcium and magnesium which are dissolved in water during its passage through the ground cause hardness in water. Hard waters are generally alkaline. Carbonates cause *temporary hardness* or *carbonate hardness*, which in the presence of carbon dioxide in water form bicarbonates. The term "temporary hardness is sometimes called "alkalinity".

Temporary hardness can generally be removed by boiling the water when the insoluble carbonates settle as precipitates at the bottom. *Permanent or non-carbonate hardness* is caused by sulphates, nitrates and or chlorides of calcium and magnesium and which are not removed by boiling. These salts are very corrosive to steam boilers and deposit scales in boilers and also in distribution system. Hard-waters produce intestinal troubles and take more fuel and time to cook, there is loss of tenderness and palatability of vegetables and meat cooked in it. Each degree of hardness destroys 2.5 oz. of standard soap in each 100 gallons of water used for washing, thus resulting in waste of soap. Waters with hardness of over 100 to 150 p.p.m. when used for laundries and boilers usually require softening.

Hardness has no limit over drinking water but 5 to 6 degrees of hardness is considered best. Permissible limit of hardness for domestic use is 15 degrees with 30 degrees maximum (taken on Clark's scale) for human consumption. Water is considered to be hard for drinking when it contains more than 7 grains of dissolved mineral matter in a gallon of water.

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 100,000 parts of water and is 7/10 times the Clark's system. In other words, 1 grain of salt in 1 gallon of water is 1 degree in hardness on Clark's scale and  $1\frac{7}{10}$  degrees in hardness on Metric scale.

Moderately hard waters are more palatable than those of the softer quality as soft waters are "flat" and tasteless (can be improved by the addition of a little common salt). *Soft waters* have corrosive action on metals; dissolve lead (from lead pipes) and become dangerous to health. Pure and very soft waters corrode iron and produce tuberculation in cast iron pipes. But a permanently hard water should be rejected for the supply of a town. Water from wells, particularly deep wells, has dissolved salts, and is generally hard. Water from a reservoir, lake or river (except the one which flows through hills having soluble



metallic salts) does not contain dissolved salts and is usually soft.

#### 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 powdered 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 this 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.

**Hardness Table**

I		II	
<i>p.p.m.</i>	<i>Degree of Hardness</i>	<i>p.p.m.</i>	<i>Degree of Hardness</i>
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
130	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

1 p.p.m. = 10 lbs. per million gallons.

1 grain per gallon = 14.3 p.p.m. = 1.43 parts per 100,000

1 lb. = 7000 grains.

(p.p.m. is parts per million by weight. To change the result into grains per Imperial gallon, divide by 14.3, and to change into U.S. gallons, divide by 17.1).

Sea water contains from 3 to 4 per cent by weight of salt, i.e., about 5 oz. to the gallon. The saturation point is reached when 35 oz. to the gallon are present. Further concentrations cause salt to be deposited.



### Water Softening

Waters which are excessively hard are often softened before distribution. Water softening is usually effected either by lime, with or without the addition of soda, or by the base-exchange (zeolite) process.

The temporary hardness due to soluble bicarbonates can in large measure be removed by treatment with the necessary quantity of hydrated or slaked lime (calcium hydroxide) and subsequent sedimentation and filtration. Generally an over dose of lime is used which is later on neutralized with carbon dioxide. Lime softening is not recommended for waters of high permanent hardness. Permanent hardness due to sulphates or chlorides can be removed by caustic soda or soda-ash (sodium carbonate). Almost complete softening can therefore, be achieved, where water contains both carbonate and non-carbonate hardness, by the lime-soda process; the chemical changes that take place, the lime removes the temporary and the soda the permanent hardness. Quantity of slaked lime usually employed is 1 oz. per 100 gallons for every degree of hardness present. The amount of lime must be carefully regulated as an excess of free lime would render the water alkaline. The lime and soda may be used either in solution or dry. A continuous feed is preferred to an intermittent and a thorough agitation and sufficient contact must be provided to prevent stratification. The treated water must be allowed to remain in the tank long enough for the precipitate to settle down completely.

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 re-carbonated, generally by the application of carbon dioxide gas. In certain cases a dose of alum is added in the settling tank.

### Zeolite or Base Exchange Process

The base-exchange process is suitable for waters of any temporary or permanent hardness, and is a very convenient method, easily operated. A zeolite softener is a mechanical plant—a closed steel cylinder. The filters are either of the rapid gravity type or the pressure type, more usually the latter. Zeolite softeners remove hundred per

cent of the hardness in water, and for ground waters they often form the only treatment process. The zeolite used is an artificially prepared hydrated alumino-silicate in granular form. 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. When zeolite ceases to be effective, it should be flushed with brine solution and the calcium choride formed should be washed out.

The zeolite process is not suitable for acidic waters and waters containing iron or manganese, or turbid waters or waters with much temporary hardness for which pre-treatment is necessary. 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, it is customary to soften by 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.

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 containing much permanent hardness the addition of soda increases the cost. The lime process, with or without soda, is generally effective with waters containing iron or manganese, without pre-treatment, and the softened water is always alkaline and non-corrosive.

## 2. WATER PURIFICATION & TREATMENT

It has been stated earlier that water found in nature is seldom, if ever, absolutely 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 to be dealt with. 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 for a period of 2 to 4 weeks. In America, water is stored at some works from 6 months to several years. Stored water deteriorates biologically.

**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 1" to 3", and the fine screens with  $\frac{1}{8}$ " to  $\frac{3}{8}$ " openings ; the two types are operated in series.

**Sedimentation.** Most surface waters require a period of sedimentation so as to remove much of the suspended matter and reduce load on filters. Before undergoing any subsequent treatment and in many cases this sedimentation is assisted by chemical and mechanical flocculation. Waters somewhat heavily charged with sediment usually benefit from a preliminary sedimentation with a detention period of  $1\frac{1}{2}$  days to 3 days, unassisted by chemical or flocculating devices, followed by the main sedimentation with a detention period of  $1\frac{1}{2}$  to 4 hours. This is called *Clarification*.

In the storage and plain sedimentation process suspended impurities are removed which settle down by the action of gravity and other precipitating forces but it does not ordinarily give adequate treatment of water. The production of pure and clear palatable water requires the use of filters and sterilization.



Time required to settle through one foot depth in water for different types of impurities :—

Impurity	Size mm.	Time taken
Coarse sand .. ..	1 to 0.5 .. ..	3 seconds
Medium sand .. ..	0.05 to 0.25 .. ..	15 "
Fine sand .. ..	0.25 to 0.10 .. ..	40 "
Silt or colloidal matter	0.01 .. ..	30 minutes
Bacteria .. ..	0.001 .. ..	35 hours

### Settling Basins or Sedimentation Tanks

These are built for preliminary settlement of solids, as has been explained earlier, where the water obtained is from a canal or a river and is muddy. They are either with continuous flow or intermittent flow. Continuous flow tanks are built for a detention period of 3 to 8 hours flow for plain sedimentation, and 3 to 4 hours flow for flocculated flow based on the turbidity of the water, depth and length of the tank and the time the solids will take to deposit. Intermittent flow tanks are designed for storage of 2 to 3 days and are not now much in favour. Coagulation arrangements are added with the sedimentation tanks where there is not much of coarse material, but where most of the suspended matter is coarse and will settle rapidly without the aid of coagulants, a settling basin with detention period of 3 to 8 hours is preferable and will reduce the cost of chemicals for water treatment.

Sedimentation tanks are of various shapes but the most common forms are rectangular for the larger plants and circular for the smaller plants. Circular tanks are built in duplicate and rectangular tanks are built in two or more compartments for facility of cleaning, which form longitudinal units with long baffle walls extending up to a few inches below the water surface. The proportion of breadth to length in each compartment is generally 1 : 2, with 2 : 3 minimum and 1 : 4 maximum. Long, relatively narrow tanks are less affected by inlet and outlet disturbances and cross currents caused by breezes. Narrow tanks are more efficient since the time taken by the suspended matter to settle at the bottom is less. The velocity of flow in

the tanks is kept  $\frac{1}{2}$  to 1 ft. per minute. Depth is generally 10 to 15 ft., with 6 ft. minimum and 20 ft. maximum. Width should not exceed 40 ft. The overflow rate is 10 to 15 galls. per hour/sq. ft. for plain sedimentation and 20 to 25 galls. per hour /sq. ft. for flocculated water.

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 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 about 6" above the floor level, the draw-off being from a float below pipe so arranged that the draw-off will take place at from 15" to 18" below the surface. For outlets, some engineers prefer skimming weirs extending across the outlet end of the tank. A skimming baffle extending a few inches below the water surface near the outlet will prevent floating matter from going to filters.

To facilitate mud removal, the bottom should be sloped to one side at a grade of not less than 1 in 20, or a drain made in the centre, which runs to the outlet. Sludge or mud should be removed quite often; if the impurities are organic they are liable to decompose if allowed to accumulate for any length of time. 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**

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. By the addition of certain chemicals a flocculent



precipitate is formed which is comparatively heavy and will settle to the bottom of the liquid, and in this process of settling it will carry down with it many impurities in the water including a large proportion of bacteria. Desirable periods of retention in coagulating basins are determined by test, or by experience in operation. In practice, they seldom exceed 4 to 6 hours, and 2 hours is a frequently used period. Alum (aluminium sulphate) is the most commonly used coagulant. Coagulation is also used as a preliminary step to filtration through rapid-sand filters. For water low in suspended solids but high in colour, artificial turbidity in the form of clay is sometimes introduced to aid coagulation. Coagulant may be added to the raw water either as a dry powder or in aqueous solution, dosing in the form of solution is better. It is held in a 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. Various types of mechanical devices (Flocculators) are available in the market, which consist of tanks having paddles on horizontal or vertical shafts. The solution storage tanks are usually rubber-lined steel for small plants, and acid-resisting asphalt-lined concrete tanks for large plants.

The usual economical method is to pass the water through long channels with baffles after mixing it 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 sterilizing it.

For proper coagulation the dose of alum must be correctly determined by trial. For very clear waters it may be as low as  $\frac{1}{2}$  grain, and in the case of muddy water from a river, it may be up to 3 grains, per gallon. The normal amount used in practice varies between 1 gr. to  $1\frac{1}{2}$  grs. per gallon of water. The quantity of alum must be such as to produce visible floc. With alum water must be alkaline, if it is acidic, it must be made temporarily



hard by the addition of lime. 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.

Another coagulant is ferrous sulphate, which is used with lime; it 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.

### STERILIZATION

Sterilization of water is necessary to kill all the pathogenic 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) up to 99 per cent are removed but still some of these bacteria are very dangerous and may not be removed by simple filtration, therefore, water has to be sterilized. Sterilization of water can be done in a number of ways but the cheapest and most effective for public water supplies is chlorination. Chlorine removes bacteria, eliminates tastes and odours, improves coagulation, oxidizes iron and manganese, controls algae and other plant life and removes colour.

**Chlorination.** 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 waterworks, as chlorine gas by gaseous chlorinator. 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 will improve coagulation, reduce tastes,

odours, algae and other organisms and keep the filter sand cleaner. The dosage should be such that a residual of 0.1 to 0.5 p.p.m. goes to the filters; dose of from 5 to 10 p.p.m. are common.

The requirement of chlorine depends upon the type of bacteria and the amount present in the water, and can be determined by testing with the *orthotolidin* solution. 1 ml. of orthotolidin is dissolved in 100 ml. of water to be tested and allowed to stand for 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. After sterilization there must be residual chlorine in water 0.1 to 0.2 p.p.m. at the last tap in the distribution system after 30 minutes to ensure that all the supply has been fully disinfected. For disinfection alone a dosage of 0.1 to 0.2 p.p.m. may be required for good underground waters and 0.5 to 1.5 p.p.m. for surface waters. A dosage of about 1.0 p.p.m. will destroy most of the germs. In the case of highly polluted waters the requirement of chlorine may be as high as 3.0 p.p.m. giving a residual of 1 p.p.m. in 20 minutes. (Dosage in pounds per million gallons =  $10 \times \text{dosage p.p.m.}$ ). The bacterial efficiency of chlorine is reduced by increased pH values and low water temperatures. After chlorination the water should be allowed to stand at least 20 minutes before use. It is essential that expert advice should be taken as to which process should be employed, the dose of chlorine, and the most suitable points for the application of chlorinating and de-chlorinating agents.

The doses of chlorine used in Europe are in general much smaller than those used in America, except perhaps when "break point" chlorination is employed. Disinfection of water usually takes place in Europe only at the final stage of treatment and chlorine is not much used at any intermediate stages.

Chlorine gas is most commonly used which is supplied in steel cylinders compressed into liquid form (liquefied gas). There are a number of ingenious dosing apparatuses available for applying chlorine to water which operate



either under manual control to give a fixed dose or under automatic or semi-automatic control for varying the dose to the volume of water flow. The use of chlorine dioxide (peroxide of chlorine) produced from sodium chlorite is becoming increasingly popular in America as it has the additional property of removing odours and tastes.

Chlorine is a dangerous poison, particularly the gas, which is corrosive, dangerous to handle and breathe. 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. It is very dangerous to place any localized heat, such as an electric radiator, near the chlorine cylinder. A suitable respirator should always be available in an easily accessible position adjacent to the plant room for use in case of an accident or the necessity of shutting off the supply in the event of an escape of chlorine gas. (Based on B.S. Code of Practice CP 310.)

**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 p.p.m. are given depending on the degree of pollution, followed after a suitable interval of time by a dose of some de-chlorinating agent, *e.g.*, sulphurous acid gas (sulphur dioxide), sodium thiosulphate or potassium permanganate to neutralize the residual chlorine. 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 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.



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 re-appears as free chlorine.

The efficacy of any chlorinating process depends upon a period of contact long enough to ensure complete sterilization. If such a period cannot be provided in the normal distribution and/or storage systems between the point of treatment and the first draw-off, one or more special contact-tanks will have to be provided. Where water is supplied to the consumer direct from treatment works without the interposition of a service reservoir, the contact period after the application of chlorine may be too short.

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.

Very occasionally a small dose, 0.2 to 0.5 p.p.m., of potassium permanganate is also added to the contact tank prior to filtration in order to control tastes.

Chlorine is put into the water by a dosing apparatus regulating the quantity as desired. When the difference in pressure between water supply to chlorinator and the point of chlorine application is not large enough to operate chlorinator, it is necessary to pump chlorine under pressure. Chlorinators should be installed in duplicate so that service is not interrupted in case of a break-down, and a two weeks supply of chlorine should be at hand. The amount of free chlorine in the treated water should be determined every hour with the orthotolidin or other testing apparatus. When water is added to orthotolidin, a blue colour appears which is matched with standard glasses.

Chlorine produces unpleasant acid taste and pungent odour which vanish in course of time. If however, the taste persists, the free chlorine can be removed by adding a de-chlorinating agent as mentioned earlier, about 0.3 to 0.6 p.p.m.

*Chloramine* : Chlorine being an evaporative chemical, 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 will not only retain chlorine for much longer period but will also remove the unpleasant taste and the odour. Thus it permits a large residual chlorine content without a chlorine taste or odour. This process is called Chloramine process. Ammonia is usually added in the ratio of 1 part ammonia to 4 parts of chlorine. It is used as gas or ammonium sulphate or ammonium chloride. Ammonia is thoroughly mixed with water about half a minute before chlorine is added. Dose of ammonia is 1.5 lbs. of ammonia gas or 6 lbs. of sulphate of ammonia per million gallons of water. 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.

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.

*Bleaching Powder (Hypo-chlorite of lime)* is employed for temporary or emergency chlorination. It contains 30 to 35 per cent of active chlorine when fresh but loses its chlorine content with age and by exposure to air. The bleaching powder is dissolved in water and the solution is added to the filtered water in the required proportion. Dose varies from 15 to 40 lbs. per million gallons of water. A solution consisting of 5% of powder and 95% of water by weight is generally prepared in the form of a paste and then thinned to a slurry and injected or pumped into the service mains. It is a whitish grey powder and requires 24 hours for its action.



### Sterilization of Completed Lines

Before being placed in service the entire supply line should be chlorinated. The chlorine may be applied by any of the methods. Water should be fed slowly into the new lines with chlorine applied in amounts to produce a dosage of 40 to 50 p.p.m. and retained for a period of 8 hours or more. A residual of not less than 5 p.p.m. shall be produced in all parts of the line. During the chlorination process all valves and accessories shall be operated.

For the medium sized and smaller plants a number of ingenious tanks have been commercially produced to combine the processes of coagulation, flocculation, and sedimentation in one unit.

### Other Chemicals for Water Treatment

*Potassium-permanganate* is useful for sterilizing small quantities of water. Dose is about 1/16 to 1/8 grain per gallon and removes about 98 per cent of bacteria in 4 to 6 hours. For disinfecting an open well, dissolve one ounce of potassium-permanganate in a bucket of water and lower it frequently into the well till it is mixed thoroughly with the water of the well. The well should not be used for 48 hours afterwards.

If with the minimum quantity of potassium permanganate the pink colour disappears, there is organic matter present in the water and more of permanganate should be added until the pink colour is permanent. It destroys organic matter already in a state of putrefaction instantaneously, but it takes half an hour or more to destroy matter about to putrify. The water can afterwards be filtered through clean sand and charcoal with advantage. Potassium-permanganate is also used to oxidize taste-producing materials. Usually only 0.05 to 0.10 p.p.m. is required and it may be added to the raw, filtered or filtered and chlorinated water.

Potassium permanganate has also been used as an algicide and for removing iron and colour.

*Activated Charcoal (carbon)* is used to remove tastes and odours whether due to decaying organic matter or chlorine. It can be employed 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 gas are retained by it. It has good colour removal properties, and is usually employed before sedimentation. Activated carbon is a proprietary product and is sold under various trade names. It reduces chlorine demand of treated water, and helps coagulation, if added before filtration.

*Caustic Lime (CaO).* Dose 5 to 15 grains per gallon of water. Suitable for small quantities. Water is filtered after reaction.

*Caustic Soda (sodium hydroxide)* and silicate of soda are used for correcting corrosive propensities.

*Carbon dioxide* is used to reduce pH values and to liberate hydrogen sulphide from sulphur waters.

Copper sulphate is used to control the growth of algae and other micro-organisms as detailed earlier.

High degree of sterilization should not be done; there is a point worth consideration regarding the effect of high degree of sterilization on the human alimentary system. It is reasonable to suppose that if, after becoming accustomed to a high degree of sterilization in the water consumed at one locality, a person, for some reason, has to change his environment to a locality where sterilization of water is less or none at all, this person runs a risk of disease infinitely greater than one accustomed to consume the less sterile water.

### Aeration of Water

Aeration is the process of mixing air with water for purification and has a number of useful functions. It removes carbon dioxide and increases quantity of oxygen. 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 fall down a series of 3 or 4 steps or weirs, or one long weir, over which the water tumbles in a thin sheet, discharging it as a fountain into an

open reservoir. Passing the water through open channels is also effective.

*Spray Nozzles.* Water is pumped through pressure nozzles to spray in the open air. (A nozzle of 1-in. orifice will discharge about 70 galls. per minute to a height of about 7 ft. at a 10 lbs. pressure).

*Trickling Beds.* Water is passed through perforated pipes over beds of coke, slag or stone about 2 ft. thick.

These may be contact filters or pressure filters. Compressed air is blown through diffuser porous plates or perforated pipes at the bottom of settling tanks.

Allowing the water to fall from a pipe from a good height into a collecting basin will bring sufficient aeration. Ground waters require more efficient aeration. Too much of aeration should be avoided. Where water is drawn from great depths (by the wells) which is sufficiently pure and does not need filtration, only aeration is done to further improve the quality.

### FILTRATION

Filters are of two main types : (i) Slow-Sand Filters, and (ii) Rapid-Sand Filters or Mechanical Filters.

#### Slow-Sand Filters

A slow-sand filter is an underdrained water-tight basin containing 3 to 5 ft. of filtering material, of which about half is fine sand ; the sand being submerged under 3 to 5 ft. of water and the basin being arranged to permit the percolation of water through the sand. A depth of water equal to the thickness of filtering sand has been found to give good results. The depth of water has to be adjusted according to the rate of filtration, turbidity of raw water and the thickness of the biological film. Too great or too small heads are not desirable. The following thickness may be taken for the filtering materials :—

4" to 6" of broken stone, brick bats or gravel  $1\frac{1}{2}"$  to  $2\frac{1}{2}"$  size.

3" to 6" of gravel  $\frac{3}{4}"$  to  $1\frac{1}{2}"$  size,

2" to 4" of gravel  $\frac{1}{8}"$  to  $\frac{1}{4}"$  size and above ] graded

6" of coarse sand

1' to 2' of fine sand (residue left between 70 and 100 meshes to the inch)

6" of very fine sand about 1/120" size.



Or alternatively : 2 to 3 ft. of sand of size 0.25 to 0.3 mm. is placed on a layer of gravel 8 to 10 inches thick, of size about  $\frac{3}{8}$ " to  $\frac{1}{4}$ " graded.

The sand should be as nearly as possible of pure silica or quartz. There should be a float gauge to show the difference of level of the water above the sand and in the outlet pipe and the head under which the filter is working, which should never exceed the depth of sand in the filter.

The rate of filtration is generally 30 to 80 galls. (with 100 galls. max.) per 24 hours per sq. ft. of filtering media. Minimum layer of sand should be 1 ft., and for sand less than 2 ft. rate of filtration should be 2 galls. per sq. ft. per 24 hours per inch of sand depth. The rate of filtration is sometimes expressed in terms of the rate of vertical descent of the water approaching the sand, in inches per hour. In a newly cleaned filter flow is rapid but as the filter works, the biological film formed at the top becomes thicker and the flow is reduced. As a rule more slowly filtration takes place, the purer will be the filtrate.

Vertical air vents passing through the sand should be provided for the passage of air to and from the bottom of bed to avoid disturbance of the filtering layers. These air vents are omitted where formation of "negative head" is avoided. Air binding can be prevented altogether by making it physically impossible for the filtering head to exceed the depth of water over the sand. This can be achieved by building a weir in the outlet chamber. (This has been explained further.) If arrangements are 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.

Bricks are laid on the floor under the gravel in transverse rows in two or three layers with spaces in between, each layer at right angles to the one below so as to form a series of channels. Water percolates through the gravel into the channels and is led to a central channel made under the floor. Or alternatively, series of underdrains are made. Underdrains are laid not more than 10 to



12 ft. apart and are surrounded by a layer of coarse gravel upon which the layer of graded gravel rests. The under-drains are laid with open joints to permit the entrance of water into them. A central main drain is laid lengthwise into which all the cross drains pour.

The floor of the chamber is made of cement concrete and in such a manner that the main drain can be fixed at a lower level than the cross drains. The chambers may not have roofs in hot climates. A covered chamber complicates the periodical washing of the filter sand. The rate of flow through the filter is usually controlled at the outlet either manually or by a float-controlled weir upon which a constant depth of water is maintained. The filter chamber size is fixed according to the flow and should be made in four units, two working at a time, no plant should have less than two filter units; and should have sides in the proportion of 2 to 3. Ordinarily the size of large units is 1 to  $1\frac{1}{2}$  acres, and small plants  $\frac{1}{2}$  to 1 acre.

*Filtration* is performed by a thin biological film formed after sometime on top of the sand which arrests suspended matter and entraps bacteria. When the filter gets clogged, top  $\frac{1}{2}$ " to  $1\frac{1}{2}$ " sand is removed by scraping off, washed with clean water and re-used. Provision should be made for drawing off the raw water from the top of the filter when it needs cleaning. 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 of 2 or 3 ft. at the end of the run. Cleaning is done usually once in a month. All materials should be cleaned and washed or changed at least once every six years. Measuring devices are better placed at the outlet, where they can show the rate at which the bed is filtering. Inlet measuring weirs record only the flow on to the bed and are misleading as the water level in the bed may be rising or falling.

Slow-sand filters are quite efficient and cheap if raw water load as regards colour, bacteria, algae and turbidity is low. They are also valuable as final process after rapid sand filtration of very polluted waters. Bacteria removal efficiency is about 95 per cent, colour removal efficiency

is about 50 per cent. Raw waters with over 100 p.p.m. turbidity cannot be handled; over 40 p.p.m. may produce unsatisfactory effluent.

The filter should be filled with filtered water from below, if possible, for at least 4" 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 the rate of  $1/5$ th of normal filtration rate. The water may then be turned into filtered water reservoir, but only at reduced rate of say  $1/3$ rd normal, which can be increased to the normal maximum rate in the course of the next 3 or 4 days.

Turbid waters should be passed at rapid rates through preliminary or rough filters, or treated as explained earlier, before passing on to a slow sand filter.

Experience shows that fine sand is more efficient in removing bacteria than coarser sand, and the finer the sand the greater the head is necessary to pass water through the bed at a given rate. The finer the sand the quicker the bed clogs, necessitating more frequent scraping. The finer 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 inside corner of each filter compartment into which the raw water enters at a low level and is gently introduced on to the surface by means of a pipe discharging vertically upwards. A valve is fixed on the raw water pipe outside each compartment for control of flow.

*Filtering Head* is the difference in level between the water on the top of the filter and the clear water chamber, which is about 4 to 6 inches in a freshly cleaned filter, and should not be greater than the depth of the filtering sand. This difference gradually increases as the filter gets clogged. When the loss of head becomes excessive and before a negative head is formed the filter should be cleaned.

*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 in the top few inches. When the loss in the top layer becomes greater than the head of water above, 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 for this, vertical air pipes are provided through the sand.

The depth of water should be greater than the maximum loss of head through the sand and the level of water at the outlet should not be below the level of the surface of the sand.

Usually the water receives no preliminary treatment other than plain sedimentation except when the raw water is very turbid. Slow-sand filters are less commonly used than rapid-sand filters. Slow-sand filters have simpler mechanism, need less supervision and less running cost than rapid-sand filters. There is less loss of head, are very suitable for small towns where land is available but initial cost is high.

#### **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 and then by filtration, with sterilization as the final process. A preliminary treatment consisting of coagulation and sedimentation is essential with a rapid filter.

*Advantages* : Initial cost relatively small ; rapid rate of filtration and small space required ; useful for waters highly turbid or coloured.

*Disadvantages* : Maintenance more expensive ; expert knowledge for handling machinery required ; cleansing difficult ; not so efficient for bacteria removal.

There are mainly two types of these filters, the *gravity* type, and the *pressure* type. Gravity filters are 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. The area of each unit is 20 to 30 ft. long and 12 to 20 ft. wide and are made of concrete or masonry, open at the top. The filter bed consists of 21" to 24" of clean gravel resting on the false floor and graded in shallow layers in sizes ranging from  $1\frac{1}{2}$ " to 3" in the bottom layer to  $1/10$ " size in the top layer. Above the gravel is a bed of sand, 25" to 30" thick formed of sand sizes between 0.50 and 0.70 mm. The max. gravel size is sometimes only  $1\frac{1}{2}$ " and the various sizes are varied in 2 or 3 layers each. Various types of under-drains are provided. The general method of their operation is as follows :—

Raw water is dosed ( $\frac{1}{4}$  to 2 grains per gallon) with a coagulant, generally sulphate of alumina (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 80 to 150 galls. /sq. ft. per hour (equivalent to over 50 times the rate admissible with slow sand filters). The filtration head in rapid filters varies from 5 to 12 ft. Rapid filters are usually found more satisfactory in India, provided the necessary skilled supervision can be given. Gravity filters are cleaned by filtered water, pumped or otherwise forced upwards through the sand.

The wash water is generally stored in an elevated tank at a height of 25 to 30 ft. or is supplied by means of a pump. The wash water required varies from about 1 to 3 per cent of the water filtered, depending on the frequency of washings. Washing is done after 2 to 3 days.

**Pressure Filters.** A Pressure filter is a type of rapid-sand filter which is in a closed cylindrical container and through which water passes under pressure. 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. They are, however, used for the softening of and removal of iron from ground waters. Filtration rates range from 2 to 4 galls. per minute per sq. ft. of filter area.

Details regarding these filters can be obtained from the manufacturing firms for the particular requirements.

### 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.

Design of masonry reservoirs has been detailed in Section 7. 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 from a fluid weighing 35 lbs./c.ft. Therefore, the wall/walls of such a reservoir need be designed only for this pressure.

The term "service reservoir" is used for a tank which is used for the storage of pure water which has undergone



whatever purification treatment is necessary before distribution to the consumer. A service reservoir provides for fluctuations in the daily demand. The term "impounding reservoir" is generally used for a tank which stores untreated surplus rain-water for use in dry seasons during a year.

Usual economic depths of ground reservoirs :—

Capacity (million galls.)	$\frac{1}{2}$	1	$1\frac{1}{2}$	2	3	5	8	10
Economic depth in ft.	12	14	16	18	19	20	23	25

(Also see under "Elevated Tanks")

Such reservoirs should preferably be built half in and half out of the ground, the material of excavation being used for banking material for the walls and covering over roof slab to keep the water cool.

**Service or Distributing Reservoirs** are built to furnish 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 depends 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.")

**Economical forms of Reservoirs.** For a single-compartment reservoir of a given depth the form most economical in materials is first a circle, then an ellipse, then a square and then a rectangle. For a reservoir to be sub-divided



into two compartments by a mid-wall the greatest economy will result when the breadth is approximately equal to two-thirds of the total length.

**Domestic Storage Tank.** Rainwater can be collected from roofs and catchment areas and stored in under-ground (or partly so) tanks. The filtration consists of passing the water through graded sand. Two small chambers are built on one side of the storage tank in line with each other; the first one is of a smaller size and the second of about double the size of the first and a partition is made in the second chamber up to about 6" below the roof. Water is collected in the first chamber which passes on to the second chamber in which coarse sand is filled and from where the water passes on to the third compartment with fine sand in it, and then to the clear-water storage tank. Water passes at top level between the first chamber and coarse sand and at bottom level from the coarse sand compartment to the fine sand and then again from top of fine sand to the clear-water storage tank. Suction pipe or draw-off is fixed at not less than 3" above the bottom of the clear-water tank. The water can be drawn out by a suction pump. The tank and the chambers are roofed in which manholes are provided. Arrangements are made to separate the first flush of rain from the after-fall which goes to the storage.

### Elevated Tanks

**Design of Tanks.** For small tanks a height equal to or somewhat greater than the diameter is an economical ratio. For large capacities and limited variations of pressure allowed, the tank should be relatively large and shallow.

**Steel Tanks.** Small tanks are generally made square and of pressed steel plates. For a circular steel tank made of ordinary steel plates, we need a plate  $3/16"$  thick up to a depth of about 10 ft. ; increase thickness of plate by  $1/16"$  per 10 ft. depth.

Thickness of side plates of a circular steel tank :—

$$t = \frac{2.6HD}{12000e} \quad (\text{Take min : thickness } 3/16")$$

$t$ =thickness in inches;  $H$ =ht. of water in ft.,  $D$ =dia. of tank in ft.,  $e$ =efficiency of joints (60 to 75 per cent.), 12000 is allowable tensile stress in steel.

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  $\frac{1}{4}$ " dia. Steel rod cross stays should be provided inside steel tanks to hold the opposite sides together.

For design of reinforced concrete tanks, see under "Reinforced Concrete."

The cost of elevated tanks 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. There is greater security with an elevated storage tank, than with a booster pump.

**Level of Service Reservoirs.** The ideal would be, that the supply of topmost floor in the highest building in the zone must be assured by gravity alone, and in the case of fire the jet from the hydrant must command all the parts of the highest roof. 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. It is essential (if practicable) that all service reservoirs are constructed at the same level so that one may feed into the other in case of a breakdown.

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 it should preferably be situated on the opposite side of the high consumption district from the pumping station. During periods of highwater use, 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 a vertical pipe with a bell mouth, placed a few inches above the top water level and connected to the scour pipe below the sluice valve which controls the sluice valve.

All cisterns or tanks will have the main supply pipe or inlet connected at top and the outlet or down service pipe connected on the opposite side of the inlet pipe 2" to 6" above the bottom 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 strainer the aggregate area of which should be at least double the area of the pipe. A stop-tap or stop-valve should be provided on the supply pipe outside the tank. A wash-out pipe with a stop-valve for cleaning and emptying the tank must be provided at the bottom. A waste pipe known as an overflow or warning pipe carrying off the surplus water when being overfilled (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. All connections should be screwed connections, with backnuts both inside and out. The reservoir should preferably be divided into compartments and a sluice valve should be fitted on the outlet pipe from each compartment.

*Ball Taps.* Where the internal diameter of the inlet of the ball tap is not more than  $\frac{1}{2}$ ", the float should be of an external diameter of not less than  $4\frac{1}{2}$ ". The thickness of a copper ball should not be less than 26 S.W.G. in cases where the external diameter of the ball does not exceed  $6\frac{1}{2}$ " and not less than 24 S.W.G., in cases where the external diameter of the ball exceeds  $6\frac{1}{2}$ ".

Steels tanks should be painted with leadless paint. Galvanized iron or lead lined tanks should not be used for drinking water. The tanks should be covered and ventila-



tors; the covers should be well fitting but not so as to be air-tight. Small cisterns of up to 30 galls. capacity should be provided with hand holes of 6" dia. and bigger sizes with man-holes.

*Break Pressure Tanks in Hills.* Provide with capacity of 500 gallons, fitted with a ball-valve at the inlet; the outlet should be at least  $2\frac{1}{2}$  ft. below the inlet.

**Cisterns.** Small rectangular galvanized mild steel tanks can be made of the following specifications:—

Capacity in Gallons	Specifications
Up to 40 galls. Depth up to 2 ft.	16 gauge or $1/16$ " thick plate with $1\frac{1}{2}" \times 1\frac{1}{2}" \times 3/16"$ angle irons or corner plates.
42 to 100 galls. Depth up to $2\frac{1}{2}$ ft.	14 gauge plate, and ditto.
100 to 200 galls. Depth up to 3 ft.	12 gauge plate with $1\frac{1}{2}" \times 1\frac{1}{2}" \times 3/16"$ angle irons at top, $\frac{1}{2}"$ corner plates, and two $\frac{3}{4}"$ dia. gal. W.I. stay rods, one riveted to angle iron framing at top and the other riveted to angle iron in the body of the cistern.
250 to 500 galls. Depth up to $4\frac{1}{2}$ ft.	$\frac{1}{2}"$ plate with $1\frac{1}{2}" \times 1\frac{1}{2}" \times 3/16"$ angle irons at top, and three $\frac{3}{4}"$ dia. gal. W.I. stay rods, one riveted to angle iron framing at top and two in the body of the cistern.
600 to 700 galls. Depth up to $4\frac{1}{2}$ ft.	8 gauge plate with $2" \times 2" \times \frac{1}{4}"$ angle irons at top and three $\frac{3}{4}"$ dia. gal. W.I. stay rods one riveted to angle iron framing at top and two in the body of the cistern.
800 to 1000 galls. Depth up to $4\frac{1}{2}$ ft.	$3/16"$ plate with $2" \times 2" \times \frac{1}{4}"$ angle iron at top, and four $\frac{3}{4}"$ dia. gal. W.I. stay rods, one riveted to angle iron framing at top and three in the body of the cistern.

Welding is better than riveting.

Each side, end, and bottom of the cistern should be in one piece, strongly and neatly riveted to flanged plates at angles with  $\frac{3}{8}"$  rivets, 1" pitch, and with angle iron and corner plates at top, riveted with  $\frac{3}{8}"$  rivets, 4" pitch. Bottom plates are generally  $1/16"$  thicker.

For average domestic property the main service pipe from the tank should not be less than 1", with  $\frac{3}{4}$ " branches to the baths and  $\frac{1}{2}$ " branches to lavatory basins, kitchen sinks, w.c. cisterns, etc. Where possible it is always advisable to run drinking-water supplies direct from the main service line before it enters the tank.

*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  $\frac{3}{4}$  to  $\frac{1}{2}$  of the nominal capacity since space has to be left above the water-line for pipe connections.

*Wooden cisterns* should be lined with lead of not less than 5 lbs. per ft. super or with copper of not less than 22 S.W.G. sheet.

*Hot Water Cylinders and Tanks* : Can be of the following thickness of iron sheets :—

Capacity, galls.	20	20 to 40	40 to 80	80 to 100	100 to 200
Thickness S.W.G.	14	14	12	10	3/16"

#### 4. DISTRIBUTION OF WATER

##### Consumption or Demand of Water

Rate of demand of water depends upon many factors, viz., pressure (demand increases with pressure); water charges, whether the supply is metered; if sewage system will exist; climate; population; class of consumers; gardens and lawns to be watered, etc. The water consumed varies widely in different houses, from day to day and month to month. Maximum demand is during mornings and 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.

Distribution mains, pumps and storage arrangements may be designed for maximum hourly consumption of three times the average hourly consumption, and service pipes and feeders for twice the average demand. This



will meet the normal peak demand. The maximum hourly demand may be from two to three times 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. Sedimentation tanks and filters are generally designed for average rate of demand.

Following are the approximate demands for various purposes :—

Drinking	$\frac{1}{2}$	to	1	gall. per head per day
Cooking	1	to	2	—do—
House washing and cleaning pots	3	to	5	—do—
Bath	10	to	20	—do—
Washing clothes	5	to	8	—do—
Water closets	6	to	8	—do—

For design, for drinking and domestic purposes, take consumption at 20 gallons per head per day ; where sewage system is provided, take additional 8 to 10 gallons per head. An allowance of 40 gallons per head per day is not much on the whole. This is where meters are installed.

Metering all services reduces 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 have proved deterrent to the expansion of domestic meter use. Domestic supplies are not generally metered in Europe while they are metered in America.

Where the entire supply of a town is given through street "standpoints" and where wells exist in addition, 10 gallons per head per day may be taken.

For buildings other than dwelling houses the following figures may be taken :—

Hotels	20	galls. per head per day
Offices	8	—do—
Hospitals	30 to 50	—do—
Schools	8	—do—
Hostels	20	—do—
Barracks	15	—do—
Factories	10	—do—



*Public purposes :—*

Road watering	3 gallons per 100 sq. ft. of road surface
Sewer flushing	1 gallon per head per day

*Irrigation :—*

Road-side trees	10,000 gallons per mile per day
Public parks	1,500 gallons per acre per day
Private gardens	—do—

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 stand-posts, bathing tanks, Dhobi ghats, and Industries, etc. It is estimated that each cattle requires 10 to 15 galls. per day. For washing cars, 30 gallons per washing, per week is taken.

**Fire demand.** It is estimated that only about 1 per cent of the total water supply is actually used to fight fires, but a substantial provision is usually made especially in America where over 60 per cent of plant costs in the small towns, falling to 20 or 10 per cent for the large towns, are provided to meet fire risks. This is probably due to large number of wooden buildings.

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 15 to 60 persons per acre 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 200 to 300 persons per acre. In the

closely built up residential areas in Old Delhi, density of population has been reported to be 900 to 1100 persons per acre.

### Water Consumption Rates in Some of the Principal Towns in India

Town	Galls. per head	Town	Galls. per head
Agra ..	28	Hyderabad	45
Allahabad ..	25	Kanpur ..	45
Bangalore ..	20	Karachi ..	50
Banaras ..	25	Lucknow ..	20
Bhopal ..	22	Madras ..	35
Bombay ..	70	Muttra ..	26
Calcutta ..	42	Patiala ..	20
Delhi ..	35	Patna ...	25
Gaya ..	45	Poona ..	60
Gwalior ..	20	Rangoon ..	100

In most of the towns this supply is considered inadequate.

In England the average consumption generally varies from 30 to 50 galls. in most of the towns, while in America it is from 70 to 200 galls. per head per day, with an average of 130 galls., less in small towns and more in large towns, and in some cases up to even 350 galls. In Rome it is 220 galls. This high rate of consumption has not been satisfactorily explained.

### 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 it is still considered to be not less than 20 per cent in a 100 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. In America, a leakage allowance of 50 to 70 gallons per mile of pipe line, per inch of diameter is considered adequate. Total loss may amount to as much as 50 per cent of the supply; or 8 to 16 galls. per head per day.

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.

**Leakage in Water Mains.** There is always some leakage in cast-iron pipes with lead joints due to expansion and contraction from temperature changes accompanied by a slight slipping of the lead on the iron at each joint; settlements also cause movement in the joints. Old and badly caulked joints will have more leakage.

The following table gives approximately leakage for a length of 1,000 ft. of pipe line, in gallons per 24 hours with a pressure of 100 lbs.

Dia.	3	4	6	8	10	12	16	20	24	30	36
Gall.	220	300	400	550	650	750	1000	1250	1500	1800	2300

The aggregate amount of leakage, apart from misuse, may be from 10 to 25 per cent of the total output. Where the pressures are high and efficient methods of waste prevention are not adopted, the wastage may be as much as 50 per cent. The amount of water wasted from a  $1/16$ " hole in a water main under 100 ft. head of water may be 350 galls. per day and from a  $1/4$ " hole, 10,000 galls. per day. A dripping tap may waste as much as 20 to 50 galls. per day and a tap running full 2,000 to 4,000 galls. per day.

**Tracing Leakage.** It is rather difficult to detect leakage in the underground mains. A metal rod 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 called "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. Where chemicals are employed : one grain of fluorescein is sufficient to colour 100 tons of water ; one grain of indigo dissolved in sulphuric acid and mixed with supply, will colour a ton of water.

### **Distribution System**

In a water-supply scheme distribution system represents a very large proportion (about 60 to 80 per cent) 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.

**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 frequently best located on the track of the main on a by-pass arrangement 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.

(a) **Gravity Distribution** is where the source of supply is above the town 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.

(b) **Distribution by Pumps and Elevated Storage Tanks.** (See under "Service Reservoirs".) The excess of water pumped during periods of low consumption is stored in the tanks or reservoirs and during periods of high consumption the stored water is drawn upon to augment that pumped. This method allows uniform rate of distribution and pumping and meets demands during emergencies and breakdowns, therefore, is fairly reliable.

(c) **Direct Pumping Without Storage.** This is the least reliable system and should be used only for small supplies. The pumps force water directly into the mains. The rate of pumping has to be varied according to the demand and a number of pumps have to be installed of varying capacities. A failure of the power will mean complete failure

#### **Supply Mains**

Mains fall into three groups according to the primary purposes which they serve, viz., trunk, secondary and service. A "trunk main" is usually an important main which carries water from one place, *e.g.*, a pumping station or reservoir, to another or to a district where the water will be used. As far as possible, a trunk main should not be used for supplying 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 connections for smaller mains are taken through them. They are used to supply the large consumers direct.

Minimum size of cast iron water mains is 4 inches for mains fed from one side and 3 inches for mains fed from both ends. No branch feeder should be less than 3 inches dia. 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 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 wash-out 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 3 ft. under roadways, and not less than 2'-6" (min. 2'-0") 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.

Trunk mains should be planned so as to avoid main streets and points of traffic congestion and should be laid on one side of roads. As far as practicable all important mains should be laid in duplicate so as to avoid the risk of the supply being totally cut-off in case of a burst or repairs.

**Lay-out of Pipe Lines.** There are four systems of laying the distribution pipes : (i) Tree or Dead-end System ; (ii) Grid Iron System ; (iii) Circular or Ring System ; and (iv) Radial System. The type of lay-out system of pipes to be adopted depends upon the topography of the town, and the location as well as the elevation of the source of supply. No 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 larger 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 a single main is laid, the whole supply will be cut off in case of repairs.

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. Where centrally laid separate reservoirs are provided for each district or zone, the supply pipes should be laid radially taking off from the central reservoirs and joining with a sub-main laid round the boundary of the district. As far as possible, large mains should be provided at intervals of from  $\frac{1}{4}$  to one mile 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. Feeders may be joined with the mains with small pipes of about 3-in. dia.

In working out a project it is necessary to start at the consumer's end and work backwards. It is usually difficult to calculate the size of distribution pipes and pressure with accuracy except when a single trunk main is laid. Probably maximum demand should be calculated for each district and the size of main fixed accordingly. (See further under "Losses of Pressure in a Distribution System".)

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 less than 2-in. 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.

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, the closer the pipe to the h.g.l. the lower will be the pressure in the pipe. High pressures at steep slopes can be avoided by breaking the hydraulic grade line with overflows or auxiliary reservoirs or by installing relief valves. No section of the pipe line should be more than 20 ft. (max. 25 ft.) above the hydraulic gradient at that section, as otherwise vacuum will be formed and flow will cease. Where a water main goes above the h.g.l. by more than 20 ft., pumping will have to be resorted to, to keep up the pressure.

**Supply Pressure.** Taps should have a residual head of 5 ft. minimum (2 ft. at all cisterns) and should preferably be 20 to 25 ft. to ensure ample supply under good pressure. No domestic service pipe should be less than  $\frac{3}{4}$ " dia. except when the pressure is very high, it may be  $\frac{1}{2}$ ". Avoid pressures above 100 lbs./sq. in. in the mains. (Also see under "Preparation of Project Estimates" and "House Services").

### Strength of Pipes :

*Pressure of Water in Pipes* =  $0.4335 H$ , in lbs./sq.in.

$H$ =head of water in feet.

*Bursting Pressure in Pipes* : Internal diametral pressure tending to burst the pipe produces a stress of :—

$$\frac{P \times d}{2t} \text{ lbs./sq. in.}$$

Longitudinal stress is :—

$$\frac{P \times d}{4t} \text{ lbs./sq. in.}$$

$P$ =Water pressure in lbs./sq. in..  $d$ =dia. of pipe in



inches (inside),  $t$ =thickness of pipe wall in inches.

$$\text{Riveted Steel Pipes : } t = \frac{Pd}{2fe}$$

where :

$t$ =thickness in inches,  $P$ =total pressure in lbs./sq. in. (includes allowance for water hammer),  $d$ =dia. of pipe in inches,  $f$ =safe working stress in lbs./sq. in.,  $e$ =efficiency of longitudinal joints.

For riveted lap joints, take efficiency of the joints as 55 per cent for single and 70 per cent for double riveting. For a triple riveted lap joint, efficiency is taken up to 80 per cent and for a lock bar joint 100 per cent. Strength of welded pipes is taken at 90 per cent. Thus, a thicker plate is required for a riveted pipe than for an all welded pipe.

Steel riveted pipes have generally a single row of rivets at the circumferential joints and two rows of rivets along the longitudinal joints, as the diametral pressure (tending to burst the pipe) is double the longitudinal tension in the pipe.

The value of  $t$  found from the formula is taken to the next larger sixteenth of an inch and then  $1/16''$  is added to allow for corrosion. In large pipes under small pressure, the thickness of metal computed must still be increased to give the pipe the necessary stiffness. The usual practice is to have plates  $\frac{3}{8}''$  thick inclusive of an allowance of  $1/16''$  for corrosion.

*R. C. Pipes :*

$$A_c = \frac{P \times d \times 12}{2 \times 16000} \text{ sq. in. ; } A_l = \frac{P \times d \times 12}{4 \times 16000} \text{ sq. in.}$$

$A_c$ =area of circumferential reinforcement per ft. run,

$A_l$ =area of longitudinal reinforcement per ft. run,

16000=steel reinforcement fibre stress.

*Spacing of Pipe Supports :*

A footing structure shaped to fit the pipe it supports is called a *cradle*.

Stress due to bending is :

$$f_b = \frac{3WL^2}{2Z}$$



$L$ =distance between supports in ft.,  $W$ =total load (pipe+water) in lbs. per ft. run,  $Z$ =section modulus of pipe in inches, (steel area in R.C. pipes).

Longitudinal fibre stress due to water pressure is :

$$f_t = \frac{P}{4t} \times d$$

Total stress =  $f_b + f_t$

$$\text{whence } L = \sqrt{\frac{2Z}{3W} \left( f - \frac{PD}{4t} \right)}$$

Take  $f=11,000$  lbs./sq. in. for steel pipes,  
8,000     "     "     W.I. pipes.

$$Z = \frac{\pi}{32} \left( \frac{D^4 - d^4}{D} \right)$$

where  $D$  is the outer diameter of the pipe in inches.

*Bending Moment in Pipes :*

Taken as thin elastic ring loaded uniformly on diameter :

$$BM = \frac{WD}{16} \quad D \text{ is external dia. of ring.}$$

*Time for Emptying a Water Tank :*

With a round hole at the bottom (well rounded or bell mouth).

$$T = \frac{A\sqrt{H}}{200a} \quad T = \text{time in minutes, } H = \text{ht. of water in ft.,}$$

$A = \text{area of water surface in sq. in., } a = \text{area of hole in sq. in.}$

(Also see under "Hydraulics")

**Water Hammer or Surge Pressure :**

Water hammer is the momentary pressure produced by the sudden stoppage of moving water in a closed conduit or pipe and is usually much in excess of the normal static pressure. This is caused by the too rapid closing of a valve or the sudden starting of a pump.

Allow pressure due to water hammer as follows in addition to the static pressure in pipes :—

Dia. of pipe in ins.	3 to 10	12	16	20	24	30	36	40 to 60
Pressure in lbs./sq. in	120	110	100	90	85	80	75	70

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. (If valves are closed slowly, hammer pressure is considerably reduced but this is not always practicable, hence allowance for this pressure). Water hammer can be guarded against by the use of a tower surge tank or standpipe at the end of the line, or relief valves. At pumps and plumbing systems, air chambers are placed at or near the closure points to absorb the shock of water hammer.

**Velocities in Mains should not generally exceed :**

(with max. of 10 ft./sec.)

3 ft./sec. for	4" pipes	With high velocities in small mains loss of head by friction is excessive.
4 " " "	6" "	
5 " " "	10" "	
6 " " "	16" "	
7 " " "	service pipes.	

Velocities, however, should be high enough to prevent deposits of silt in the pipes ; 2 to 2½ ft./sec. will be satisfactory.

**Max. Economical Velocities given by Fanning for Long Water Mains**

D	4	6	8	10	12	14	16	18	20	22	24	27	30	32
V	2.5	2.8	3.0	3.3	3.5	3.9	4.2	4.5	4.7	5.0	5.3	5.8	6.2	6.5

D is diameter in inches, V is velocity in ft./sec.

Unwin's formula for finding out the economical velocities in town water supply mains :-  $V = 1.45d + 2$

d is dia. of pipe in feet ; V is velocity in ft./sec.

### Capacity 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. All the maximum demands of the separate services may not occur simultaneously, therefore, it is generally considered that the carriage capacity of mains



should be three times the average hourly demand of water and that of the service pipes should be twice the average demand as has been stated earlier. The maximum hourly demand may be from 2 to 3 times or more of the average hourly demand. (Also see under "Distribution of Water").

For the design of cast iron water mains an allowance should be made in the diameter for the incrustations that will occur in course of time and will decrease the discharge. 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 corrode rapidly, in others they may remain permanently as smooth as they were first laid. A common rule is to calculate the size of the pipe required when clean and add 1 inch 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.

**Scraping of Pipes.** Although scraping of pipes increases their carriage capacity, but unfortunately, 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.

**Pipe Trenches.** The sides of the trenches should be as nearly vertical as possible, the width at the bottom being at least 18" wider than the socket of the pipe so as to allow room for ramming the refilled material. Width of the trenches may be as follows:—

3" to 6" pipes ..	2'-0"	Minimum width of a pipe
7" to 9" pipes ..	2'-6"	trench should be 21" even
10" to 12" pipes ..	3'-0"	for the smallest pipe, to facilitate working

Where joints are to be made the trench should be wide enough for the jointers to work. Where wide excavation is not permissible such as in roads and streets, shoring will have to be resorted to. The excavated material should be thrown on to one side of the trench and pipes stacked on the other side.



**Pipe laying.** Cast iron with spigot and socket-lead joint : All pipes and fittings should be sounded with a light hammer to detect any cracks, before laying. 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. Where there is gradient, pipe laying should proceed in an "up-hill" direction to facilitate joint making. Pipes should be laid with their sockets facing up-hill irrespective of the direction of flow on inclines. Pipes of large diameter should be laid with a minimum rise or fall of  $\frac{1}{4}$  in. (preferably  $\frac{1}{2}$  in.) in 12 ft. to ensure that air travels to the air valves. After laying, the open end should be plugged to prevent access of animals, soil, etc. When pipes are laid on brick pillars, the socket should be well clear of masonry to allow the lead joint to be caulked. Any deviation, either in plan or elevation of less than  $11\frac{1}{4}$  deg. should usually be effected by laying the straight pipes round a flat curve of such radius that the minimum thickness of lead at the face of the socket is not reduced below  $\frac{1}{4}$ ", or the opening between the spigot and socket increased beyond  $\frac{1}{4}$ ", at any joint. A deviation of about  $2\frac{1}{4}$  deg. at each joint can be effected in this way. The spigots should be carefully centered in the socket by one or more laps of (at least  $\frac{1}{2}$  in. diameter) hempen spun yarn with about an inch of overlap, sufficient yarn only being forced into the socket to leave a correct depth of lead. The object of the use of yarn is to centre the spigot in the socket, to prevent the flow of molten lead into the bore of the pipe and also to reduce the amount of lead required to complete the joint. The proper depth of each joint should be tested before running the lead by passing completely round it a wooden gauge notched out to the correct depth of lead. 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.

#### **Laying and Jointing Cast Iron Water Mains**

(Also described in the Section on "Sewerage").

**Joints.** Cast iron pipes are supplied with three kinds of joints : (i) flanged (bolted) ; (ii) turned and bored (spigot and socket) ; and (iii) spigot and socket (lead),

also called spigot and bell joints.

**Flanged joints.** Pipes are made with flanges which are jointed together with bolts and nuts. Such joints are used in pipes for (i) valve or meter connections which may have to be removed for repairs, (ii) vertical, inlet and outlet pipes of reservoirs, (iii) for pipes where pressure is high, (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 used for inside work or for connections in confined or vertical positions. Packing ring or gasket (washer) of rubber, leather, compressed fibre board or gutta-percha, smeared with graphite paste or a 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  $1/16$ " thick for 8-in. pipes and smaller, and  $3/16$ " thick for bigger pipes.

**Turned and Bored joints.** In this type of joint the inside of the socket and the outside of the spigot are turned to an accurate fit with a slight taper. No lead or other jointing material is used and water-tightness depends solely on the perfect fit between the tapered machined faces of the socket and spigot. The ends are smeared with red lead and driven home by blows of a wooden mallet. A small amount of either cement or red lead is caulked into the recess provided towards the outside edge of the socket as an additional safeguard against leakage. These joints are absolutely rigid and do not lend themselves to any deviation from the straight line or any expansion or contraction, and cannot be laid to curves. Turned and bored joint are more economical and require less skill in laying but are not much used now-a-days.

**Spigot and Socket joint** lead caulked is the most commonly used and lends itself to expansion and contraction under moderate changes of temperature and slight settlements which occur with buried pipes. Curves of large radius can be built with straight pipes by deflecting each joint slightly.

The alignment of the pipeline is first marked on the ground by stakes driven 100 ft. apart on straight lengths



and 25 to 50 ft. apart on curves.

**To lead the joints.** The pipes must be clean and absolutely dry, or else, the lead may sputter and blow out and the joint be spoiled. 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 3" wide and  $\frac{3}{4}$ " thick and 4" longer than the circumference of the joint. Wrap this round the joint with the overlap on top and make a V-shaped 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.

When a section of a few hundred feet is leaded, the caulking will be put in hand. Each gang of caulkers should consist of at least four blacksmiths, all working on the same joint at a time. For large pipes it is also necessary to leave one or more air vents around the lower half of the joints. The lead should be rendered thoroughly fluid heated to a temperature such that when stirred it will show a rapid change in colour. Scum shall be removed before pouring, 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. Lead should be caulked only sufficiently tight with a hand hammer weighing not less than 4 lbs., to prevent leakage as a lead joint excessively caulked will become rigid. Lead joints should be finished  $\frac{1}{8}$  in. inside the face of the socket. The lead used should be soft and free from admixtures of tin. Blue pig lead makes good joints. Lead wool (or shreaded lead) instead of molten lead, should be used in wet locations.

Mixtures of sand or fine stone dust and hot sulphur in equal proportions are sometimes used instead of lead, and the same method and process is followed as for lead caulked joints. Proprietary materials are also available which are melted on the job and poured in the joint instead of the



lead. Cast iron pipes can also be jointed with asphalt or tar mixed with stone dust or cement. Joints leak or 'sweat' slightly at first but tighten up in a short time. Rigid joints should not be used to connect a newly laid line to an old one as the settlement of the new line will cause a broken pipe.

**Flexibility of joints.** Poured and lead wool joints permit some movement for expansion and contraction, and the pipes can also be laid on the sweep round curves the min. radii of which are approximately :—

220 ft. for 3" pipes, 260 ft. for 6" pipes, 380 ft. for 9" pipes. These figures are based on pipes 9 ft. long, for pipes 12 ft. long the minimum radii should be increased by 33 per cent.

It is advisable, if possible, to 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. Thrust is exerted at bends when a pipe is under pressure. A concrete block should be provided under a bend in soft grounds except in the case of small pipes under low pressure. Where the hydraulic thrust is in an upward direction, anchor-blocks of sufficient weight should be formed to which the pipes should be secured with steel straps.

On a steep hill side the cumulative weight of the pipes may be so great as to injure the joints. To remedy this the pipes should be anchored at intervals to masonry pillars. 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.

### **Testing of New Pipe Lines**

After each section of the pipe line has been laid, it should be tested for water-tightness before being covered in. This can be done by closing each end by means of a watertight expanding plug of which there are several types, or a valve, and filling the pipe-line with water. The pressure can be raised by means of a small hand force pump or hydraulic pressure pump till it registers 25 per

cent above the highest working pressure, on the gauge. Some engineers recommend that the test pressure should be 50% in excess of the normal working pressure. Each section to be tested should be slowly filled with water, care being taken to expel all air from the pipes; if necessary the pipes should be tapped at high points to expel the air. The pressure should be applied for half-an-hour, and all pipes, fittings, valves, hydrants, and joints carefully examined for defects. The test pump having been stopped, the test pressure should maintain itself without measurable loss for at least half-an-hour.

No pipe installation should be accepted unless the leakage (evaluated on a pressure basis of 150 lbs./sq. in.) is less than 85 galls. per 24 hours per mile per inch diameter of pipe of 12-ft. lengths and proportionate for other lengths of pipes (due to joints). Leakage is generally due to the newly made joints.

All pipes and fittings are tested under a pressure equal to twice the maximum working pressure. While undergoing the test, the pipes are struck with a small hammer to detect any leakage through cracks.

Leakage tests on cement joints should be made after at least 2 weeks and on sulphur compound joints after 4 to 5 weeks. Leaking lead joints should be recaulked and sulphur compound or cement joints showing more than slight sweating should be cut out and replaced. Re-done joints should be retested.

### **Service Connections from Mains**

The pipe extending from the distribution main to the 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 branch which can be



taken off any given size of the main. The diameter of the branch should not exceed  $\frac{1}{3}$ rd the diameter of the main. Over 2-in. holes are not generally tapped into the mains to avoid weakening them.

The maximum size of ferrule which can be used safely is limited to  $\frac{1}{2}$  in. with 3-in. mains,  $\frac{3}{4}$  in. with 4-in. mains, and 1 in. with 6-in. mains. 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 instead of on the top of the pipe. 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. Screw-down ferrules are used. The ferrule is  $\frac{1}{8}$  to  $\frac{1}{4}$  in. less in diameter than the pipe and the pipe can then be stepped to increase the discharge.

A short length of lead pipe (say about 2 ft.-6 in.) or "gooseneck" is used to make a flexible connection between the service and the ferrule cock. Unless flexibility is provided, unequal settlement will break the service pipe from the main. Wiped lead joints are used for connections.

For making connections in asbestos-cement mains, special saddles may be affixed before the ferrules are screwed in.

**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.

**House Services.** For an average house with about 15 inhabitants the service main should be of 1" dia. with  $\frac{3}{4}$ " and  $\frac{1}{2}$ " branches. For smaller houses, no domestic service pipe should be less than  $\frac{3}{4}$ " dia. except when the pressure is very high it may be  $\frac{1}{2}$ ". The delivery from the pipes should be as follows :

Dia. of pipe	$\frac{1}{2}$ "	$\frac{3}{4}$ "	1"	1 $\frac{1}{4}$ "	1 $\frac{1}{2}$ "	2"
Dis. galls./mt.	2	4	6	10	15	25





### Fittings for Service Pipes

**Stop-Cocks.** In every case a stop-cock should be provided within the boundary of the premises 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 big buildings another stop valve should be placed on the service pipe just inside the building as that would help in case of repairs.

*Bends* are manufactured to certain radii (about 20 sizes) varying from  $1\frac{1}{4}$ " to 20" internal radius, and 4" to 40" 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 2" 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 diameter.

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.

For cast iron pipes the standard specials are : 90°, 45°, 22½°, and 11½° bends, equal and reducing 45° branches tapers and collars. (B.S.S. provide for 5½° bends as well) A 90° bend is also called ½ bend, and a 45° bend ¼ bend and so on.

**Fire Hydrants.** May be located at distance apart of about 500 to 1000 ft. and with an area of about 40,000 sq.

ft. to 100,000 sq. ft. according to the density of population and importance of the locality as regards its protection against fire. They should be provided at all mains and crossings. Residual pressure recommended at the fire hydrants is :—

With pumping engines . . . . . 10—20 lbs./sq. in.  
For direct flow from hydrants . . . 50—75 lbs./sq. in.

The minimum rate of flow required to deal with a substantial fire in a small house is about 200 galls. per minute.

Hydrants are of two types : Pillar or Post Hydrants and Sunk or Flush Hydrants. Pillar hydrants are conspicuous and easily located and the hose can be speedily manipulated on them, but are an obstruction on the road and in the way of traffic. Vertical pipes are screwed on the sunk hydrants when required, and hose pipes fixed on to them. Hydrants can be bolted direct on to the flanged tees of the main with pipes 3" to 6" diameter (prefer bigger size) and should be located at street intersections. Common arrangement of hydrants is recommended to be one 4½" connection for fire engine pumper and two 2½" connections for hose outlet. A flow of not less than 150 gallons a minute is necessary. Prefer 250 gallons through a 1½" smooth nozzle. Velocity heads in the jets issuing from the hydrants can be tested by means of hydrant Pitot tubes.

*Fire Stand-pipes inside buildings.* The diameter of the stand pipe should be 3" to 4". The branches for inside fire valves should be 1½" size. The valves should be of the full waterway or gate pattern. The fire hose should be able to stand a pressure of 400 lbs./sq. in. and be of 1½" size. Every fire hose to be provided with a nozzle 1" to 1½" opening.

### Losses of Pressure in a Distribution System

The determination of the pipe pressure losses in a distribution system by computation is complicated and the result is uncertain. Complicated networks of pipes permitting flow by many routes are often treated by approximations or by trial and error. The compound pipe method



offers an appreciable degree of precision and is described below :—

The diameter of the pipe equivalent to all of the pipes in the system is first to be determined and then the head loss in the system will equal to the head loss in the equivalent pipe. A pipe is said to be equivalent to one or more other pipes when the head loss in the pipe is equal to the head loss in the one or more other pipes for the same rate of flow. To determine the diameter and length of one pipe equal to a "series" of pipes, the following procedure is to be followed :

(i) Assume any convenient rate of flow through the series of pipes and find the sum of the head losses through all of the pipes of the series for this rate of flow.

(ii) Find the diameter of one pipe of any convenient length which will carry the rate of flow assumed in the preceding step for the same total loss of head. This is the diameter of the equivalent pipe.

To determine the diameter and length of one pipe equivalent to a number of pipes connected "in parallel", proceed as follows :

(i) Assume a total head loss between the two points at which all of the pipes are connected. This head loss is the same for each pipe of the system.

(ii) Find the sum of the rates of flow through all of the pipes of the system for the assumed head loss.

(iii) Find the diameter of one pipe which will carry the sum of all the flows in each of the pipes of the system at the assumed head loss. The length of this one pipe may be any convenient figure. This is the equivalent pipe for the system.

If both series and parallel pipes are connected to each other in a distribution system, they should be studied separately and then the equivalent pipes combined finally to make one equivalent pipe for the entire system.

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.

To find the diameter of a pipe of given length to deliver a given quantity of water under a given head, use the following equation :

where :

$$d = 0.234 \left( \frac{Q^2 L}{h} \right)^{\frac{1}{5}}$$

d = dia. of pipe in feet,  
Q = discharge in cubic ft./sec.  
L = length of pipe in ft., h = head in ft.

Derivation from Manning formula for loss of head due to friction in pipes :- (Also see under "Hydraulics").

$$h = \frac{ALV^2}{\sqrt[3]{d^4}}$$

A = 2.88 n<sup>2</sup>; d is in ft.  
h/L is actual slope of pipe.

Material	Value of A	n
Clean coated cast iron .. .. .	0.00042	0.012
Clean uncoated cast iron .. .. .	0.00049	0.013
Riveted steel .. .. .	0.00065	0.015
Galvanized iron .. .. .	0.00057	0.014
Brass, copper .. .. .	0.00029	0.010
Smooth concrete .. .. .	0.00042	0.012
Cement mortar finish .. .. .	0.00049	0.013
Vitrified sewer pipe .. .. .	0.00035	0.011

Pipe dia. in ins.	Value of $\sqrt[3]{d^4}$	Pipe dia in. ins	Value of $\sqrt[3]{d^4}$
1	0.0364	24	2.52
1½	0.0625	30	3.39
2	0.0917	36	4.33
3	0.157	42	5.31
4	0.231	48	6.35
6	0.397	54	7.43
9	0.682	60	8.56
12	1.000	66	9.71
18	1.720	72	10.90

Delivery in gallons per minute through a pipe of d inches diameter is approximately = 2Vd<sup>2</sup>.

Box's formula :

$$H = \frac{G^2 L}{(3d)^5} \quad H = \frac{LV^2}{1600 D}$$

where : H = friction losses in ft. ; G = discharge in galls. per minute ; L = length of pipe in yards ; d = dia. of pipe in inches ; D = dia. of pipe in ft.

## Friction Losses in Fittings

Fittings	Equivalent length of straight pipe in feet
Sluice valve, full-bore type ..	15
Non-return valve, full bore type ..	20
Foot valve, full bore type ..	20
Strainer .. .. .	40
Bends .. .. .	15*
90° elbows .. .. .	20 or 30d
Tees and sharp or square elbows ..	25 or 50d

\*This length is taken for 9-in. dia. and 90° bend, 10-in. dia. and 45° bend, 12-in. dia. and 22° bend. The length of pipe equivalent is increased with larger pipes and decreased with smaller.

Recommended Sizes of Pipes for Various Lengths of Discharge Lines :—

Dis. in galls. per min.	Length of pipe line in feet									
	50	100	250	500	750	1000	2000	3000	4000	5000
10	1	1½	1½	1½	1½	1½	1½	2	2	2
15	1½	1½	1½	1½	2	2	2	2½	2½	3
20	1½	1½	2	2	2½	2½	2½	3	3	4
30	1½	1½	2½	2½	2½	3	3	3	4	4
40	2	2	2½	2½	3	3	3	4	4	5
50	2	2	3	3	3	4	4	4	5	5
60	2	2½	3	3	3	4	4	4	5	5
80	2½	2½	3	3	4	4	5	5	5	5
100	2½	3	3	4	4	4½	5	5	5	6
120	3	3	3	4	4	5	5	5	6	6
150	3	4	4	5	5	5	5	6	6	6
200	4	4	4	5	5	5	5	6	6	6
250	4	4	4	5	5	5	6	6	6	7
300	4	4	5	5	5	6	6	7	7	8
400	5	5	6	6	7	7	8	8	10	9
500	6	6	6	7	7	8	8	10	10	10
650	6	6	7	7	8	8	10	12	12	12
800	7	7	8	8	10	10	12	12	14	14
1000	8	8	8	10	10	10	12	12	14	14
1200	8	8	10	10	12	12	14	14	15	16
1500	10	10	10	12	12	14	14	15	16	16



**Flow of Water in New House Service Pipes**  
*Discharge in c. ft. per minute*

Pressure in Main lbs./sq. in.	Nominal internal dia. of pipe in inches							
	$\frac{1}{2}$	$\frac{3}{4}$	1	1 $\frac{1}{2}$	2	3	4	6
Through 35 ft. 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	28.5	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	269
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. 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. of service pipe and 15 ft. 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. of service pipe and 30 ft. 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



Length of Equivalent Pipes—(Contd.).

Dia. in.	13	14	15	16	17	18	20	22	24	27	30	33	36	40	44	48
13	1															
14	1.22	1														
15	1.48	1.20	1													
16	1.73	1.39	1.18	1												
17	2.03	1.64	1.37	1.17	1											
18	2.35	1.87	1.59	1.34	1.16	1										
20	3.08	2.43	2.08	1.74	1.52	1.30	1									
22	4.00	3.09	2.70	2.16	1.98	1.65	1.26	1								
24	4.92	3.84	3.32	2.75	2.43	2.05	1.57	1.24	1							
27	6.82	5.16	4.60	3.70	3.32	2.75	2.11	1.67	1.34	1						
30	8.72	6.54	5.88	4.80	4.30	3.57	2.74	2.16	1.74	1.29	1					
33	11.3	8.52	7.58	6.11	5.58	4.55	3.49	2.75	2.21	1.65	1.27	1				
36	13.9	8.41	9.37	7.59	6.85	5.65	4.34	3.42	2.74	2.05	1.58	1.24	1			
40	15.3	13.5	12.4	9.95	9.01	7.34	5.64	4.44	3.57	2.66	2.08	1.65	1.26	1		
44	22.0	17.5	15.9	12.5	11.5	9.34	7.17	5.66	4.55	3.39	2.61	2.00	1.65	1.27	1	
48	28.8	23.1	19.4	15.6	14.2	11.6	8.92	7.03	5.65	4.21	3.24	2.55	2.05	1.58	1.24	1

To find how many pipes of small diameter will take to discharge same quantity as one large pipe

$$\text{No. required} = \frac{\sqrt{(\text{dia. of large pipe})^5}}{\sqrt{(\text{dia. of small pipe})^5}}$$



## Loss of Head due to Friction in Water Pipes

Head Lost in Feet per 100 ft. length of Pipe Line

Bore of Pipe	$\frac{1}{2}$ "	$\frac{3}{4}$ "	1"	1 $\frac{1}{4}$ "	1 $\frac{1}{2}$ "	2"	2 $\frac{1}{2}$ "
Discharge in gallons per minute	Gradient or head lost in ft. per 100 ft. length of pipe line						
2	36	4.7	1.1	0.35	0.14	0.03	
3	83	10.5	2.4	0.78	0.31	0.07	
4	147	19.0	4.3	1.4	0.55	0.13	..
5	230	29.3	6.7	2.2	0.86	0.20	0.06
6		42.0	9.7	3.1	1.2	0.28	0.09
8		75.0	17.0	5.6	2.2	0.50	0.16
10		117.0	27.0	8.7	3.4	0.78	0.25
12			38.7	12.5	4.9	1.13	0.36
15			60.5	19.5	7.7	1.76	0.56
18			87.1	28.1	11.1	2.50	0.80
20			108.0	34.7	13.7	3.1	0.99
22				42.0	16.6	3.8	1.2
24				50.0	19.8	4.5	1.4
26				58.6	23.1	5.3	1.7
28				68.0	26.8	6.2	1.9
30				78.0	31.0	7.1	2.2
35				106.0	50.0	9.6	3.0
40					55.0	12.5	4.0
45					99.0	16.0	5.0
50					85.6	19.6	6.2
60						28.0	8.9
70						38.4	12.1
80						50.0	15.9
90						63.5	20.1
100						78.5	24.8
110							30.0
120							35.7
140							48.6
150							55.8
160							50.4

The above table should be used for permanent installation.

When head lost is required for new smooth pipes, the above values should be reduced by 30 per cent.

Dia. ins.	Discharge in thousand galls. per hour							
	2	4	6	8	10	12	16	20
3	40	140	350	500	—	—	—	—
4	9	35	80	140	220	320	—	300
5	3	12	28	50	75	110	190	110
6	1.1	5	10	18	28	40	75	55
7	0.5	2.1	6.5	9	14	20	35	24
8	—	1.0	2.1	4	6	8.5	16	14
9	—	—	1.2	2	3	5	9	8
10	—	—	—	1.2	2	3	5	8
12	—	—	—	—	—	1.0	2	3.5

[illegible]

# FLOW OF WATER IN

## TABLE GIVING DISCHARGE

	Bore of Pipe															
	3	4	5	6	7	8	9	10	12	14	15	16	18	20		
10	90	196	385	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	750	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	68	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	5300		
90	29	64	118	190	290	420	580	750	1240	1850	2240	2660	3680	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	285	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	2460		
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	745	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		

The table is based on a roughness co-efficient  
New clean cast iron pipes will carry about 33%



# ENCRUSTED CAST IRON PIPES

## IN GALLONS PER MINUTE

inches											ft. per second	Hydraulic Gradient—ft. in 1000
21	22	24	27	28	30	33	36	40	44	48		
18400	21000	26300	36700	40000	—	—	—	—	—	—	10	
14800	17000	21400	29500	32600	40000	—	—	—	—	—	15	
12800	15600	18400	25400	27800	34000	44300	—	—	—	—	20	
10200	11600	13900	20600	22500	27400	36000	—	—	—	—	30	
8750	10000	12800	17600	19400	23400	30600	39300	—	—	—	40	
7800	8850	11200	15800	17500	21000	26400	35100	—	—	—	50	
7100	8100	10200	14200	15600	18900	24700	31500	42800	—	—	60	
6500	7400	9400	13000	14200	17400	22500	28500	39450	—	—	70	
6150	6950	8650	12100	13300	16500	21400	27000	36800	—	—	80	
5650	6500	8250	11200	12500	15400	19150	25500	34500	43500	—	90	
5300	6150	7800	10600	11700	14600	18800	24000	32250	42000	60000	100	
4950	5600	7100	9750	10700	13200	17300	21880	29450	38200	52000	120	
4350	5000	6400	8700	9500	11650	15400	19500	26100	33800	43000	150	
3750	4270	5350	7500	8400	10000	13200	17000	22550	29000	37000	200	
3370	3940	4900	6750	7350	9000	11800	15000	20250	25800	33700	250	
3040	3450	4350	6000	6510	8100	10500	13500	18150	23200	30000	300	
2740	3150	4000	5490	6000	7350	9500	12300	16650	21400	27000	350	
2550	2920	3750	5100	5610	6900	9000	11500	15600	20000	25500	400	
2420	2740	3530	4850	5250	6500	8500	10900	14650	18800	24000	450	
2320	2600	3340	4570	5000	6200	8100	10200	14250	17800	22500	500	
2170	2470	3150	4350	4720	5850	7700	9750	13100	17200	21700	550	
2100	2400	3000	4200	4570	5550	7200	9300	12500	16400	20800	600	
2000	2320	2850	3980	4300	5300	7050	9000	12000	15650	20000	650	
1940	2170	2750	3900	4200	5100	6680	8500	11450	14900	19000	700	
1800	2050	2580	3640	3900	4750	6250	8100	10500	14000	17900	800	
1700	1940	2420	3370	3670	4500	5800	7500	10000	13100	17000	900	
1610	1810	2280	3150	3450	4230	5450	7100	9500	12200	15800	1000	
1460	1650	2090	2850	3150	3850	5000	6450	8625	11000	14300	1200	
1280	1480	1860	2550	2750	3450	4500	5700	7700	10000	12800	1500	
1100	1270	1620	2200	2400	2900	3850	4950	6600	8150	11000	2000	
900	1020	1280	1790	1950	2400	3150	4000	5250	6900	8850	3000	
770	880	1090	1530	1660	2000	2600	3400	4600	5850	7500	4000	
680	780	975	1350	1485	1820	2380	3040	4000	5180	6750	5000	
620	705	890	1225	1340	1660	2140	2740	3680	4750	6150	6000	
565	645	825	1120	1240	1530	1900	2510	3440	4350	5600	7000	
530	604	785	1060	1160	1425	1880	2380	3230	4130	5000	8000	
500	565	720	990	1070	1340	1760	2240	3000	3870	4500	9000	
465	540	684	955	1040	1275	1680	2130	2850	3680	4000	10000	

of 90 in Hazen and Williams' formula.  
more discharge with the same bore and gradient.

Allowance should be made in the bore to provide against the loss of discharging capacity due to internal incrustation of the pipe in the course of time. New cast iron pipes should be designed to carry 30 to 40 per cent more discharge than finally required in foul pipes of the same size. Pipes of lead, copper, brass and galvanized iron are little subject to corrosion, therefore, no allowance need be made forage.

Some engineers recommend the following figures to allow for reduction in carrying capacity of pipes due to age :-

Type of pipe	Q for which to design in terms of required discharge
Uncoated cast iron ..	1.55 Q
Asphalted cast iron ..	1.45 Q
Asphalted riveted wrought iron or steel ..	1.33 Q
Neat cement or concrete ..	1.06 Q

Also see Hazen and Williams' formula under 'Hydraulics.'

**Relative Discharging Capacities of Full Pipes  
or Circular Sewers (Discharge varies as  $\sqrt[3]{d^5}$  approx).**

Dia. of pipe in in.	Relative discharging power	Dia. of pipe in in.	Relative discharging power	Dia. of pipe in in.	Relative discharging power
$\frac{1}{2}$	.031	4	32	16	1024
$\frac{3}{8}$	.086	$4\frac{1}{2}$	43	17	1192
$\frac{1}{2}$	.117	5	56	18	1375
$\frac{3}{4}$	.485	$5\frac{1}{2}$	71	20	1789
1	1.0	6	88	22	2270
$1\frac{1}{4}$	1.75	7	130	24	2822
$1\frac{1}{2}$	2.76	8	181	26	3447
$1\frac{3}{4}$	4.10	9	243	28	4149
2	5.66	10	316	30	4930
$2\frac{1}{2}$	7.60	11	401	32	5793
$2\frac{3}{4}$	9.90	12	498	34	6741
$2\frac{1}{2}$	12.54	13	609	36	7776
3	15.59	14	733	38	8870
$3\frac{1}{4}$	22.92	15	871	40	10,150



**Types of Supply—***Intermittent and Constant Systems:*

When supply is stopped all the water rushes in to the low level mains from 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. 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 in the mains and for 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.

**5. PIPES OF DIFFERENT METALS**

**Choice of materials for piping.** In choosing the material of the piping account should be taken of the character of the water to be conveyed and of the nature of the ground in which the piping is to be laid.



## Cast Iron Pipes

(Laying and Jointing has been described before)

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 400 ft. of vertical head of water, and up to 30-in. dia., and above this size, the use of either steel or R.C. pipes should be considered. Diameters beyond 4 ft. are not convenient to handle and fix.

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. 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 the pipes from the corrosive action of waters.

Cast iron water pipes are manufactured in 9 ft. and 12 ft. lengths and are termed Class A, B, C, or D. Joints are either spigot and socket, turned and bored, or flanged.

Class A for test pressure of 200 ft. head.	Working pressure (exclusive of water hammer pressure) should not exceed half the test pressure.
Class B for test pressure of 400 ft. head.	
Class C for test pressure of 600 ft. head.	
Class D for test pressure of 800 ft. head.	

**Table Showing Weight of Cast Iron Pipes and the Materials Required Per Joint for Joining them.**

*Class B Spigot and Socket Pipes*

Dia. of pipe ins.	Length of pipe in ft.	Thickness of pipe in ins.	Weight of each pipe			Total depth of socket in ins.	Weight of lead per joint, lbs.	Weight of hemp per joint, lbs.	Depth of lead in ins.
			cwt.	qtr.	lbs.				
3	9	0.38	1	0	17	3	4.5	0.25	2.00
4	9	0.39	1	2	3	3	5.5	0.38	2.00
	12		1	3	25				
5	9	0.41	1	3	26	3½	7.0	0.44	2.40
	12		2	2	6				
6	9	0.43	2	1	24	3½	8.0	0.44	2.40
	12		3	0	21				
7	9	0.45	2	3	26	3½	9.5	0.50	2.40
	12		3	3	13				
8	9	0.47	3	2	11	4	12.0	0.63	2.90
	12		4	2	16				
9	9	0.49	4	0	20	4	13.5	0.69	2.90
	12		5	1	17				
10	9	0.52	4	3	14	4	14.75	0.75	2.90
	12		6	1	7				
12	9	0.57	6	0	25	4	17.0	1.06	2.90
	12		8	0	8				
14	12	0.61	10	0	11	4½	21.5	1.38	3.25
15	12	0.63	11	0	16	4½	23.00	1.50	3.25
16	12	0.65	12	0	27	4½	24.25	1.63	3.25
18	12	0.69	14	2	5	4½	31.50	2.07	3.10
20	12	0.73	17	0	3	4½	34.50	2.25	3.10
21	12	0.75	18	1	11	4½	36.25	2.38	3.10
22	12	0.77	19	3	12	5	38.25	2.50	3.60
24	12	0.80	22	1	24	5	41.25	2.66	3.60
26	12	0.83	25	0	21	5	44.50	2.84	3.50
27	12	0.85	26	2	27	5	46.75	2.94	3.50
28	12	0.86	28	0	5	5	50.25	3.06	3.50
30	12	0.89	31	0	8	5	58.25	3.20	3.40
32	12	0.92	34	0	23	5	63.00	3.40	3.40
33	12	0.94	36	0	2	5	67.50	3.65	3.25
36	12	0.98	40	3	18	5	71.50	4.15	3.25

Add 5 per cent of lead for wastage in melting. For joints made with lead wool, allow 2/3rd of the weights of lead given in the table. Weights of hemp and lead in each joint are variable and also the depth of joints. Weight of lead is 0.41 lbs/c. in., varying 15% depending on purity.



5-in., 7-in. 11-in., and 13 in. are uncommon sizes. Lengths are measured over the flanges. The usual length of pipes below 12 ins. dia is 9 ft. and above 12 ins. 12 ft.

### Standard Weights of Cast Iron Bends

Dia. of pipe	Bends						Dia. of pipe	Bends					
	90°			45°, 22½°, 11¼°				90°			45°, 22½°, 11¼°		
ins.	cwt.	qtr.	lb.	cwt.	qtr.	lb.	ins.	cwt.	qtr.	lb.	cwt.	qtr.	lb.
3	0	1	13	0	1	18	22	11	2	1	9	1	0
4	0	2	4	0	2	5	24	13	2	23	10	2	27
5	0	2	27	0	3	4	26	15	2	21	12	1	3
6	1	0	4	0	3	27	27	17	1	26	14	0	11
7	1	1	2	1	0	27	28	18	1	6	14	3	1
8	1	3	0	1	2	6	30	20	1	6	16	1	3
9	2	0	7	1	3	14	32	23	1	4	19	1	1
10	2	2	22	2	0	25	33	24	2	7	20	1	7
12	3	2	1	2	3	23	36	27	3	11	13	0	3
14	4	1	23	3	3	22	38	31	3	0	26	3	18
15	5	0	27	4	1	25	40	34	0	2	28	3	9
16	5	3	0	4	3	17	42	36	2	26	31	0	14
18	6	3	20	6	0	13	44	41	2	10	35	3	17
20	8	2	13	7	1	25	46	44	2	17	38	2	5
21	9	3	15	8	1	6	48	47	1	9	40	3	10

All pipes should be coated with Dr. Angus Smith's composition before leaving works, which is a standard treatment. The surface coating must be black with a bright gloss and thoroughly incorporated with the metal. This treatment makes the pipe rust-proof and increases its life. Care should be taken while jointing pipes that the coating is not injured.

Dr. Angus Smith's Solution is a varnish of coal tar, pitch and oil consisting of: Coal tar 112 lbs.; Paraffin wax, or tallow 7 lbs.; Quick lime 10 lbs.; Resin 4 lbs.

Water mains and specials are generally of Class B spigot and socket pipes. Cast iron pipes have also been described in the Section "Drainage and Sewerage."



**Spun Iron Pipes.** Cast iron pipes are now being superseded by spun iron pipes which are manufactured up to a diameter of 27 ins. Spun iron pipes are about three-quarters of the weight of vertically-cast pipes of the same class, the greater tensile strength of the spun iron due to the closer grain allowing the use of a thinner wall than for that of a cast iron pipe of equal strength. Inner surface is also smoother than that of the ordinary cast iron pipe.

### **Steel and Wrought Iron Pipes**

Steel pipes are generally used for long exposed rising mains especially when they are of large diameters (3 or 4 ft.), trunk mains, inverted siphons, and on bridges and other structures where strength and least weight are required, and also where pressures are high (above 100 lbs.)

Steel pipes are of three types—(i) riveted, (ii) welded and (iii) solid drawn. They have mostly flanged ends and made in long lengths (18 to 36 ft.) and have less joints and are flexible but of great strength. Field jointing may be by coupling of sleeve or split sleeve type. Special types of flexible and expansion joints are available with rubber gaskets or rings. Expansion joints are necessary for long lengths of pipe lines. Steel pipes up to 12-in. diameter are also made for direct coupling to cast iron mains of similar size. Steel pipes can be made of about  $\frac{1}{2}$  the thickness for small diameters and  $\frac{1}{3}$ rd the thickness for large diameters of cast iron pipes; consequently saving in weight, transport, pipe-laying and jointing costs. 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 form of flexible joint, which forms an additional safeguard against failure. Steel has now largely superseded wrought iron.

Owing to the difficulty of making connections or repairs to a pipe line in case of a burst, these pipes are seldom used for distribution mains. The removal of rivets and the substitution of a new pipe takes an unusually long time. A steel pipe is very much liable to the actions of acids and alkalis in water and a slight trace of these will produce rust and incrustations. The presence of rivets further

**Thickness of Steel Pipes in B.G. Gauge No.  
for Various Pressures and Diameters**

Bore of pipe ins.	Test Pressure in ft.										
	2,000	1,900	1,800	1,700	1,600	1,500	1,400	1,300	1,200	1,100	1,000
6	6	..	7	..	8	9	..	..	..	..	10
7	4	5	..	6	7	..	8	..	9	..	10
8	4	..	..	5	..	6	7	..	8	..	9
9	3	..	4	..	..	5	..	6	..	..	8
10	..	..	3	..	4	..	..	5	6	..	7
12					..	3	..	4	..	5	6
14						..	..	..	..	4	5
15							..	2	3	3	4
16								..	..	..	3
18								..	1/0	1	2
22								..	..	..	1/0
	950	900	850	800	750	700	650	600	550	500	450
14	..	5	..	..	6	..	8	..	..	10	..
15	..	..	..	5	..	6	..	..	8	..	10
16	..	4	..	..	5	..	6	..	..	8	..
18	..	..	..	4	..	5	..	..	6	..	..
20	2	..	3	..	..	4	..	5	..	6	8
21	..	2	..	3	..	4	..	5	..	6	..
22	..	..	2	..	3	..	4	..	5	..	6
24	1/0	1	..	2	..	3	..	4	..	5	..
26	..	1/0	..	1	..	2	3	..	..	..	..
27	..	1/0	..	1	..	2	..	3	..	4	..
28	..	2/0	..	1/0	1	..	2	..	3	..	4
30	..	2/0	..	..	1/0	1	..	2	..	3	..
32	..	2/0	..	..	1/0	1/0	..	2	..	..	3
36	..	2/0	..	..	..	2/0	..	1/0	..	2	..
40	..	3/0	..	..	..	2/0	..	2/0	1/0	..	..
44	..	5/0	..	..	..	3/0	..	2/0	..	1/0	..
48	..	5/0	..	..	..	3/0	..	..	2/0	..	1/0
54	..	6/0	..	..	..	5/0	..	..	3/0	..	..

Ordinary working pressure (exclusive of water hammer pressure) is half of the test pressure, but some engineers take 75 per cent for mains designed for a constant flow and not subjected particularly to shock and vibrations, and 60 per cent where mains are subjected to vibrations as in streets and under railways.



gives better nucleus to the process of rusting. Cast iron withstands rust better. Fresh water corrodes wrought iron more rapidly than cast. Therefore, thinner walls and greater susceptibility to corrosion are likely to cause high maintenance charges and shortened life. Steel pipes need frequent painting to save them from corrosion.

The weight of wrought iron or steel supply pipes shall not be less than the following :—

Bore	$\frac{1}{2}$ "	$\frac{3}{4}$ "	1"	1 $\frac{1}{2}$ "	1 $\frac{3}{4}$ "	2"	3"	4"	5"	6"
lbs./ft.	0.87	1.2	1.6	2.6	2.7	3.6	7.5	10.6	14.5	18.7

(The above weights are as usually prescribed. Some authorities give slightly different weights.)

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. Steel pipes are not suitable for under ground works under heavy fills.

Steel pipes should be stored not over four tiers high, with 1"  $\times$  12" wooden strips not over 4 ft. 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.

**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. These pipes can be lined with cement mortar; such pipes will not corrode and will retain carrying capacity indefinitely. Fittings should be malleable-iron screwed fittings.

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 G.I. pipes is easy and they are also easy to transport and mani-



ulate. G.I. pipes are generally manufactured in sizes of from  $\frac{1}{4}$ -in. to 3-in. diameter.

**Cement Lined Pipes.** Are prepared by spreading 1 : 3 cement mortar evenly on the clean inside surface of the pipe and then revolving it. The standard average thickness of this coating is  $\frac{1}{16}$ " for 6" or smaller;  $\frac{1}{8}$ " for 8" to 16"; and  $\frac{3}{16}$ " for larger pipes. The outside of the pipes receive the usual coating. Such pipes stand corrosion exceedingly well.

**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 best 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.

Where a steel pipe is to be laid in a swampy or otherwise water-logged, or soil containing deleterious earth or salts, the particular length of the pipe can be protected by first painting the steel pipe before laying with some composition as specified elsewhere 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.

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 Electrolitic 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 particularly suited to work inside buildings owing to the ease with which they can be bent to follow an irregular line. Lead or leadlined pipe 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 piping is liable to corrosion by contact with fresh cement, cement mortar, or concrete. Lead is also liable to corrosion when in direct contact with certain timbers, especially when they are damp.

Lead and lead-alloy piping should be jointed to cast-iron, wrought-iron, steel or copper piping by the use of copper-alloy screwed unions or ferrules.

*The action of water on lead is tested as follows :—*

A strip of sheet lead is scraped 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. Jointing is done by "Wiped Joint." 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.

**Copper Pipes.** Are not liable to corrosion; are easily bent and do not sag when hot. They are thus particularly suitable in interior work for both hot and cold supplies. They can be made of very thin walls and for any diameter



their weight is much less than pipes of other materials and their first cost compares favourably. Copper piping of small diameter are jointed to other pipes by copper-alloy screwed unions or ferrules.

**Cement Concrete Pipes.** A cement concrete pipe would be affected by water having  $\text{CO}_2$  gas or free acid in solution and a part of the cement of the bore might be dissolved and smoothness effected to some extent. They can either be made locally or Hume pipes of patent manufacture obtained. If properly made they are better than metal pipes in many respects as there is no rusting and incrustations inside the bore.

The action of acids and alkalies in water is not the only factor that makes the bores of pipes rough. Temporary hard waters and waters charged with sediments leave deposits inside the pipes which become hard incrustations in the course of time.

#### Thickness of Plain Concrete Pipes for Water Supply

Dia., in.	8	10	12	14	16	18	20	24	30
Thickness, in.	$1\frac{1}{2}$	$1\frac{1}{2}$	$1\frac{1}{2}$	$1\frac{1}{2}$	$1\frac{1}{2}$	$1\frac{1}{2}$	$2\frac{1}{2}$	$2\frac{1}{2}$	$2\frac{1}{2}$

**R.C.C. Pipes.** (See under "Culverts" in Section 19.) In concrete pipes of 24-in. dia. or over steel cylinders are sometimes used instead of rods or fabric reinforcement, with thin reinforcement cages both inside and outside for gripping the concrete. 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 concrete used.

**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 tenon joints with collars. Joints have been described in the Section on "Sewerage."

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 2" to ensure watertightness. Hemp soaked in cement slurry 1 : 2 is wrapped all round before a cement concrete collar is made around the joint.

Unreinforced concrete pipes can be used only for low pressures, say about 40 to 50 ft., with 100 ft. max., head of water, and reinforced pipes up to 250 ft. head of water.

**Asbestos-Cement Pipes.** This pipe is a comparative new-commer in the field and consequently its life is unknown. Advantages claimed for it are that the carrying capacity remains substantially the same as when first laid as it has a smooth inside surface and is immune from attack from corrosive soils and alkalies or acids in water and tuberculation ; it is also light in weight. It can be drilled and tapped for connections, but has not the same strength or suitability for threading as iron and any leakage at the thread will become worse as the time passes. Leaks are very difficult to detect and repair. Absestos-cement pipes are fragile and cannot stand heavy impacts and blows during handling, carriage and laying, and are unsuitable in situations subject to vibrations and subsidence. Joints are made with collars and two rubber rings.

## 6. VALVES, METERS & TAPS

**Valves.** Functions of various types of valves used in a water distribution system are described below :—

**Sluice valves** are the commonest and most important as they control the flow of water in a distribution system, and are provided at the point of connection of one main to another. Whenever a small pipe branches from a large one, the former is provided with a valve. The larger pipes need not have valves at each junction with small pipes. Valves are generally placed at street corners where lines intersect, a valve in each line at each intersection. 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 quarter of a mile if it is a long one, and

branches at 1000 to 1200 ft. Valves on the "run" of large mains should normally be half-a-mile to a mile apart, depending upon circumstances. The nominal size of a valve is generally three-quarters of that of the main; the valve may have an area equal to half that of the pipe without much loss of head. A stop-cock or sluice valve is fitted on the outlet pipe from each reservoir. Sluice valves are used on cast iron mains and screw-down stop-cocks on wrought iron pipes. Sluice valves are also called **Stop valves or Gate valves**. Each valve should be provided with a cast iron valve box, or a pit built around it; generally over 12-in. 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.

Large size of sluice valves are also often provided with small by-passes to relieve pressures on opposite sides of the gate. The following sizes are usually recommended:—

Valve size	up to 8"	9" to 12"	14" to 21"	22" to 36"	36" to 48"
By-pass	$\frac{1}{2}$ " to $\frac{3}{4}$ "	1" to $1\frac{1}{4}$ "	2" to 3"	3" to 4"	4" to 6"

**Reflux valves** are also known as non-return, check, flap or foot valves and 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. They are placed at intervals in long pumping mains to prevent back pressure on the engine and 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 permit a large loss of water by backward flow, such as at the entrance to a reservoir. **Foot Valves** are similar to reflux valves but usually placed in the vertical position on the end of suction lines of pumps.

**Air valves or Air-Relief valves** are necessary on some trunk and secondary mains at summit points, and wash-outs at lower points between summits, to allow accumulation



of air to escape when the pipes are filled, and 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. 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 quarter to half-mile 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 masonry pit for protection and a weep hole is provided for the escape of water which may pass the air-valve with the escaping air. A 2-in. air-valve will do for pipes up to 10 ins. diameter. At some places "double-acting" valves are used which have a large 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.

**Scour valves or Blow-off valves** (also called wash-outs) are controlled outlets on a pipe line provided at all depressions and dead ends to drain out the waste water or sediments collected. Scour valves are usually 3 to 6 inches diameter leading from the main from a flanged tee branch to a ditch with a sluice valve control. They should be fixed in a manner so that the inverts of the outlet branch and the main are at the same level. This valve is essentially a sluice valve. 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 downstream ends of long lengths of mains, or where



water hammer is likely to occur, to relieve excessive pressure. It is most important that the correct type and size of valve is selected for the particular line or purpose. These are heavily weighted spring controlled valves which open under pressures exceeding those for which they are set.

Valves are usually flanged to enable them to be removed, repaired, and re-inserted without disturbing the rest of the pipe line.

**Meters.** A meter should measure accurately all flows through it whether large or small, with min. loss of head, and should not clog with impurities in the water. A meter should be fitted between two stop-valves and with unions. If the meter is fixed underground, a suitable brick chamber should be built, large enough to include both meter unions, and covered with an iron surface box. If the meter is fixed in an exposed position, it should be rigidly supported in such a position as to ensure correct working of the mechanism, that is, normally with the inlet and outlet horizontal and the dial facing upwards. Consider loss of pressure head due to meters while calculating pressure in the distribution system. There are two main types of meters, positive and inferential. Inferential meters are cheap but not very accurate.

**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. Automatic direct recording arrangements are generally fitted. The meters are made for mains 3-in. diameter and upwards. The indications are not regarded as reliable for cases in which the velocity of flow is small. For small supplies the main meter on the rising main should be of the positive type.

**Water Taps**

The following sizes of bore are generally used :—

$\frac{3}{8}$ ",  $\frac{1}{2}$ ",  $\frac{5}{8}$ ",  $\frac{3}{4}$ ", 1",  $1\frac{1}{4}$ ",  $1\frac{1}{2}$ ", and 2"

All water supply fittings, with the exception of ball taps should be tested for a pressure of 300 lbs. per sq. in.

Bath taps for hot water should not be less than  $\frac{5}{8}$  in. bore and for all lavatory taps, not less than  $\frac{3}{4}$  in. bore.

Washers for cold water fittings are of specially prepared leather, and for hot water fittings, of good quality fibre.

**7. 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).

**Economic diameter of pumping mains.** The following economic diameters of pumping mains for 16 to 24 hours pumping daily are usually recommended :—

Galls. per hour	6,000	12,000	24,000	36,000	48,000	60,000	90,000	120,000	150,000	180,000
Bore of pipe	6"	9"	10"	12"	14"	15"	18"	24"	24"	26"

The velocity for economic pumping should lie between  $2\frac{1}{2}$  and  $4\frac{1}{2}$  ft. per sec. Choosing too small a diameter is more uneconomical than erring by choosing too large a diameter.

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.

$$\left. \begin{array}{l} \text{Brake H.P.} \\ \text{required} \end{array} \right\} = \frac{G \times \text{head in ft.}}{3300 \times \text{pump efficiency}}$$

$G$  = water to be raised in galls. per minute,

head = static head (positive for rising mains and negative for gravity mains) plus friction head. For losses due to friction and slip of valves, add one-third to two-thirds of the static head.

The Pump Horse Power (P.H.P.) will vary between 55 to 80 per cent of the Brake Horse Power depending on the type of pump used, and B.H.P. (in the case of steam engines) will amount to about 80 per cent. of the Indicated Horse Power. Thus, the P.H.P. may be about 44 to 64 per cent of the I.H.P.

I. H. P. = 33,000 lbs. lifted one ft. in one minute,  
= 530 c.ft. of water lifted one ft. in one minute.

**Suction Lift.** The pressure of the atmosphere will support a column of water about 34 ft. high 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 34 ft. 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 greater total suction height than 24 ft. and which too generally varies from 15 ft. to 20 ft. according to the manufacture of the pump. For design purposes it is usual to assume 16 ft. for centrifugal pumps and 20 ft. for reciprocating pumps. Suction lift is reduced approximately 1 ft. for each 1000 ft. of altitude above sea level owing to reduced atmospheric pressure. Increase of temperature also reduces suction lift. Pumps have gene-



rally to be placed as low as possible in pits in order that allowable suction lift may not be exceeded. Sometimes increase of the suction pipe size increases the suction lift. When the water is more than 24 ft. below ground (or pit) level a pump of the "deep well" type has to be used.

**Suction and Delivery Pipes.** Suction pipes should never be of lesser diameter than delivery pipes. The suction line should be made perfectly air-tight paying particular attention to all the joints to prevent air leakage and the pipe should be given a rise of at least 1 in. in 15 ft. of length (towards the pump) to prevent the formation of air pockets which seriously interfere with the flow of water. In order to keep the water in the pump whilst standing and to facilitate priming a foot valve is necessary. 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 of the pump. 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 a 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 a bell-mouth entry, an immersion of at least  $v^2/g$  is desirable. Both suction and delivery pipes should be as free from bends as possible. A check valve is placed on the discharge pipe to prevent backward surges of pressure which might injure the pump, should the pump stop. 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 the centrifugal type. The delivery valve must never be closed on a displacement pump (rotary and reciprocating type) or dangerous pressure will arise.

Leakage in suction pipes can be detected by the following methods :—

(a) Allow the water pressure from the delivery column to come back on to the suction piping while the pump is

not running ; (b) By filling the well sump with water up to the motor bed plate ; (c) By a candle flame when the pump is running.

*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.

*Total Pump Head :* Is the total static head plus friction and velocity head ( $= V^2/2g$ ) and this represents the total head for which the power exerted by the pump has to be calculated.

*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.

## 8. 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 radial 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.

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*. The advantages of direct coupled pumping sets over the belt operated sets are : They require about 25 p.c. less power for operation and there is comparatively less loss of energy and they are also cheaper for their initial cost. A common foundation is required and there is no strain on the bearing and the pump of the engine due to tension of the belt. Lesser maintenance cost with greater efficiency.

### **Types of Pumps Used in Various Classes of Water-Works Services**

1. *Deep-well ordinary* :  
Deep-well Turbine (Multi-stage Centrifugal) ; Air-lift
2. *Deep-well small* :  
Deep-well Turbine 4" min. size ; Air-lift ; Rotary ; Continuous flow Reciprocating ; Ejector Jet.
3. *Low lift* :  
Centrifugal-low head ; Centrifugal propeller ; Centrifugal mixed flow.
4. *High pressure* :  
Centrifugal, multi-stage ; Rotary ; Reciprocating.
5. *Booster* :  
Centrifugal, single or multi-stage ; Reciprocating.
6. *Fire* :  
Centrifugal (with positive priming device) ; Rotary ; Reciprocating.
7. *Small Supplies* :  
Small centrifugal ; Reciprocating ; Rotary ; Peripheral (Centrifugal) ; Hydraulic-Ram.
8. *Stand-by* :  
Centrifugal (with positive priming device) ; Reciprocating.

### **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 of Prime Mover.** The choice of plant to be provided at each installation must be judged on its merits in relation to the availability of a power or fuel supply and should be suitable for the type of pumping unit which has been selected as most suitable for the duty to be performed.

(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 at once. 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 very suitable for use with loads with wide fluctuations.

(c) *The Gasoline Engine* : Because of their high operating costs and low first costs they are generally used as standby 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 moment. 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. A.C. motors are more efficient than D.C. but are not suitable for installations in which the speed of operation varies. Care should be

taken in making selections that the motor 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 Diesel or Gasoline plant should be kept as a standby for emergency.

#### **Pumping Machinery Efficiency :**

Electric : 90 to 95 per cent.; Oil : 70 to 80 per cent.

Steam : 60 to 70 per cent.

**Efficiency Tests of Pumps.** The following points should be observed : (i) There should be no leakage of water between the pump and the delivery end. (ii) Stuffing boxes and water sealing devices should work satisfactorily with no leakage of air into the pump. (iii) The pumps should run without much of shocks, vibrations or hammering. There should be no rubbing of the rotor against the casing or neck rings, and the bearings should run cool and well lubricated. (iv) Suction sluice valve should be fully open, and there should be no obstructions in the delivery pipe.

Rapid fluctuations on the vacuum gauge or mercury column indicate air leaks in the piping or pumps.

Table showing Horse Power required for various types of Power for different ranges of Head in ft. multiplied by discharge in galls./minute :—

Head in. ft. $\times$ galls. per minute			H.P. of Motor
Type of Motor			
Petrol	Oil	Electric	
850	1300	1650	Up to 5
1000	1600	2000 ✓	" " 10
1100	1700	2150	10 to 25

#### **Fuel Required for Plants :**

*Steam :* One H.P. requires about 4 lbs. of coal per hour.

*Diesel :* At full load it will consume about 0.5 lb. of fuel per B.H.P.-hour and the semi-Diesel about 10 per cent. more.

**Pump Combinations.** It is frequently necessary in water works services to use pumps either in series or in parallel



The rule is that in series heads are added while in parallel discharges are added. In series, the principle is made use of in high buildings or high pressure zones where "booster" pumps are used to raise the water or increase pressure. Parallel operation is frequently used at city pumping plants where additional pumps are installed to discharge into the same distribution systems to augment the increased demand.

### Centrifugal Pumps

A centrifugal pump consists of a circular casing with an impeller or runner (which has a number of spiral vanes) inside it keyed on a shaft which is driven by a power unit. 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 the suction pipe, and which is whirled out and discharged at its outer periphery by centrifugal force. Multi-stage pumps comprise several impellers on one shaft in order to generate a greater head than can be obtained from a single-stage pump.

A centrifugal pump is relatively low in initial cost, is very dependable and durable, with low maintenance cost. It is simple in construction and compact, simple to operate, light in weight and occupies small floor space for its foundations. It starts quickly and is suitable for both electric and steam turbine drive. A centrifugal pump is capable of delivering large body of water as compared to its size and can deliver sandy, gritty and muddy waters without injury to the pump. Single-stage centrifugal pumps are used for the 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. Centrifugal pumps have heretofore covered a vast field of duties leaving only the high pressure low quantity duties to the reciprocating pump and similar high viscosity duties to the rotary pumps. The two most common types of centrifugal pumps are the *volute* type and the *turbine* pump. Turbine pumps have slightly higher efficiency but are costlier.



Centrifugal pumps if properly designed and installed should give efficiencies of 55 to 75 per cent. The larger pumps being more efficient. Pumps may be either horizontal or vertical spindle type according to the central shaft. The vertical type are more expensive and have shorter life.

Ordinary single-stage centrifugal pumps have lifts limited to 60 ft. (but generally 40 ft. is estimated). Multi-stage pumps are used for high lifts; usually the head is increased 100 to 150 ft. per stage. Special design single-stage pumps can be had for about 120 ft. lift. Placing appropriate number of stages side by side in the pump casing they can be made for lifts up to even 1000 ft. 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.

While purchasing a centrifugal pump, the head against which it must operate and the discharge required must be accurately specified. If there will be variations in head, which is common in pumping water in a distribution system, this should also be mentioned.

The size of a centrifugal pump is designated by the diameter in inches of the discharge pipe. The diameter of the suction pipe is  $\frac{1}{4}$  in. bigger for 1 in. pipe,  $\frac{1}{2}$  in. bigger for  $1\frac{1}{2}$  in. to 3 in. pipe than the delivery pipe, and for 4 in. and above, the size of the suction and delivery pipe is generally the same.

*Points to be Observed when Starting a Centrifugal Pump :—*

A centrifugal pump 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 cock and the delivery sluice valve are closed. Automatic priming devices also are now available (self-priming pumps). A *Foot-valve* which is a check valve placed on the end of the suction pipe is used to keep water in the pump at all times. 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 centrifugal pump should always be started and stopped with the delivery sluice valve closed. This pump can run with the delivery valve shut producing no dangerous pressure but it should not be run in this manner for longer than is necessary as it is liable to expel water and get overheated.

The outlet (discharge pipe) should be submerged so as to induce siphon action in the pipes and should be flared at the lower end so that the water will discharge at a velocity of not over 5 ft. per sec.

**Turbine Pumps** are of centrifugal type. Where the head is such that large number of stages would be necessary and where the output required is less than 1000 galls. per minute, a turbine pump is used in preference to a centrifugal pump.

Centrifugal pumps should be fixed as near the average spring (October) water level as possible. The well sump can be made 6 to 8 ft. below the spring water level to enable the pump to be raised or lowered according to the variations in the water level from season to season. This will also eliminate any difficulties in priming the pump. Where this low level is not considered suitable for dampness, vertical spindle pump can be installed. Pump is fitted at the spring level and motor installed at ground level, connected by vertical shafts.

### **Reciprocating Pumps**

Reciprocating pumps are the displacement class of pumps and are either bucket type or piston type. They are single-acting or double-acting. A *single-acting* pump draws water into the cylinder on one stroke and discharges on the return, giving an intermittent supply. In a *double-acting* pump water is drawn in and discharged on each stroke ; but there is still some variation in the flow and to remedy that *double-acting duplex* or *triplex* pumps are made.



Reciprocating pumps are subdivided into (a) Lift Pumps and (b) Force Pumps. The common lift pump consists of a pump barrel connected to a suction pump having the lower extremity at the bottom of a well from which water is to be raised. Inside the pump barrel is the bucket controlled by a piston, which works alternately upwards and downwards. In the force pump, the barrel is fitted with a solid bucket controlled by a piston rod working up and down. The piston has a valve that is open on the down-stroke and closed on the upstroke. A reciprocating pump is suitable for high heads and fluctuating loads and gives greater efficiency, up to 90 p.c., than a centrifugal pump for high lifts. But its efficiency falls off rapidly when the head is below 100 ft., when a centrifugal pump is more efficient.

Reciprocating pumps have the disadvantage of pulsating flow, first high cost, need frequent adjusting, unsuitability for sandy water, possible damage through sudden stoppage by closed valve or other obstruction. They are complicated, heavy and require large floor space. All types of reciprocating pumps are noisier.

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 24 ft. otherwise the pump will fail to raise the water.

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 passages so as to allow water to flow only in one direction. An air chamber is placed in the discharge piping close to the pump to counteract the shocks and water hammer caused by the varying rate of discharge of the pump.

#### **Rotary Pumps**

Rotary pumps 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 from the lightest to the heaviest irrespective of viscosity or volatility, 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; can work to within 1 inch of mercury pressure, or pressure up to 100 lbs. per sq. inch.

#### **The Pulsometer Steam Pump**

Besides ordinary pumping work, semi-liquids and gritty substances 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 10 ft. to 12 ft. is possible, but may be increased to 22 ft. 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 (when a moving mass of water is suddenly stopped "water hammer" is produced). The ram is used in small water supply installations where a relatively large amount of water at a moderate head is available such as, a river or a stream with rapid fall, where the pump can be installed in its bed to pump a small volume to a higher level than the supply. Water flows down an inclined pipe (called drive pipe) whose slope should not be less than 1 vertical to 8 horizontal (with 1 to 5 max.), and is generally 1 to 10. A fall of 2 to 12 ft. is required for the drive pipe. 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 times the fall ob-

tained in the drive pipe (with 125 ft. max. lift). The hydraulic ram is simple, durable, reliable and inexpensive and does not need much attention and works without fuel oil). The disadvantage being considerable wastage (of water and noise of operation).

#### **Bore-hole Pumps or Deep Well Turbine Pumps**

Bore-hole pumps are used where the ground water-level is very low and the sump for a centrifugal pump would become very deep. The bore-hole pump is lowered in the main tube-well pipe itself while the motor is located at ground level and drives a vertical shaft extending down to the pump. The type of pump now almost invariably adopted is the vertical spindle turbine pump. 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 about two turns for the air from the rising mains to escape. The cost of installation and running of a bore-hole pump is higher than an ordinary pump; they are electrically driven and quick starting. These pumps are made for diameters varying from 4 to 13 inches and the usual length of the pump is 8 to 24 inches.

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 extremely useful for testing bore-holes and for emergency supplies for other sources in which the water-level is below the maximum suction lift of 30 ft. Air lift pumps are used to lift water from deep open wells up to 200 ft. and sometimes even 500 ft. (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 sectional area of the air pipe is only  $1/7$  to  $1/5$  of the delivery tube.

No working parts are underground. The submergence of the rising main (air being supplied thereto at the bottom) should be about twice to  $2\frac{1}{2}$  times 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 or if it gets empty by chance.

The advantages of an air-lift pump are : It will produce a large volume of water from a 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 a very simple arrangement and is easily operated and maintained. But its discharge cannot be varied according to demands and only limited horizontal pumping is possible.

## 9. ESSENTIALS FOR DESIGN OF PUMPING STATIONS

Location of the pumping station needs 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 pressures 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 pressures.

(a) Adjust size of storage tanks to allow uniform rates of pumping for long periods of the day.

(b) Provide different capacity pumps 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) Design for no suction lift if possible.

(f) Provide a separate suction intake for each pump if possible. If not, provide tapering header with "Y" branches.

(g) Avoid high spots in suction pipe to prevent air entrapment.

(h) Provide flexible couplings to facilitate placing and replacing flanged pipe.

(i) Provide check valves on discharge side of each pump and gate valves on both sides of each pump.

(j) For reciprocating pumps on long suction lines an air-chamber near the pump is recommended.

(k) It is well to slope the suction lines towards the source of water supply. The slope permits draining the line and avoids air-pockets.

## 10. GROUND WATERS 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. This accumulation of water under the surface is called *ground water* or sub-soil water and the level to which this water has saturated the ground is called *water table*. 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*.

*Capillary fringe* is a belt that overlies the zone of saturation and contains capillary interstices which are filled with water, that is continuous with that in the zone of saturation. This water is held above that zone by capillary action against gravity.

*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 springs 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 subterranean 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 the 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 spots where springs are concealed. Electro-magnetic instruments are sometimes employed for locating springs.

**Water Bearing Strata.** Sands 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 contents be for the velocity of water cannot be high in fine sand. The supply of water is more reliable in an alluvial soil. Sandstone is the best water bearer and the quality of water is also very good. Gravel and boulder strata also give good discharge. Clay, although highly porous is so fine grained as to be practically impervious. Clay serves as a confining layer to other porous strata. Limestone which gives hard water is not sufficiently porous to furnish much water unless it has many fissures and faults; marbles are better than limestone rocks; slates are better than shales due to joints. Igneous rocks, gneisses and schists are generally disappointing. The quantity of supply is more in highly fissured rocks, but the fissures should be only in the upper layer of rock and the lower layer should be water-tight.

In very small size of grains the ground will not yield its water readily, due to capillarity, in the ground. Coarse-grained sands can permit a velocity of 20 to 100 ft. per day, fine sands 0.05 ft. per day, whilst in sandstones of fine texture the flow may be as low as 10 ft. per year. The



porosity of common soils and rocks is taken as follows so far as public water supplies are concerned :

- |  |                     |
|--|---------------------|
| (i) Sands and gravels of fairly uniform size and moderately compacted .. | 35 to 40 per cent.  |
| (ii) Well graded and compacted sands and gravels ..                      | 25 to 30 per cent.  |
| (iii) Sandstone ..   | 3 to 20 per cent.   |
| (iv) Chalk .. ..   | 15 to 45 per cent.  |
| (v) Limestone .. ..  | 0.5 to 15 per cent. |
| (vi) Top soils .. ..   | 35 to 65 per cent.  |
- (Percentage is of the total volume)

### 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 can be determined by measuring the elevation of the water in a series of bore-holes, enough measurements being made to determine not only the amount of slope but also the direction of greatest declivity. The direction of flow is the direction of greatest declivity. The sub-soil generally follows the topography of the surface for its flow. It is necessary to know the direction of flow in order to make thorough investigations. 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.

The actual velocity of ground water may be measured by introducing a solution of salt into one or two wells and noting the time required for its flow to the other well, the time being determined by making frequent analysis of the water. The quantity of underground flow is 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 a depression, roughly an inverted cone in shape as shown in the figure, is formed in the water table ; the base of the cone coincides with the water-table and the



apex with the pumping level. It is commonly called *Cone of Depression* and the area within which the water table is appreciably affected is termed the *area or circle of influence*. Usually a ground water has a natural slope and a flow in a definite direction. The only reliable method of determining the zone of influence is to observe the standing water levels in small diameter observation bores sunk previously for the purpose at known distances.

**Depression Head**—Is the vertical difference in height between the normal undisturbed ground water level (before pumping) and the level of the water in 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.

**Infiltration head**—The head of the underground water which makes the water flow or infiltrate through the media.

The artificial gradient (forming the cone of depression) 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, (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, whether 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.

**Wells and Bore-holes.** There is no defined demarcation between a "well" and a "bore-hole." A bore-hole may be defined as a shaft of any diameter up to 3 ft. (sometimes

up to 6 ft.). Similarly there is no defined demarcation between a "shallow well" and a "deep well". A deep well, however, derives its supply from a deep water-bearing stratum.

*An Infiltration Gallery* is a surface collector for percolating water or an underground conduit or reservoir.

*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*.

*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.

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. 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 other wells.

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 localities. Drawing water out of shallow wells will lower the water-table and might effect the adjoining structures, but deep wells give large and uniform yield.

A spacing of 200 to 600 ft. between two wells is generally necessary according to the nature of the strata and the depth of the sub-soil water level.

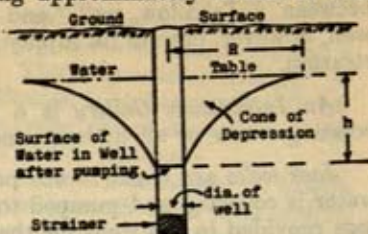
Maximum discharge can be obtained by placing the wells in line at right angles to the direction of the maximum slope of ground water-table so that almost equal heads in different wells can be secured.



Method for determining approximately radius of cone of depression :—

$$R = \sqrt{\frac{2Q}{\pi h}}$$

$R$  = radius of cone of depression in feet,



$Q$  = discharge in gallons per day (or certain period),  
 $h$  = depression of water in well during the above period in feet which is the difference of water levels inside and outside of the well. This is called percolation head, depression head, depletion head, infiltration head, or draw-down.

*Capacity of a well* is the safe maximum rate at which it will yield water.

**Size of a well :** (Approximate for Punjab Soils)

$$A = \frac{D \times 10,000}{h}$$

$$A = \text{area of well in sq. feet} = \frac{\pi d^2}{4}$$

$D$  = discharge in cusecs,  $h$  = depression head in feet.

A design formula for general use : Dia. of pipe =  $\sqrt{\frac{Q}{1000}}$

where  $Q$  = discharge in gallons per minute.

**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.

Tube-wells are not generally suitable for small discharges. Discharge from a tube-well is taken roughly equal to 10 gallons per hour per square foot of the strainer area per foot of depression head. On an average, for



estimating purposes, it is taken equal to about 500 to 600 gallons per minute, or 1.5 cusecs, that will irrigate about 300 acres. Strainers for such tube-wells are from 100 ft. to 150 ft. length located in the water bearing strata. In average sandy soils of the Gangetic plains, about 1400 to 1500 gallons per hour is considered safe extraction from an open well of 6 ft. diameter.

**Tests for Yield of a well.** By the application of certain theoretical formulae in conjunction with pumping tests, reliable information may be obtained in many cases as to the permeability of the strata.

*Yield* is the maximum amount of water that can be continuously drawn from a well with a fixed depression head.

*Specific capacity* is the rate of yield of well in c.ft. per hour under a head of one foot, or per unit draw-down. This is also sometimes called "specific yield".

*Specific yield* is the ratio of the volume of water which after being saturated, a soil will yield under gravity to the volume of the soil.

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 rises 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 than the surface spring, the pressure (and hence the supply) is poor.

**Bailer test** is a test of the rate of depression conducted while boring to estimate the probable yield from any water bearing stratum.

**Pumping Test.** When water is pumped out of a well there is a fall in the water level as the pumping progresses; this fall is proportional to the rate of discharge and the flow of water in the sub-soil. A point is reached when the water does not fall any further and the maximum critical velocity is arrived at which is not exceeded, and the yield at this depressed level is measured by the known pumping capacity or by the quantity that can be bailed or pumped out in a given time so as to maintain the level constant. This

method is rather difficult as it involves steady pumping at a fixed level and an accurate discharge of the pumps.

**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 becomes less as the water level rises up.

Test pumping should be carried out as far as possible at the time of lowest ground-water level, so that the most reliable yield figures may be obtained. 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. Tests are usually carried out by submersible-type of pumps, though the air-lift type may be used under suitable conditions.

**Trial holes.** As a preliminary to almost any scheme where more than 1 million galls. 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 6 ins. If, however, information is required as to the probable yield of permanent works, then bores of up to 24 ins. in dia. should be sunk. Samples of all changes of strata and at 10 ft. intervals throughout the full depth, should be taken and preserved, in air-tight bottles if of cohesive soils, otherwise in wooden boxes. Sulphate tests should be made on clays, and sands should be mechanically analysed.

Ground water level depends upon a combination of the permeability of the strata and the head causing the water to flow (hydraulic gradient). It is possible, according to the inclination of the different strata, for the water tables in surrounding ground to be higher or lower than the water level in streams, as the movements of the ground water are to a large extent, independent of the stream.



Maximum yield will be obtained from a well when it is worked continuously without exceeding the critical velocity. A factor of safety of 2 to 4 is usually allowed but it may have to be as high as 10.

Yield of a well can be roughly calculated from the formula :—

$$Y = \frac{h\sqrt{h}}{10} \times S^2 + \frac{D \times h}{5} \times S$$

Y=yield in galls. per hour; h=depression head in ft. when pumping is in progress; S=the length in ft. of the slope of water surface in the ground for a fall of one ft. (Take R. L. of water level at a distance of about 500 ft. from the well towards 'up-stream' side. Difference between this R. L. and the R. L. of the water surface at the time of pumping gives the fall for a length of 500 ft. Work out the length for a one ft. fall.) D=Outer dia. of the well in ft.

(Discharge from a strainer tube-well: For fuller details see page 259 of the Journal of the Institution of Engineers (India), Vol. 34, No. 2, Dec. 1953.)

The observations are to extend, not for uniform period of time but for uniform rises of levels, say for every 3 ins. 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.

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.

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 5 ft., while in U.P. 18 to 20 ft. of depression is considered suitable for tube-wells in the average sandy strata. That is the critical head which 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  $4\frac{1}{2}$ " diameter tube strainer well with that of percolation wells, it is found that one single tube of  $4\frac{1}{2}$ " dia. gives the same quantity of water as three ordinary wells each 12 ft. 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 so small a hole. The chief advantages of the 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.

The best method of measuring discharge is the V-notch method where a right angled V-notch is cut in a steel plate of about  $\frac{1}{4}$ " thickness and of sufficient size to allow the full discharge to pass through it. The method of measurement has been explained under "Hydraulics."

### CONSTRUCTION OF TUBE-WELLS

**Methods of Boring.** Boreholes may be sunk either by percussive boring or by rotary boring; each of these methods may be carried out in a number of ways.

**Rotary Boring :** This method is used for unconsolidated materials of fine texture. Drilling is accomplished by rapidly rotating a wrought iron or steel pipe fitted with a cutting shoe at the lower end. In firmer materials either a fishtail bit or a diamond-shaped bit is rotated in the well to cut and loosen the material. Water is continuously pumped into the well under pressure through holes in the cutting shoe to rise between the side of the hole and the casing carrying with it the loosened material, all formed into mud or slush. This mud plays an important part in drilling. When the bore has been drilled to the required depth casing is inserted to support its walls. Machines are used for rotating. The casing is sunk as excavation

proceeds. Rotary rigs are not found suitable in clay, but are satisfactory in sand.

The counterflush or reverse circulation method of drilling is also used in loose soils.

**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 up the rock so that it 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 Jet Boring.** Is used for boring through hard and tenacious soluble clays and for deep borings. 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 is 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.

**Percussion Rope Boring** As the name implies, entails drilling by the percussive action of a chisel on the stratum to be penetrated and is the most commonly used of all the methods by which a hole is made into the ground by reducing the strata to powder. This method can be adopted for all types of strata, but is generally confined to the drilling of relatively soft formations such as clay, sand, chalk and friable sand-stones. A pit is dug at the site about 6 to 8 ft. in diameter and 15 to 18 ft. deep. A tripod (shear legs or derrick) is erected over the site with a pulley attached at the top so that a plunger suspended from over the pulley may be exactly above the casing pipe. The legs of the tripod should be buried about 2 ft. into the ground to prevent their slipping or tilting. The leg opposite the hoist



or crab-winch must be securely buried and anchored so that the tripod will not turn over when subject to a pull from the winch. A crab-winch and a steel rope is required for working the plunger inside the boring tube. (The plunger is also called—sludger, bucket or sand-pump and has been described later.) A cutter shoe is fitted to the bottom of the boring tube (or casing tube) which is lowered into the pit. The cutter shoes are of slightly bigger diameter than the boring-tubes, therefore a clear passage is provided for the boring-tubes as they are sunk or withdrawn.

The cutter shoes are of two types, "*slip shoes*" and "*screw shoes*", and are made of tempered steel. A slip-shoe is slipped on to the boring-tube when starting boring and is left at the bottom of the bore when the tube is withdrawn. This type of shoe is used in stiff clay or rock soil and is of a slightly larger diameter than the screw-shoe for the same diameter of tube. A screw-shoe is generally used for sandy soils and for average size of tube-wells. It is screwed to the bottom end of the boring tube and comes out with the tube when it is withdrawn. This shoe has female threads to receive the screwed end of the boring-tube which fits in butt to butt with the shoe.

The boring tube is clamped with wooden sleepers 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 boring-tubes. Pipes should always be secured with additional clamps to safeguard against accidents. As the pipes are lowered down more are screwed down one after the other. After the boring tubes have been lowered in the pit and clamped in position they are partly filled with water. The plunger is worked inside the boring tube several times (it is lifted up and suddenly released so as to give it a "reciprocating" action) until it is full. 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 it is full) the plunger is taken out and the



material collected inside is removed. The spring action of the rope causes a rebound of the plunger and prevents its jamming.

Above the water-table, water has to be added during boring to assist in the formation of a "slurry". Where a chisel is used for breaking up the strata, a bailer or shell is used for removing the debris from the bore after it has been "slurried" which consists of a tube 10 ft. to 15 ft. long, of slightly smaller diameter than the bore being drilled, and fitted with a check valve near the bottom.

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 heavy 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 strainers is lowered down into the boring-tubes and kept suspended from top and the boring-tubes jacked up and extracted one by one. As the tubes are extracted the soil surrounding these gets loosened and grip the strainers 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 materials. 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.** A simple method is to make two similar discs of iron plates about  $\frac{1}{8}$ " 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 1" diameter at a distance of about 10 ft. and tightened with nuts. A knob is fixed on the top nut to which a thin steel wire is attached and the discs suspended into the tube by the wire passing over a pulley 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 vertical, 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. The 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 4 inches from the vertical per 100 ft. of boring is generally accepted.

**Setting Right Wells Sunk Out of Plumb.** A simple method is to loosen the soil on the side towards which casing pipe is to be pulled back to bring it into plumb again. The casing pipe is forced back by jacks which should be applied a few feet 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.

**Extraction of Strainers and Pipes.** Where there is an "eye" on the bail plug (as described earlier) and the pipes have not jammed, 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 loosely fitting inside the tube, which is attached with an iron bar 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 resting over the tube-well pipe and another wooden clamp is fixed outside the well pipe lower down under which jacks are worked up. By this method pull will be transmitted to the iron rod which will pull up the pipe. Special tools are also available which will expand when lowered inside the tube and grip the pipe.

### **Difficulties in Boring**

*Sand blowing.* Where fine sand is sandwiched between hard strata and is under considerable pressure, it will rush through the boring pipe and hinder the progress. The simplest remedy is to 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 cleared out from inside the bore. 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.

*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 will only jam it more. Sometimes dynamite is used



to loosen the soil around the pipe.

### **Descriptions of Boring Pipes and Shoes etc., for Tube-Well**

*Blind pipe* : 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 4 to 5 feet 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. A length of plain pipe in which there are no slots or holes.

*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 an inch are stronger than 10 threads to an inch but it requires thicker pipes. Therefore, it is preferable to have 10 threads when the thickness of the pipe is less than  $\frac{1}{4}$ " and 8 threads to an inch for thicker pipes.

*Sludger or Sand Pump* : Is a boring tool used with percussion drills, and is made of a steel pipe about  $\frac{1}{4}$ " thick. The diameter is about 2" less than the bore of the casing pipe and length varies from about 6 ft. to 10 ft. A cutting shoe of hard steel is welded to the bottom of this pipe and is slightly 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.** The size of the openings of a strainer depend upon the character of the soil encountered. The openings 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 not more than 5 ft. per sec. A velocity of 3 ft. is necessary in the tube to keep in suspension and be lifted out with water the fine sand passed by the strainers. If the velocity is low, the fine sand will settle down and is likely to gradually choke the well. Some engineers prefer to have a very low velocity, not exceeding 2 to 3 ft. per sec., so that much of sand is not moved with the water and the strainer is not clogged. In the case of a centrifugal pump the most suitable velocity of the water going to the suction pipe is considered to be about 8 to 10 ft. per second. Where this is not suitable with the sand of the strata, shrouding is done. However, 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. 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) 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.

There are various proprietary makes of strainers and are generally made of iron sheets, brass or other non-ferrous metals, about  $\frac{1}{8}$ " thick or more according to the diameter of the strainer. The section of the slots is trapezoidal, the narrow end conforming with the size of the strainer which is on the outside and in practice the width generally varies from 0.004 inch upwards and the opening is flared towards the inside to prevent packing of fine particulars in them. The screens are generally measured in thousandth of an inch.



Thus a 20 size strainer implies a size of 20/1000 inch. Where a gauze (straining material) is wrapped round the perforated tube, the straining material (gauze) should not be in direct contact with the perforated tube, as there would be a likelihood of the perforations being choked up and also of the area of the perforations being reduced but should be fixed at a distance. A strainer which has slit like openings is less likely to choke than one having woven wire mesh or small holes. Since the wells sometimes fail because of corrosion of screens, a corrosion-resistant metal should be specified. 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 generally more robust than the wire-wound type and should be used in formations where falls may be expected.

The indraw into the strainer is not uniform throughout the length of the strainer and the gross discharge does not vary directly with the length of the strainer and thus, it would be uneconomical to have a uniform diameter of the strainer throughout its length. It is, therefore, desirable to vary the diameter of the strainer retaining the optimum velocity of at least 3 ft. per second as far as possible. When there is freedom of choice as to the length of a strainer, a very satisfactory proportion is to make the length 150 times its diameter. Suitable lengths of strainers of varying diameters from 5" to 10" may be used with blind pipes in between. But where the strainers have to be extracted for some reasons, a pipe with different diameters will offer a great difficulty. For a discharge of one cusec, the size of the pipe will be about 6" to give a velocity of about 5 ft. per sec. and if the discharge is  $1\frac{1}{2}$  cusecs, the velocity will be about 8 ft./sec. A 10" diameter strainer of 130 ft. total length is sufficient to give a discharge of 1.5 cusecs with a max. depression head of 12 ft. and a velocity of about 3 ft. in the tube. It is not necessary to increase the size of the blind pipe above 6", if the strainers are of a bigger size. There will be no difference in the discharge as long as the depression is the same.



### Shrouding or Gravel Packing of Tube-Wells

Where the water bearing stratum is of uniform fine sand which is subject to flowing at the velocity the water enters the screen or consists of unconsolidated formations, a layer of coarse sand and gravel is interposed between the strainer and the soil to a thickness of at least 3 ins. This will prevent the incoming of the fine sand, increase the area of percolation and lower the velocity of the water as it leaves the aquifer. Various methods of construction are used but the commonest is to drill the well to accommodate a larger outer casing which is lowered to the bottom of the well and the strainer and the inner casing is centered into it. The outer casing is withdrawn 1 to 2 ft. at a time and gravel filled in gradually. For the first 2 ft. (at the bottom) generally coarse sand is added. The packing gravel should not be very coarse otherwise the well will not work. It should preferably be graded, of size passing through a screen of 10 meshes to the lineal inch (with max. size up to  $\frac{1}{4}$ ") and retained on a 40 mesh to an inch screen. Over-pumping is used to develop the well. The larger the diameter of the shrouding the better the result will be. Normally, for a 6-in. diameter tube-well which is to be shrouded, boring is done with casing pipes of 12 to 15 in. so as to have sufficient thickness of the gravel.

**Development of New Wells.** A tube-well draws at first a considerable quantity of the fine sand with the water which gradually clears up with pumping, but sometimes it is in such quantities that it has to be removed by bailing. Overpumping is sometimes practised to facilitate this process. Where the velocity is high it will carry some fine sand with it and some will also stick on the outside of the straining material gradually diminishing the flow. Reversed flow should be resorted to under such conditions. 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. This is known as "educating the tubewell."

**Choked Strainers : Remedial Measures :—**

Clogging of screens is the most frequent source of trouble. Excessive pumping may carry sand into the screens, or incrustations of lime may occur; this is called, (i) mechanical and (ii) chemical choking. Chemical action will deteriorate strainers by corrosion which can be very much reduced by providing a large slit area or low velocity of inflow, which will mean less depression head and reduced liberation of carbon dioxide.

Reserved motion of water called "back blowing" or "back washing" is helpful to promote flow of sand particles which have clogged the passage. (Back washing is the process of cleaning the strainer by forcing water or air under pressure from inside the tubewell, through the strainer slots into the surrounding strata.) Surging is done by raising and lowering a plunger which on the downstroke induces reverse flow forcing water outward through the screen. Air is blown into the strainer under pressure or water is pumped into the well with a pump or from a high level storage tank under pressure through a water jet tube over the strainer openings. This will dislodge fine sand or clay particles clogging the strainer holes.

Incrustations of lime can be removed by pouring sulphuric or hydrochloric acid into the well, allowing it to remain several hours and surging to dislodge the loosened material. This is called "acidification" or "acidizing". Heavy pumping should then bring up the deposits. Repetition of the above process may be needed. Where the stratum is clayey or is high in organic matter, double acid treatment is used, the first as described above and the second using commercial sulphuric acid.

Another method is to operate a turbine pump without a foot valve and then stop it, the water in the pump column will cause the back flow.

Mechanical choking (due to sand or silt) can usually be avoided if the velocity of inflow is lower than the optimum velocity capable of disturbing the materials surrounding the strainer.



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 collapse 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.

### House-hold Tube-Wells or Hand Pumps

Hand operated pumps are used where small quantities of water are to be raised from shallow wells or tanks. The common house-hold tube-well fitted with a hand-pump is called the *Abyssinian tube-well*. Such pumps will generally raise water with a suction lift of 24 feet, but it should not generally be kept more than 20 feet. These pumps generally require priming. In operation the valve in the piston is closed on the upstroke (and opened 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. There are various designs of these pumps and generally deliver water somewhat intermittently. Force pumps are generally made for raising water from 50 feet to 75 feet and do not require priming. Inside the pump barrel is the bucket controlled by a piston fixed with steel rods, generally submerged in water.

In its simple form it consists of a wrought iron strainer tube of about 4 to 8 ft. length and  $1\frac{1}{2}$  to  $2\frac{1}{2}$  inches 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 upper-most 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 30 ft. a hand pump with a 4-in. barrel and a stroke of 9 or 10-in. is convenient for one man to work, and will deliver about 5 galls. per minute.

In the case of a well for drinking the top of strainer should be 50 feet min. below ground to avoid surface pollution unless unusually impervious earth (10 feet of compact clay) occurs at the surface.

### OPEN WELLS

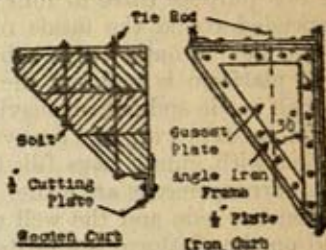
Open wells with steenings can be made economically up to a depth of 100 ft. and Kacha wells up to 20 ft. For an open well with impervious lining in alluvial soil, the maximum safe discharge is taken to be 0.15 cusec. An irrigation well should be sunk about 20 to 25 feet below the spring level so that in years of draught there is a margin of about 10 feet leaving about 10 to 15 feet 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  $\frac{3}{4}$  to  $\frac{1}{2}$  in. size at the top. These layers are about 1 ft. thick each and serve as filters so that a dangerous cavity is not formed during the years of draught.

In the case of pervious lined wells (with dry bricks or intertwined brushwood) in sandy soils, if brick-ballast of small size (about  $\frac{3}{4}$ " to 1") 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, jamba. It is of two thicknesses of wood for wells 6 ft. in dia. and under, and of three thicknesses for larger 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 across 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  $\frac{3}{8}$ " to  $\frac{1}{2}$ " 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  $\frac{3}{4}$ " dia. are fixed in the curb at about 3 to 6 feet intervals and brought up through the centre of the steening masonry. 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  $\frac{3}{8}$ " to  $\frac{1}{4}$ " thick and 3" to 6" 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.

In some places where majority of 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 commence-



ment of the steening the finished diameter of the well is 2 ft. greater than below it.

Where a well is to be built on a curb, a circular pit is dug at the site about 4 to 5 ft. 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 at least  $1\frac{1}{2} : 1$  in good soil or  $1 : 1$  in light soil cut in steps up to 6 ins. above the water table. The curb is laid and steening built up to about 8 to 10 feet above ground level. In under-sinking wells great care should be taken to keep the steening truly vertical; for this purpose three or four plumb lines should be kept suspended round the inside of the steening, and the well should not be out of plumb by more than 1 in 50 (max.).

A platform is built on the steening masonry projecting half inside and outside leaving about 5 ft. diameter space inside for removing the excavated stuff. The platform is loaded with gunny bags filled with sand or some other usual arrangements are made. If there is a hard layer of soil on one side and the well does not sink uniformly, 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. Sinking of the well should be done after allowing setting time of at least 14 days to the well masonry built above the curb.

In sandy soils sinking is expedited by bailing out the water from inside the well. Care should however, be taken to stop bailing out water 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.

**Well Linings or Steening.** The steening is usually made of brickwork or masonry. Large cement concrete rings,



2 to 3 ft. high are also used.

In alluvial soils, wells without lining are practicable only up to depths of about 18 to 20 ft.

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 8 to 10 ft. from the surface and about 3 ft. thick. Bricks with mortar are sometimes used for a distance of about 10 ft. from the ground surface with dry joints below. Where the whole of the lining is made impervious, the water can percolate into the well from the bottom only. Weep holes 6"×3" for the admission of water are left at intervals in the steening and which should be omitted in shallow wells.

### Thickness of Steening of Wells

Depth of well from ground level in ft.	Suggested thickness in ft. for diameter of well						Where high skin friction is anticipated thickness of the steening should be increased.
	Diameter of well						
	5'	10'	15'	20'	30'	40'	
10	1½	1½	1½	1½	1½	1½	
20	1½	1½	1½	1½	1½	2¼	
30	1½	1½	1½	1½	2¼	3	
40	1½	1½	1½	2¼	2½	3½	
60	1½	1½	2¼	2½	3	3½	
80	1½	2¼	2½	3			
100	2¼	2½	3	3½			
120	2¼	3	3½				

For rubble masonry steening for smaller depths, the thickness may be 3" to 6" more than the respective figures given above for brickwork steening.

*Kachha wells* without lining which are of temporary nature can be made only where the spring level is not below 10 to 15 ft. 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 5 to 6 ft. 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 well.

*Boulder well* is a cavity well through aquifer containing boulder, gravel and sand.

Discharge of Water Lifting Devices (Approx.)

	Device	Dis. in galls. per minute	Lift in ft. for which suitable
1	Persian wheel ... ..	20—60	15—50
2	Mote or Charsa ... ..	20—35	20—60
3	Rati ... ..	15—30	5—20
4	Lot, Pikotah or Denki ... ..	5—15	4—15
5	Archmedian screw .. ..	20—35	3—5
6	Doon ... ..	20—35	2—3
7	Basket ... ..	35—70	1—3

## 11. PREPARATION OF PROJECT ESTIMATES FOR WATER SUPPLY SCHEMES

(For fuller details see Section on "Estimating.")

A project estimate should give the following details in the Report :—

- (i) Reasons necessitating an improved and 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 approx. number of animals to be catered for.
- (v) Estimated daily allowance per head in gallons, and how calculated.
- (vi) Provision for future developments.
- (vii) Whether 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 " "
Meters on consumers side	..	4 to 5 "

Total cost of a water supply scheme comes to about (including interest and depreciation) Rs. 300 for big schemes to Rs. 1,000 for small schemes, per million gallons of water supplied.

Maintenance of distribution system including valves, fire hydrants and services will be about Rs. 900 to Rs. 1,800 per mile of mains annually.

Water treatment costs will be about Rs. 100 to Rs. 180 per million gallons.

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 maps may be prepared on the following scales :

Index maps of zones	..	$1''=300$ ft. or 400 ft.
Alignment	...	$1''=200$ ft.
Index maps of the whole town	...	$1''=1$ mile.
Longitudinal Sections :		
Hor.	..	$1''=200$ ft.
Ver.	..	$1''=10$ ft.

A contoured plan of the town on a scale of  $8''=1$  mile, 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  $1''=10$  ft.

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**DRAINAGE & SEWERAGE**

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## 1. SURFACE DRAINAGE OR FLOOD DRAINAGE

(Also see Sections 17, 18 and 19.)

**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 rainfall 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 sewers 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 on the ground that the damage done by such flooding does not justify the provision of large sewers 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 in inches (one hour rainfall) ;  $F$  = total rainfall of the storm in inches ;  $t$  = duration of the storm in hours.

For calculating flood discharge for the design of bridges, the severest storm ever experienced in the region is con-

sidered. For the design of minor bridges and storm water drains, it will be sufficient to take "one-hour rainfall" as 4 to 5 inches for areas of intense and prolonged rainfall and 2 to 3 inches for other areas in India. This has to be modified further with the time of concentration to arrive at the critical (or design) intensity.

The critical intensity for a catchment is that max. intensity which can occur in a time interval equal to the concentration time  $T$  of the catchment and can be found from the equation :

$$I = i \left( \frac{2}{T+1} \right) \quad I \text{ is critical intensity}$$

The highest intensity of rainfall in inches is given below for some of the towns in India, observed during recent years (as supplied by the Meteorological Deptt.) :—

Town	†Rain-fall	Town	†Rain-fall	Town	†Rain-fall
Agartala	1.55	Chikaltana	1.46	Madras	3.12
Ahmedabad	2.08	Chirrapunji	5.00	Mahabaleshwar	2.00
Aligarh	2.00	Delhi	3.12	Mangalore	1.58
Allahabad	2.86	Dhanbad	1.93	Nagpur	1.99
Amritsar	1.55	Gaya	2.40	Poona	2.10
Asansol	1.42	Hazaribagh	1.66	Port Blair	2.33
Bangalore	3.00	Hyderabad	1.72	Saugor Island	3.00
Baroda	1.49	Jagdalpur	1.45	Trivandrum	1.99
Bhopal	2.40	Jaipur	1.74	Vengurla	1.80
Bokaro	1.89	Jamshedpur	3.38	Veraval	1.00
Bombay	5.06	Jodhpur	1.96	Visakhapatam	3.00
Calcutta	2.68	Kodaikanal	2.58		

†Highest Intensity of rainfall in inches per hour.

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 point in the sewer 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 be of less intensity relative to that area than floods from a smaller catchment. A rational method has been evolved in the form of the following general equation for calculating the flood discharge of an area :—

$$Q = R \times A \times P$$

where :  $Q$  = total run-off in c.ft. per sec.;  $Q$  is expressed as c. ft per sec. for the reason that 1 in. per hour from 1 acre = 1 c. ft. per sec.  $R$  = intensity of maximum rainfall in inches per hour, based on concentration time;  $A$  = drainage area in acres contributing to run-off;  $P$  = 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 macadamized 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 .. ..	0.05—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 %

(Also see under "Water-ways for Bridges" in Section 19.)

A run-off of 1 inch per hour from 1 sq. mile = 645.33 c. ft. per second.

A run-off of 1 inch per hour from 1 acre = 1 c. ft. per second (approx.), and a run-off of  $\frac{1}{2}$  inch per hour from 1 acre =  $\frac{1}{2}$  cusec., and so on.

1 acre covered 1 inch deep = 22,650 gallons.

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.

The usual practice in the Punjab is to design storm water drains for a flood capacity of 4 cusecs per sq. mile of the catchment area in the heavy rainfall or canal irrigated tracts. In dry areas, drain capacities of 1 to 2 cusecs is considered sufficient.

For the design of branch sewers, calculation is made for each small component area of the total drainage area. The flows in the main sewers are not the total flows of the branch sewers so determined because the storm-water takes longer time to reach the main sewers than the branch sewers, and therefore the applicable rainfall intensities are smaller.

There are a number of 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.

The rate at which rain water reaches a sewer or a culvert :—

$$\left. \begin{array}{l} \text{C. ft. per sec.} \\ \text{per acre reaching sewer} \end{array} \right\} = C \times \left. \begin{array}{l} \text{Av. c. ft. of} \\ \text{rain-fall per} \\ \text{second per} \\ \text{acre, during} \\ \text{heaviest fall} \end{array} \right\} \times \frac{\left. \begin{array}{l} \text{Av. slope of} \\ \text{ground in ft.} \\ \text{per 1000 ft.} \end{array} \right\}}{\sqrt{\text{No. of acres drained}}}$$

C is impermeability factor.

## 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 or sullage and the rain water. In the separate system, two sets of sewers 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  $1\frac{1}{2}$  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 30 inches, 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 weather 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 5 to 10 gallons per head per day will seem sufficient according to the situation and the habits of the inhabitants, and availability of water.

As regards the rainwater, it is very expensive to pro-



vide 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. (Also see further under "Capacity of Sewers." at page 16/11.)

**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 cunette 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.

#### 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  $\div 2 \times 6$  (gallons 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 (26 to 53 U.S. galls.) 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.** The maximum rate of flow is usually taken twice the average peak consumption, therefore the capacity of a sewer is 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 rainwater or infiltration as the peak domestic flow is not usually more than three times the average dry weather flow. An average sewage flow of 30 gallons per capita per day (including bath and sullage water) will do for most of the towns. Extra provision for infiltration should be made at 10,000 to 40,000 gallons per mile of sewer per day, where considered necessary. 1000 U.S. gallons per inch dia. of sewer is allowed in America.

Where it is proposed to discharge part of the rainwater (from roofs, paved yards, 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.

#### *Alternative Design for Capacity of Sewers.*

Where drains or sewers have to be designed for sullage (or soil-sewage) and part of storm water (from paved areas) combined, the capacity of the sewers is made large enough to carry twice the average dry weather sewage plus the run-off due to storm water. 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 1 in. per hour, drains should be designed to carry a flow of 1 in. per hour from the area under consideration in towns and  $\frac{1}{2}$  in. per hour in country side. In medium rainfall districts, an allowance of  $\frac{1}{4}$  in. in urban and  $\frac{1}{8}$  in. per hour in rural areas may be taken. In dry districts a run-off of  $\frac{1}{4}$  in. an hour may be taken for main drains and  $\frac{1}{8}$  to  $\frac{1}{16}$  in. for branch drains.



Some engineers recommend the following run-off for the design of surface drains in town areas :—

Av. annual rainfall	up to 20"	20" to 40"	40" to 80"	80" to 120"	above 120"
Run-off per hour	$\frac{1}{8}$ "	$\frac{1}{4}$ "	$\frac{1}{2}$ "	$\frac{3}{4}$ "	1"

In town and developed areas more of the rain water 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 30 to 40 inches. The town should be divided into separate sectors or areas for the particular branch and main sewers:

Drainage area in acres	Inches of rainfall per hour	Drainage area in acres	Inches of rainfall per hour	Drainage area in acres	Inches of rainfall per hour
20	1.00	100	0.45	400	0.33
25	0.92	125	0.44	450	0.31
30	0.84	150	0.43	500	0.29
35	0.75	175	0.42	550	0.27
40	0.67	200	0.41	600	0.25
45	0.58	250	0.39	700	0.22
50	0.50	300	0.37	800	0.20
75	0.47	350	0.35	1000	0.18

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.

Scope for future developments (for about 30 years) of the area and change of habits of the 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.

**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 2 ft./sec. for large sewers and  $2\frac{1}{2}$  ft./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  $2\frac{1}{2}$  to 3 ft./sec. for open drains and  $2\frac{1}{2}$  to  $3\frac{1}{2}$  ft./sec. in sewers, to prevent deposition of grit and other solid matter. Gradient to give cleansing velocity depends upon the quantity of water flowing down the drain, its cross-section and the full capacity (see further). Greater velocities are required for storm and combined sewers than for sanitary sewers. Velocities are calculated when running full or half-full and which will fall below these figures when low flows are being carried.

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. During the hours of lowest discharge, the depth of flow in the sewer may be only a fraction of the total diameter of the pipe, and the velocity may be much lower than when the sewer flows to its full capacity. It is desirable, therefore, 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."

A sewer flowing half-full will maintain itself in good

order at a much flatter gradient than one flowing only quarter-full. Smaller sewers require greater inclination than larger ones, and stone-ware glazed pipes lesser inclination than brick sewers. Volume of flow in relation to the diameter of the pipe is an essential factor in determining velocity.

Sewers with small average flow should have the following min. gradients :—

		These gradients give	
4" dia. : 1 in 40	12" dia. : 1 in 150	} a velocity of a little less than 3 ft. per second when flowing one-quarter full.	
6" " : 1 in 70	15" " : 1 in 200		
8" " : 1 in 75	18" " : 1 in 250		
9" " : 1 in 100	21" " : 1 in 300		

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.

4½ ft. per seconds 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.

Limiting gradients for various pipes to give velocities of about 4½ ft. per second when flowing half-full :- (Required for brickwork—not very smooth surface.)

6" dia. 1 in 40	21" dia. 1 in 170
9" " 1 in 60	24" " 1 in 200
12" " 1 in 90	30" " 1 in 250
18" " 1 in 150	36" " 1 in 300

The velocity must not be more than 6 to 7 ft./sec. for brick drains, 8 ft./sec. for concrete drains, 10 ft./sec. for cemented drains and 15 ft./sec. for vitrified pipes or drains.

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.



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.

Sides of open drains may be with  $\frac{1}{2}$  to 1 slope and lined with half bricks and clay puddling where necessary, or make vertical walls (say 1 brick thick) according to the nature of the soil.

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.

**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, and where inverted siphons are constructed, it is essential that some form of flushing device be incorporated in the system. This can be done by making grooves at intervals of 150 to 200 ft. 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. Or, a chamber can be built at the head of each sewer. 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. The capacity of a flushing tank is taken one-tenth of the total cubic capacity of the length of the sewer to be flushed. The tank is made

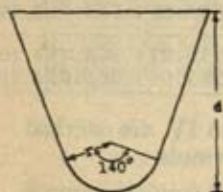


about 15 to 20 ft. high.

Velocities in sewers should not, however, be less than 1.5 to 1.8 ft. per sec. where flushing arrangements are provided.

Sometimes automatic flushing tanks are used which are located at the upper ends of the laterals, or at the heads of branch sewers. Automatic flushing devices require much maintenance and care to keep them in operating condition, and are not generally recommended. 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.



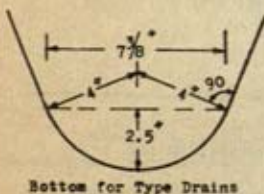
Values in feet of Hydraulic Radius (R) and Area (A) of Peg top Drains with 140° central angle

Proportion of depth d on invert to Radius r of base		Radius r of segmental portion					
		3"	3½"	4"	4½"	5"	6"
d = 2r	R	.193	.225	.257	.289	.322	.386
	A	.225	.347	.453	.572	.710	1.02
d = 3r	R	.246	.286	.327	.368	.408	.491
	A	.455	.620	.810	1.03	1.37	1.83
d = 4r	R	.293	.342	.392	.440	.490	.588
	A	.702	.960	1.35	1.58	1.95	2.28

TABLE SHOWING VELOCITY

A=area in sq. ft. ; R=hydraulic radius in ft. ;

Type of Drain	A	R	Slope 1 in—	25	50	75	100	125	150	175	200	250
I	.098	.125	V	6.2	4.32	3.55	3.1	2.77	2.52	2.33	2.18	1.95
			Q	0.61	0.43	0.35	0.30	0.27	0.25	0.23	0.21	0.19
II	.27	.21	V	8.76	6.18	5.04	4.38	3.92	3.57	3.31	3.09	2.72
			Q	2.38	1.68	1.37	1.19	1.07	0.97	0.90	0.84	0.75
III	.50	.27	V	10.36	7.3	5.95	5.18	4.63	4.23	3.91	3.66	3.27
			Q	5.22	3.68	2.99	2.61	2.32	2.13	1.97	1.84	1.65
IV	.76	.32	V	11.69	8.18	6.67	5.8	5.19	4.72	4.38	4.1	3.67
			Q	8.92	6.25	5.1	4.43	3.96	3.62	3.35	3.14	2.8
V	1.109	.37	V	11.78	8.30	6.77	5.90	5.24	4.81	4.45	4.16	3.78
			Q	13.06	9.20	7.50	6.55	5.82	5.33	4.94	4.61	4.19
V	1.45	.42	V	12.79	9.02	7.37	6.40	5.71	5.27	4.84	4.52	4.10
			Q	18.55	13.08	10.67	9.28	8.28	7.64	7.02	6.55	5.95

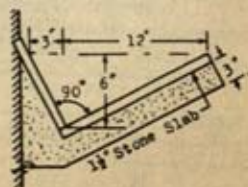
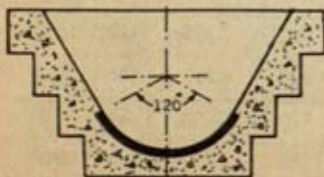


Drains Types I to IV are worked out with the formula :—

$$V = 124 \sqrt[3]{R^2 \sqrt{S}} \text{ (smooth cement concrete surface)}$$

Drains Types V and VI are worked out with the formula :—

$$V = 114 \sqrt[3]{R^2 \sqrt{S}} \text{ (brickwork in good surface condition)}$$



The above two figures show useful cross-sections.

## AND DISCHARGE IN TYPE DRAINS

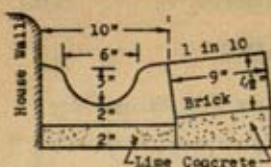
V=velocity in ft. per sec. ; Q=discharge in c. ft. per sec.

300	350	400	450	500	550	600	650	700	800	900	1000
1.78	1.65	1.54	1.45	1.38	1.31	1.26	1.20	1.17	1.09	1.02	..
0.17	0.16	0.15	0.14	0.14	0.13	0.12	0.12	0.12	0.11	0.10	..
2.53	2.34	2.19	2.06	1.95	1.86	1.79	1.70	1.66	1.55	1.44	1.38
0.69	0.64	0.60	0.56	0.53	0.51	0.49	0.46	0.45	0.42	0.39	0.37
2.99	2.77	2.59	2.43	2.32	2.20	2.11	2.01	1.96	1.83	1.71	1.64
1.51	1.40	1.30	1.22	1.17	1.11	1.06	1.01	0.99	0.92	0.86	0.82
3.35	3.10	2.90	2.73	2.59	2.46	2.37	2.25	2.19	2.05	1.89	1.83
2.56	2.37	2.22	2.08	1.98	1.88	1.81	1.72	1.67	1.57	1.44	1.39
3.40	3.17	2.94	2.79	2.63	2.51	2.40	2.31	2.23	2.10	1.95	1.86
3.77	3.51	3.26	3.09	2.92	2.78	2.66	2.56	2.47	2.33	2.16	2.06
3.69	3.44	3.20	3.03	2.86	2.74	2.61	2.51	2.42	2.28	2.13	2.02
5.35	5.00	4.64	4.39	4.15	4.00	3.78	3.64	3.51	3.31	3.09	2.93

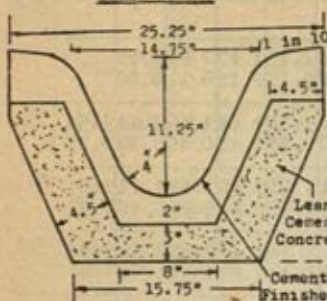
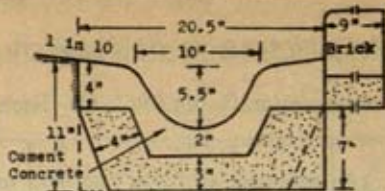
Table of Quantities for Type Drains in c. ft.  
per 100 ft. length of Drain

Type of Drain	I	II	III	IV
1. Excavation	40	157	214	272
2. Lime concrete or lean cement concrete (1: 6: 12) in foundations	14	60	82	98
3. Cement concrete (1: 2: 4) finished smooth.	32	54	62	71

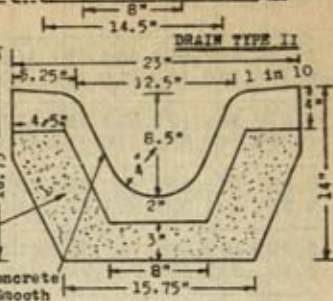




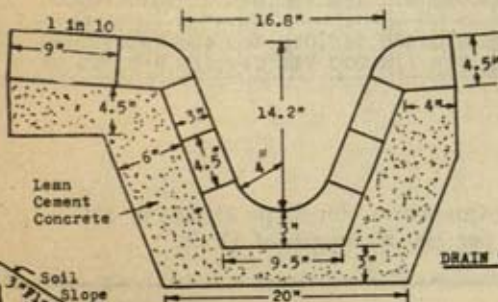
DRAIN TYPE I



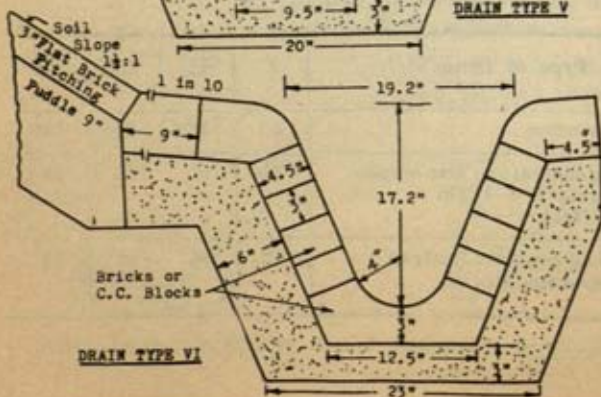
DRAIN TYPE IV



DRAIN TYPE III



DRAIN TYPE V



DRAIN TYPE VI

STANDARD TYPE DESIGN OF DRAINS

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 the Section on "Hydraulics".

**R.C. Drains.** Where it is proposed to make drains of R.C.C. the same should be not less than 2" thick and should be reinforced with 3 longitudinal bars  $\frac{1}{2}$ " diameter and 2 cross bars of the same size in 2 ft. lengths of the drains. The drains should be cast in lengths not more than 2 ft. and the moulds removed after 48 hours. They shall then be kept well watered for a fortnight and after this, watering shall 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.

## SEWERS

### Shape 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. It gives good velocity for small rates of flow and offers less opportunity for deposits but it must run at least half-full. It is the most economical section for the quantity of material required for construction and is easy to excavate and built and is fairly stable. 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 4 ft. 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 and where there is very extreme difference between the maximum and the normal rate of 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 0.95, and min. when it is  $\frac{1}{3}$ rd 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 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 designed so 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-elliptical U-shape or,



some modifications of the above mentioned cross sections to suit the site and the circumstances. In firm grounds the bottom of the trench can be excavated to conform to the shape of the invert but in soft grounds the trench bottom has to be flat unless concrete is used.

### 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 also usually possess greater absorptive qualities. 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 surface loads, or settlements due to unstable ground below, and are liable to fracture. Usual limiting size is only up to 18" diameter, with max. up to 24", and are manufactured in 2 ft. lengths, with spigot and socket ends. All drain pipes should be "salt-glazed".

### Weights of Stoneware Pipes in 2 ft. lengths

Internal dia. in inches	3	4	6	8	9	12	16	18	21	24
Weight in lbs. ..	15	20½	32	44	64	100	130	180	200	260
Min. thickness of barrel in inches	7/8	1	1 1/8	7/8	7/8	1	1 1/4	1 1/2	1 1/2	1

**Tests for Stoneware Pipes.** The breaking weight of the stoneware pipes should not be less than 1700 lbs. 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.

**Absorption Test :**

For thickness up to	1"	7 per cent. increase in weight
	1½"	8     "     "     "
	1¾"	9     "     "     "

(More details will be found under "Laying of Pipes.")

**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 24" are reinforced, the reinforcement is either a mesh or solid steel plates and made in lengths of 4 to 5 feet. (Also see 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 6-in. 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 6 ft. and 10 ft. 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, to provide for increased strength where sewer is laid at insufficient depth, 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.')

**Masonry Sewers.** Concrete is now rapidly replacing brickwork for its advantages over the latter although brick sewers are sometimes easier to construct and need less control. Brick sewers cannot conveniently be built of lesser diameter than about 30" in open trenches or 48" in tunnels. Purpose-made radiated bricks should be used for all internal curves of smaller radius than 2'-6". Bricks with frogs or other indentations should not be used in the construction of brick sewers. Bricks for sanitary works should be especially selected; they should be hard and well burnt as they have to stand erosion from moving grit, and should be least porous and absorbent. For big works bricks should be moulded to the required shapes of the sewers or drains. Egg-shaped brick sewers having small inverts should have the base extended over a larger surface to distribute the weight; this is made with cement concrete which is generally taken up for about half the height of the sewer. The inverts are generally made of cement concrete for the space forming an angle of 120-deg. with the centre of the pipe arch. A coating of plaster or asphalt is given outside the bricks. Stone masonry is not suitable for sanitary works.

### Capacity of Sewers

The carrying capacity of a sewer should be adequate but not excessive. Design of velocity should be based on the sewer flowing quarter-full. Subject to velocity considerations, it is false economy to save a few inches in the diameter of a sewer 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 lesser diameter than 6 ins. (prefer 8 ins.) even though calculations might show that a pipe of much smaller capacity would do all the work required.

**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 3 ft. 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** (Also see under "Water Supply").

Levels are taken along the centre line of the proposed sewer, say every 50 ft., 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 2-in. pegs about 100 ft., 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 3 ins. of the finished formation level, this final 3 ins. 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 6" x 2" 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 25 ft. 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 dis-

turbed ; may be about 2 ft. 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 4 ft. 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  $3" \times 1\frac{1}{2}"$  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  $18" \times 3\frac{1}{2}"$  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 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 50 ft. apart. The excavation should be boned in at least once in every 6 ft., 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 4 ft., unless the soil is very stiff. For ordinary cases with reasonably firm earth, vertical poling boards of size say, 9" x 2" can be fixed and which are supported by 9" x 3" waling boards placed horizontally 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 6" x 6" horizontal struts fixed across the trench about 6 ft. 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 9" 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 saddle-wise along the pipe line. A space of 6" to 9" 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 21" even for the smallest pipe, to facilitate working.

Where rock occurs in a trench a cushion of sand 1" 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 3" 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 6" 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 4" for pipes under 6" in dia. and 6" for pipes 6" 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 6" above its top by granular materials.

(ii) Selected granular material is tamped under and around the pipe to a height equal to  $\frac{1}{4}$ th of the diameter.

(iii) Pipe is bedded in a 6" 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 foundations, carefully shaped to fit the lower part of the pipe exterior for a width of at least 6/10th of the external diameter of the pipe and the remainder of the pipe is entirely surrounded with concrete and up to a height of at least 12" 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 care.

fully 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 20 ft. or more of cover in trenches to be surrounded with at least 6" of concrete.

(ii) Subject to (i) all pipes and tubes with over 14 ft. of cover or 18" or more diameter to be bedded on and haunched with at least 6" 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 18" diameter and with less than 14 ft. of cover may be laid without concrete, if the joints are of socket or collar type.

(iv) All pipes and tubes with less than 5 ft. of cover under roads, or 3 ft. not under roads, to be surrounded with at least 6" 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 should 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 1 lb. for each inch diameter of the pipe. One man can make about 16 joints in an 8-in. 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 9" layers thoroughly rammed. Excess of watering should be avoided. The finest material should be selected for the first one foot 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  $\frac{3}{4}$ ". A gasket of closely twisted hemp or *oakum* is used. (*Oakum* is the long, loose fibrous material obtained by untwisting and pulling old ropes.) A suitable runner should be placed around 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 support. Pipes of large diameter should be encased in concrete.

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 50 ft. 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 pre-cast concrete blocks, two to each pipe.

Cast iron pipes with spigot-and-socket joints are jointed either with a run-lead joint or with a 'fibrous-lead' (lead

wool) joint or with sulphur-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 engaged.

**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 4 ft. 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 6" 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 12" 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° access should normally be provided at the bends or junctions. The use of quarter bends (90°) should be avoided, except at the foot of vent pipes. All junctions should be oblique, and the contained angle be not more than 45°.

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 jointed 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  $\frac{1}{3}$ rd up from the invert.

Junctions made at  $45^\circ$  are called Y junctions, and junctions formed at a smaller angle than  $45^\circ$  are called V junction 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**

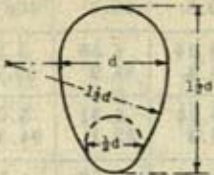
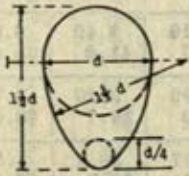

(i) *Connection to glazed-ware pipe or concrete pipe sewers.* For sewers of less than 9" diameter where it is not possible to fix a saddle, three pipes should be carefully broken out completely and replaced by two plain socketed pipes and an oblique junction pipe. Or alternatively, two pipes may be broken 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 9" 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 6" of concrete.

Breaking into the sewer should be effected by the



## Data for Egg-shaped Sewers

Type of Sewer	Depth of flow	Area	H.M.D. or R
<b>The Metropolitan Ovoid or old form.</b> Radius of invert is $\frac{1}{2}$ that of crown. 	Full	$1.1485 d^2$	$0.2897 d$
	$\frac{3}{4}$ full	$0.8795 d^2$	$0.3225 d$
	$\frac{2}{3}$ full	$0.7558 d^2$	$0.3157 d$
	$\frac{1}{2}$ full	$0.5091 d^2$	$0.2699 d$
	$\frac{1}{3}$ full	$0.2840 d^2$	$0.2066 d$
	$\frac{1}{4}$ full	$0.1862 d^2$	$0.1685 d$
<b>The New form.</b> Radius of invert is $\frac{1}{4}$ that of crown. 	Full	$1.1150 d^2$	$0.2844 d$
	$\frac{3}{4}$ full	$0.7223 d^2$	$0.3074 d$
	$\frac{1}{2}$ full	$0.2543 d^2$	$0.1920 d$
<b>Jackson's peg-top.</b> Radius of invert is $\frac{1}{4}$ that of crown 	Full	$1.0385 d^2$	$0.2680 d$
	$\frac{3}{4}$ full	$0.6458 d^2$	$0.280 d$
	$\frac{1}{2}$ full	$0.2422 d^2$	$0.190 d$

These sewers are generally built in brickwork and have transverse diameters up to 6 ft. Drains of this form though covered at the top, are technically open channels, since they are not intended to discharge under pressure.

Table of Velocity and Discharge in the Metropolitan  
Egg Shaped Sewer Running 2/3rd Full

Slope 1 in—		d					
		2'	2½'	3'	3½'	4'	5'
500	$\begin{bmatrix} V \\ Q \end{bmatrix}$	$\begin{bmatrix} 3.80 \\ 11.5 \end{bmatrix}$	$\begin{bmatrix} 4.44 \\ 21.0 \end{bmatrix}$	$\begin{bmatrix} 5.04 \\ 34.2 \end{bmatrix}$	$\begin{bmatrix} 5.59 \\ 52.0 \end{bmatrix}$	$\begin{bmatrix} 6.12 \\ 74.0 \end{bmatrix}$	$\begin{bmatrix} 7.00 \\ 134 \end{bmatrix}$
600	$\begin{bmatrix} V \\ Q \end{bmatrix}$	$\begin{bmatrix} 3.47 \\ 10.5 \end{bmatrix}$	$\begin{bmatrix} 4.05 \\ 19.1 \end{bmatrix}$	$\begin{bmatrix} 4.50 \\ 31.2 \end{bmatrix}$	$\begin{bmatrix} 5.10 \\ 47.2 \end{bmatrix}$	$\begin{bmatrix} 5.58 \\ 67.4 \end{bmatrix}$	$\begin{bmatrix} 6.47 \\ 122 \end{bmatrix}$
700	$\begin{bmatrix} V \\ Q \end{bmatrix}$	$\begin{bmatrix} 3.20 \\ 9.07 \end{bmatrix}$	$\begin{bmatrix} 3.75 \\ 17.7 \end{bmatrix}$	$\begin{bmatrix} 4.25 \\ 28.9 \end{bmatrix}$	$\begin{bmatrix} 4.72 \\ 43.6 \end{bmatrix}$	$\begin{bmatrix} 5.16 \\ 62.3 \end{bmatrix}$	$\begin{bmatrix} 5.98 \\ 113 \end{bmatrix}$
800	$\begin{bmatrix} V \\ Q \end{bmatrix}$	$\begin{bmatrix} 3.00 \\ 9.04 \end{bmatrix}$	$\begin{bmatrix} 3.50 \\ 16.5 \end{bmatrix}$	$\begin{bmatrix} 4.00 \\ 27.0 \end{bmatrix}$	$\begin{bmatrix} 4.41 \\ 40.8 \end{bmatrix}$	$\begin{bmatrix} 4.83 \\ 58.3 \end{bmatrix}$	$\begin{bmatrix} 5.59 \\ 106 \end{bmatrix}$
900	$\begin{bmatrix} V \\ Q \end{bmatrix}$	$\begin{bmatrix} 2.82 \\ 8.51 \end{bmatrix}$	$\begin{bmatrix} 3.30 \\ 15.6 \end{bmatrix}$	$\begin{bmatrix} 3.74 \\ 25.4 \end{bmatrix}$	$\begin{bmatrix} 4.16 \\ 38.4 \end{bmatrix}$	$\begin{bmatrix} 4.55 \\ 54.9 \end{bmatrix}$	$\begin{bmatrix} 5.27 \\ 99.5 \end{bmatrix}$
1000	$\begin{bmatrix} V \\ Q \end{bmatrix}$	$\begin{bmatrix} 2.67 \\ 8.07 \end{bmatrix}$	$\begin{bmatrix} 3.13 \\ 14.8 \end{bmatrix}$	$\begin{bmatrix} 3.55 \\ 24.1 \end{bmatrix}$	$\begin{bmatrix} 3.94 \\ 36.4 \end{bmatrix}$	$\begin{bmatrix} 4.31 \\ 52.1 \end{bmatrix}$	$\begin{bmatrix} 5.00 \\ 94.3 \end{bmatrix}$
1200	$\begin{bmatrix} V \\ Q \end{bmatrix}$	$\begin{bmatrix} 2.54 \\ 7.67 \end{bmatrix}$	$\begin{bmatrix} 2.98 \\ 14.0 \end{bmatrix}$	$\begin{bmatrix} 3.37 \\ 22.9 \end{bmatrix}$	$\begin{bmatrix} 3.75 \\ 34.7 \end{bmatrix}$	$\begin{bmatrix} 4.10 \\ 49.6 \end{bmatrix}$	$\begin{bmatrix} 4.76 \\ 89.9 \end{bmatrix}$
1400	$\begin{bmatrix} V \\ Q \end{bmatrix}$	$\begin{bmatrix} 2.25 \\ 6.79 \end{bmatrix}$	$\begin{bmatrix} 2.63 \\ 12.4 \end{bmatrix}$	$\begin{bmatrix} 2.99 \\ 20.3 \end{bmatrix}$	$\begin{bmatrix} 3.32 \\ 30.7 \end{bmatrix}$	$\begin{bmatrix} 3.63 \\ 43.9 \end{bmatrix}$	$\begin{bmatrix} 4.22 \\ 79.6 \end{bmatrix}$
1600	$\begin{bmatrix} V \\ Q \end{bmatrix}$	$\begin{bmatrix} 2.10 \\ 6.63 \end{bmatrix}$	$\begin{bmatrix} 2.46 \\ 11.6 \end{bmatrix}$	$\begin{bmatrix} 2.79 \\ 19.0 \end{bmatrix}$	$\begin{bmatrix} 3.10 \\ 28.7 \end{bmatrix}$	$\begin{bmatrix} 3.40 \\ 41.0 \end{bmatrix}$	$\begin{bmatrix} 3.94 \\ 74.4 \end{bmatrix}$
1800	$\begin{bmatrix} V \\ Q \end{bmatrix}$	$\begin{bmatrix} 1.97 \\ 5.96 \end{bmatrix}$	$\begin{bmatrix} 2.31 \\ 10.9 \end{bmatrix}$	$\begin{bmatrix} 2.63 \\ 17.8 \end{bmatrix}$	$\begin{bmatrix} 2.92 \\ 27.0 \end{bmatrix}$	$\begin{bmatrix} 3.20 \\ 38.7 \end{bmatrix}$	$\begin{bmatrix} 3.71 \\ 70.0 \end{bmatrix}$
2000	$\begin{bmatrix} V \\ Q \end{bmatrix}$	$\begin{bmatrix} 1.87 \\ 5.64 \end{bmatrix}$	$\begin{bmatrix} 2.19 \\ 10.3 \end{bmatrix}$	$\begin{bmatrix} 2.49 \\ 16.9 \end{bmatrix}$	$\begin{bmatrix} 2.77 \\ 25.6 \end{bmatrix}$	$\begin{bmatrix} 30.3 \\ 36.6 \end{bmatrix}$	$\begin{bmatrix} 3.52 \\ 66.4 \end{bmatrix}$
2500	$\begin{bmatrix} V \\ Q \end{bmatrix}$	$\begin{bmatrix} 1.66 \\ 5.02 \end{bmatrix}$	$\begin{bmatrix} 1.95 \\ 9.20 \end{bmatrix}$	$\begin{bmatrix} 2.22 \\ 15.1 \end{bmatrix}$	$\begin{bmatrix} 2.47 \\ 22.8 \end{bmatrix}$	$\begin{bmatrix} 2.70 \\ 32.6 \end{bmatrix}$	$\begin{bmatrix} 3.14 \\ 59.2 \end{bmatrix}$
3000	$\begin{bmatrix} V \\ Q \end{bmatrix}$	$\begin{bmatrix} 1.51 \\ 4.56 \end{bmatrix}$	$\begin{bmatrix} 1.77 \\ 8.36 \end{bmatrix}$	$\begin{bmatrix} 2.02 \\ 13.7 \end{bmatrix}$	$\begin{bmatrix} 2.24 \\ 20.7 \end{bmatrix}$	$\begin{bmatrix} 2.46 \\ 29.7 \end{bmatrix}$	$\begin{bmatrix} 2.86 \\ 54.0 \end{bmatrix}$
4000	$\begin{bmatrix} V \\ Q \end{bmatrix}$	$\begin{bmatrix} 1.29 \\ 3.91 \end{bmatrix}$	$\begin{bmatrix} 1.52 \\ 7.18 \end{bmatrix}$	$\begin{bmatrix} 1.73 \\ 11.8 \end{bmatrix}$	$\begin{bmatrix} 1.93 \\ 17.8 \end{bmatrix}$	$\begin{bmatrix} 2.12 \\ 25.6 \end{bmatrix}$	$\begin{bmatrix} 2.57 \\ 46.5 \end{bmatrix}$
5000	$\begin{bmatrix} V \\ Q \end{bmatrix}$	$\begin{bmatrix} 1.15 \\ 3.46 \end{bmatrix}$	$\begin{bmatrix} 1.35 \\ 6.36 \end{bmatrix}$	$\begin{bmatrix} 1.54 \\ 10.5 \end{bmatrix}$	$\begin{bmatrix} 1.72 \\ 15.9 \end{bmatrix}$	$\begin{bmatrix} 1.88 \\ 22.8 \end{bmatrix}$	$\begin{bmatrix} 2.20 \\ 41.5 \end{bmatrix}$

The above table is worked out with Kutter's formula with "n" = 0.013. For other values of "n" multiply the tabular numbers by K.

If n =	.01	.011	.012	.015	.017	.020
K =	1.38	1.25	1.11	0.84	0.71	0.58

When running full the velocity is about 0.94 V and the discharge about 1.43 Q. Maximum discharge occurs when the depth of the flow is 0.95, and minimum when it is one-third full.

Table Showing Gradient, Velocity, and Discharge for Circular Sewers Flowing Full

Velocity	2 ft. per sec.		2½ ft. per sec.		3 ft. per sec.		10 ft. per sec.	
Internal dia. ins.	Gradient 1 in—	Discharge c. ft. per min.	Gradient 1 in—	Discharge c. ft. per min.	Gradient 1 in—	Discharge c. ft. per min.	Gradient 1 in—	Discharge c. ft. per min.
3	96	5.9	61	7.4	43	8.8	—	—
4	140	10.5	89	13.1	62	15.7	—	—
5	186	16.5	120	20.5	84	24.5	—	—
6	240	23.6	150	29.8	106	35.6	10	115
7	295	32.1	190	40.0	130	48.3	12	159
9	415	52.9	265	66.1	184	79.2	17	261
12	610	93.9	385	118	270	141	25	464
15	810	148	520	184	364	220	33	732
18	1050	211	660	266	460	319	42	1055
21	1275	286	820	360	540	432	51	1444
24	1500	380	970	473	680	565	61	1885
27	1760	482	1100	608	790	717	73	2359
30	2050	590	1300	740	910	885	83	2930
33	2340	705	1500	889	1045	1067	94	3550
36	2600	851	1650	1069	1150	1280	105	4237
39	2885	994	1850	1249	1277	1490	120	4906
42	3200	1158	2050	1446	1425	1720	130	5743
45	3500	1331	2250	1659	1562	1985	145	6537
48	..	..	2450	1889	1700	2267	..	..
54	..	..	2875	2398	2000	2862	—	—
60	..	..	3300	2951	2300	3535	—	—

This table is based on the formula :  $V = 124 \sqrt[3]{R^2} \sqrt{S}$



Table of Velocity &amp; Discharge in Drain Pipes

Gradient 1 in—	Dia of pipe-ins.				Gradient 1 in—	Dia of pipe-ins.				
	4	6	9	12		4	6	9	12	
10	V	5.3	6.9		120	V	1.5	2.0	2.6	3.2
	D	245	718			D	71	206	612	1322
15	V	4.3	5.6	—	150	V	1.4	1.8	2.3	2.8
	D	201	590	—		D	63	186	548	1180
20	V	3.8	4.9	6.4	175	V	1.3	1.6	2.1	2.6
	D	174	510	1504		D	58	172	508	1092
30	V	3.1	4.0	5.2	200	V	1.2	1.5	2.0	2.5
	D	141	416	1230		D	55	161	424	1024
40	V	2.6	3.4	4.5	250	V	1.1	1.4	1.8	2.2
	D	122	360	1060		D	49	144	388	918
50	V	2.5	3.1	4.0	300	V	—	—	1.6	2.0
	D	109	322	946		D	—	—	388	836
60	V	2.2	2.8	3.7	350	V	—	—	1.5	1.8
	D	100	294	866		D	—	—	354	774
70	V	2.0	2.6	3.4	400	V	—	—	1.4	1.7
	D	93	272	804		D	—	—	336	724
80	V	1.8	2.4	3.2	500	V	—	—	1.3	1.6
	D	87	254	748		D	—	—	300	648
90	V	1.7	2.3	3.0	750	V	—	—	—	1.3
	D	82	240	712		D	—	—	—	528
100	V	1.6	2.2	2.8	1000	V	—	—	—	1.1
	D	78	228	674		D	—	—	—	458

V-is velocity in ft. per second when flowing quarter-full;

D-is discharge in gallons per minute when flowing full.

The table is based on the formula  $V=124\sqrt[3]{R^2/S}$ .

Flow in Slimy Sewers according to Barnes' Formula.  
For general use in determining the flow in well used  
sewers of all materials.

Dia. ins.	Velocity in feet per second													
	1	1½	2	2½	3	3½	4	4½	5	6	7	8	9	10
	Gradient 1 in—													
4	350	145	90	58	40	29	22	18	14	10	7	5½	4½	3½
5	470	210	120	78	54	40	30	24	20	13	10	7½	6	5
6	610	270	151	100	70	50	38	30	25	18	13	10	8	6
7	760	348	190	125	88	64	48	38	31	22	17	12	10	8
8	920	420	230	150	110	78	58	46	37	26	20	15	12	9½
9	1100	500	270	170	125	90	70	55	45	30	23	18	13	11
10	1300	570	310	210	140	110	80	63	50	35	26	20	16	13
12	1700	720	400	260	180	140	100	80	68	45	34	25	21	17
15	2200	1000	570	350	250	180	140	110	90	62	46	35	28	22
18	2800	1300	720	460	325	240	180	140	120	80	60	45	37	29
21	3500	1600	900	590	400	300	225	180	140	100	74	58	45	37
24	4200	1800	1200	700	490	350	270	220	175	120	90	70	55	44
27	5000	2350	1300	810	590	430	320	252	205	140	105	80	64	52
30	5850	2650	1500	980	680	495	375	300	240	170	125	94	76	60
33	6600	3000	1700	1150	780	580	430	340	225	190	140	110	86	69
36	7550	3450	1950	1350	855	635	485	385	305	210	160	120	98	78
39		3800	2220	1400	960	700	540	440	350	245	170	140	110	88
42		4200	2400	1450	1150	800	600	480	395	260	200	150	125	98
48		5000	2850	1850	1350	940	700	610	460	320	235	180	145	115
54		6000	3350	2200	1500	1150	850	690	550	270	270	210	170	135
60		7000	3950	2500	1700	1350	1000	800	640	450	325	250	200	160

Gradient is measured along the length of the pipe and not horizontally from end to end of pipe.





## in Drain Pipes

Diameter of pipe in inches											
18	21	24	27	30	33	36	39	42	48	54	60
Gradient 1 in—											
Discharge in c. ft. per min.											
1,500		68	140	245	400	640	1000	1500	3000	5800	10100
2,000			75	135	220	350	545	810	1550	3020	5800
2,500				87	145	225	350	512	1100	2100	3500
3,000				60	100	155	250	365	740	1450	2500
3,500					72	118	180	270	530	1100	1800
4,000						88	135	200	400	800	1400
4,500							112	160	320	630	1150
5,000							88	128	265	510	910
6,000									180	350	640
7,000									130	250	450
8,000										195	380
9,000										155	280
1550	3500										
810	2000	4000									
520	1300	2550	5000								
370	900	1800	3480								
270	640	1300	2500	4500							
210	500	1000	2000	3500	5800						
165	400	800	1500	2750	4500	7000					
135	300	640	1250	2250	3500	5750					
90	220	450	810	1500	2500	4000	6000	9300			
68	160	325	610	1150	1800	2950	4500	6850			
50	120	250	470	900	1450	2250	3500	5100	1100		
40	100	200	375	700	1150	1800	2750	4250	8600		
33	76	160	300	550	900	1400	2300	3300	6550	13000	

Value of C in Bazin's Formula :  $V = C\sqrt{RS}$   
(for Drains and Channels)

R	TYPE OF SURFACE				R	TYPE OF SURFACE			
	A <sub>1</sub>	A <sub>2</sub>	A <sub>3</sub>	A <sub>4</sub>		A <sub>1</sub>	A <sub>2</sub>	A <sub>3</sub>	A <sub>4</sub>
.01	45	27	13	5.3	.60	137	112	76	39
.02	62	37	17	7.6	.65	138	113	78	40
.03	76	45	22	9.5	.70	138	114	80	41
.04	80	50	25	10	.75	139	115	81	43
.05	87	56	28	12	.80	140	116	82	44
.06	91	60	31	13	.85	140	117	83	45
.07	96	63	33	14	.90	140	117	85	46
.08	100	68	35	16	.95	141	118	86	47
.09	102	70	35	16	1.00	141	119	87	49
.10	105	72	38	17	1.25	142	121	91	52
.11	107	74	41	18	1.50	143	122	96	56
.125	111	78	43	19	1.75	144	124	97	59
.14	115	82	45	20	2.00	144	125	99	62
.15	115	83	46	20	2.25	144	125	100	64
.16	116	84	47	21	2.50	145	126	102	67
.17	118	86	49	22	2.75	145	127	103	69
.18	120	88	50	22	3.00	145	127	104	70
.20	121	89	52	23	3.25	146	127	105	72
.22	122	92	54	24	3.50	146	127	105	74
.25	125	95	57	26	3.75	146	127	106	75
.27	127	97	59	27	4.00	146	128	107	76
.30	129	99	61	28	5.00	146	128	109	80
.35	131	102	64	30	6.00	147	129	110	84
.40	133	106	67	32	8.00	147	130	111	88
.45	134	107	70	34	10.0	147	130	112	91
.50	135	109	72	36	15.0	147	130	114	96
.55	136	110	74	37	20.0	147	131	114	98

A<sub>1</sub>—is for smooth finished, fine plastered or glazed surfaces.

A<sub>2</sub>—is for brick-work, cut stone, smooth concrete surfaces.

A<sub>3</sub>—is for rubble masonry or concrete work not very smooth.

A<sub>4</sub>—is for earthen channels in good order.

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 9-in. 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 4-in. private sewer need not be less than four, and as many as twenty have been so connected satisfactorily. For more than twenty houses the pipe should generally be increased to 6-in. diameter.

### 3. MANHOLES

Manholes 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.

**Spacing of Manholes.** 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 300 to 500 ft. on straight runs for sewers of 2 ft. to 4 ft. diameters. A rodding eye may serve the purpose of a manhole at the head of a shallow drain. On sewers which a man can enter for inspection, it is not essential to have a manhole at every change in alignment, but manholes should be built at tangent-points. A spacing of 600 to 800 ft. should be allowed on straight runs for sewers of 4 ft. to 6 ft. diameters which may be increased to 1000 to 1200 ft. for sewers of over 6 ft. diameter. A spacing allowance of 100 ft. per ft. of diameter of sewer is a general rule. On economic grounds, a lamp-hole may be substituted between manholes in lengths which have frequent changes of direction. Where silt and grit loads are unusually heavy, catch-pits with by-passes should be provided at every 1500 to 2000 ft., for facilitating maintenance.



**Size of Manholes.** The minimum size of rectangular manholes having not more than two branch channels on either side and up to 6 ft. depth is  $3'4\frac{1}{2}" \times 2'3"$ . For deeper manholes, it may be  $4'6" \times 2'7\frac{1}{2}"$  to  $3'9"$  up to a depth of  $4'6"$  to  $5'6"$  above the bottom, the upper portion being made into a shaft up to the ground level. The minimum internal dimension of the access shaft in a deep manhole should be not less than  $2'3"$  square where step irons are used and  $2'7\frac{1}{2}" \times 2'3"$  in shafts containing vertical ladders. A circular manhole may be  $3'6"$  dia. straight down ; a circular form is stronger than a rectangular form. Circular shafts, where provided, should be not less than  $2'3"$  internal diameter.

For big depths, the lower (bigger) portion is made up to a height of 6 ft. 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. For reducing the chamber opening the walls are either corbelled in or a roof (an arch or a slab) provided over the lower portion.

The sizes given above are the minimum and should be suitably increased according to the directions and the number of drains meeting, by 12 ins. 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 4 ft. on the line of the sewer and  $2'6"$  across.

Except in manholes not liable to super-imposed loads, all pipes exceeding 6" diameter passing through the manhole walls should have an arch formed over them in order to relieve the pipe of the weight of the wall above.

A shallow manhole should be covered by a stone or concrete slab having a hole about  $2'3"$  square in it. The cover frame should be bedded on three courses of brickwork, corbelled over to form an opening  $1'10"$  square. In the case of a deep manhole the access shaft should be brought up to a suitable level to allow a cast-iron manhole cover and frame to be bedded on the top, the cover being at road level.

Manholes can be built with 9" brick walls 1 : 4 cement mortar up to 10 ft. depth, with 6" of cement concrete (1 : 8,

or 1 :  $2\frac{1}{2}$  : 6) at the bottom projecting 6" beyond walls. For additional depths,  $4\frac{1}{2}$ " wall thickness should be added for every 6 ft. Excavations for manholes and other accessories should have 12" min. and 24" max. clearance on all sides.

The bottom of the manhole should be "benched" to have a fall towards the invert of about 1 in 10, prefer 1 in 6, to avoid fouling. 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. 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 stoneware glazed semi-circular pipes, or channels of cement concrete plastered with 1" 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. All channels in the manholes should be given a min. slope of 1 in 30.

At a junction the tops of all the sewers should be at the same level, the soffit of the smaller sewer should be not lower than the larger in order to avoid the surcharging of the former when the latter is running full. The actual gradient of the last length of the smaller sewer may be steepened sufficiently to reduce the difference of invert level at the point of junction to a convenient amount.

When a branch sewer enters a manhole at a level more than 3 ft. above the main sewer with which it has to join, the contents of the branch sewer should not be allowed to drop in vertically from a height but should be brought into it through a vertical pipe carried down from the branch sewer to the bottom of the manhole and which is fixed just outside the wall of the manhole. The branch pipe itself is also carried straight into the manhole for inspection and cleaning. It is advisable that some form of water cushion be provided to lessen the impact of the falling column of sewage. If the drop is less than about 3 ft., the drop pipe can be laid at an angle of  $45^\circ$ , but an inclined pipe arrangement is more expensive. This is called a *dropmanhole*.



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 before. 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.

The inside of the manholes is cement plastered  $\frac{1}{2}$ ", 1 : 3, up to corbelling. Where the depth of the invert exceeds 3 ft. below the surface of the ground, steps inside the manhole are necessary. These are called "treads" or "rungs" and are made of galvanized malleable cast iron or wrought iron. Steps should be built into the brickwork every fourth course (12 to 15 ins. vertical intervals), and staggered in two vertical runs which should be 9 ins. centres horizontally. In small manholes, the foot irons may be built across the corners.

The rungs should have a min. length of 10" and be at least  $\frac{3}{4}$ " in diameter or equivalent section. The top step is  $1\frac{1}{2}$  ft. below the manhole cover and the lowest not more than 1 ft. above the benching. For depths over 15 ft., a galvanized wrought iron ladder is preferable with stringers not less than  $2\frac{1}{2}$ " by  $\frac{1}{2}$ ". Ladders should not be less than 12" between stringers and the rungs at 10" centres and should be provided with brackets built into the brickwork. The minimum distance of the ladder from the wall should be  $4\frac{1}{2}$ " at the top (bent to a radius of 6"), which should be increased to 6" or more at the bottom.

All manholes on sewers of 3 ft. dia. and over should be provided with galvanized wrought iron safety chains on the downstream side. Galvanized pipe handrails of  $1\frac{1}{2}$ " bore should be provided on the edges of all benchings from which a man might possibly fall into the sewer.

**Manhole Covers.** Manhole covers are made of cast iron with double seal frames. The top surface of the covers is made rough or chequered. The following weights may be specified :—

*For the heaviest city traffic :* Circular frame 36" outside diameter, 21" inside diameter ; height of frame 9". Weight



not less than  $4\frac{1}{2}$  cwts. (frame and cover.)

*For lighter city traffic :* Same size as above except the height of frame which is 7". Weight to be not less than 4 cwts.

*For suburban traffic :* The weight may be 3 to  $3\frac{1}{2}$  cwts.

For foot traffic, the weight may be only about 150 lbs. It may be circular of 20" diameter or rectangular of  $24" \times 18"$  size.

Covers should have clear openings of not less than 20" diameter, preferably 22"

Ventilating manhole covers should not be used for domestic drains.

Circular form is the strongest in proportion to the weight of metal.

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.

**Lampholes.** When the length between two manholes on a straight run of sewer is more than usually allowed, or at places of slight change of direction, a vertical pipe about 9" in diameter is provided from the top of the sewer to the ground surface and covered with an iron grating. It forms a  $\perp$  junction with the sewer pipe. A lamp can be lowered down this pipe and the sewer inspected on the manholes on either side. Lampholes are not now commonly used.

**Ventilation of Street Sewers.** Ventilation of a sewerage system is essential for small flows and concentrated sewage in high temperatures to remove dangerous, explosive and foul gases and to provide means for the escape of air and free flow as the sewers fill, and to prevent the breaking of the seal of traps on the fittings in buildings (see under "Traps".) Ventilating shafts are provided at summits of sewers and on sewer lines at 800 to 1000 ft. apart near manholes and connected with them and which should be at least 200 ft. away from residential buildings. A ventilating shaft is a vertical column of R.C. (or iron), about

25 to 30 ft. in height and about 4 to 6 ins. in diameter (opening) at the top, tapering to 15 ins. (outside) towards the bottom for stability. The shaft is provided with a cowl or fitted with a wire guard at the top. Sometimes small vent shafts of 4 ins. diameter are provided about 400 ft. 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 the 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, hotels, etc., for which grease traps are provided.

**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 1" to  $1\frac{1}{2}$ "  $\times$   $\frac{1}{2}$ ";
- (ii) Bamboos (usually halved) or cane rods ;
- (iii) 3 or 4 ft. long rods of hard cane, bamboo, or some flexible wood, about  $1\frac{1}{2}$ " 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 200 ft., but 150 ft. is



about the limit for good practical work.

(iv) A double disc made of two circular pieces of wood held about a foot 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.

#### 4. HOUSE DRAINAGE

**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 2" for inlet pipes up to 3" dia. and 3" water-seal for 4" dia. pipes. (Also see further).

Although the primary object of a trap is to check the flow of foul air, but some of the gases which are found in the drains can be slowly absorbed by the water in the trap and given off on the other side. Therefore, the traps should be open to the outer air on the upper side. It is important that the air in the drains should be kept fresh by a well arranged system of ventilation.

Inspection Chambers are miniature form of manholes



and should be provided at every change of direction or gradient, or at 100 ft. intervals in straight lines. An inspection chamber should also be provided at the point where the vertical soil pipe joins the house drain. They are for inspection or cleaning. Min. size can be 2'-6"  $\times$  2'-0"; the longer dimension is in the direction of the main drain line. Walls can be 9" thick, cement plastered.

**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 4" in diameter and about 10 ft. 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 w.c. and bath-room. The soil pipes are taken up at least 4 ft. above the roof and which should be 15 ft. higher than the highest window of any house within 50 ft. radius. All such pipes should be provided with cowl ventilators at the top.

**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 4". 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 help.

**Gully or Gulley Traps.** 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. 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 gullies or gully traps: Those that are self-cleansing, and those that retain deposit, or catch-pit gullies. The former should always be used for sewage, and the latter for surface 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  $\frac{3}{4}$ " 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 sewage 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—6 ft.  $\times$  2 ft. or  $5\frac{1}{2}$  ft.  $\times$  1 ft. (inside) and about 1 ft.-6 ins. deep, according to the flow. A galvanized perforated loose tray should be fitted in 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 4" to 6" above the floor and the out-flow pipe which is fixed at the water level, should bend downwards inside the tank about



6 ins. above the bottom.

**Anti-Siphonage Pipe or Vent Pipe.** When water-closets of more than one floor are connected with the same soil pipe, an anti-siphonage pipe is fixed 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 2 inches and be connected with the arm of the soil pipe at a point not less than three and not more than twelve inches 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 seven feet above the highest connection of any fitting to the soil pipe.

**One and Two pipe Systems.** In the "one pipe" system the various discharge pipes from closets, sinks, baths, etc., are all connected with a common down-pipe, the waste fitments are all provided with deep seal traps. This combination ensures economy in cost in connection with high buildings where the sanitary arrangements can be grouped, but for smaller buildings where the sanitary arrangements cannot be grouped, it has not the same advantage. In the "two-pipe" system, soil and waste are discharged to separate stack-pipes and before the waste pipe is connected to the soil pipe a gully or disconnecting trap is provided to prevent foul air from the soil pipe passing through the waste drain and stack-pipe.

**Sanitary Appliances.** Appliances fitted to a drainage system for the collection and discharge of foul or waste matter.

**Sinks.** Are made in great variety of sizes, according to the purpose for which required. Max. size for a hotel sink is 63"  $\times$  21". Height of a sink to the top of the front edge is 2'-10" to 3'-0", and that of a draining board 3'-10", above floor level. Kitchen sinks are about 18"  $\times$  18",



24" × 24", 16" × 24", 20" × 30", etc., with depth of 10". In food preparation rooms, canteens, hospitals, sinks should be fixed 3 ins. clear of the walls to facilitate cleaning.

**Baths.** Usual dimensions :

Length	.. .. .	60" to 72"
Overall width	.. .. .	28" to 34"
Depth inside at waste	.. .. .	17" to 18"
Height overall, with feet	.. .. .	23" to 24"

Supply pipe to have a min. diameter of  $\frac{3}{4}$ " and the waste pipe  $1\frac{1}{2}$ " to 2". It should have an overflow pipe discharging into free air and a seal trap. It is connected with the anti-siphonage pipe.

Showers for women should be fitted with shoulder height spray heads.

Minimum size of an Indian type *Bath Room* is :  
4'-6" × 3'-6" ; prefer at least 20 sq. ft. Ventilator 2 sq. ft.

**Water-closets.** In residential buildings water-closet rooms should be located according to the direction of the prevailing winds and should preferably be made against external back walls. w.c. 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 w.c. 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 minimum dimensions of a water-closet room are :—

3'-4 $\frac{1}{2}$ " wide by 4'-6" long along the length of the pan where the doors open outside. The width may be reduced to 3'-0" where a pedestal type of closet is installed. The smallest space permitted for a water-closet compartment, where doors open out, is 2'-6" by 3'-6". If the doors open in, the compartment should be 3' × 5'. A good ordinary size is 4' × 6'. The height of the room should be at least 7'-6" and it should have a window on

the outside of not less than 3 sq. ft. area and also should have 3 sq. ft. of ventilation in addition. The walls of closet rooms should be cement plastered for a height of at least 3 ft. above the floor.

#### Sizes of water-closet pans :

Commode, pedestal or hopper type : overall height—18".  
overall length—20" to 25".

Pans are made 10", 12", and 14". high for children of various ages.

#### Indian type

Top is flush with the floor level.

..length at top—23" to 27"  
width at top—9" to 11"  
over-all height—16" to 21"  
(including trap)

The pan, (usually of fireclay) has an S-trap with outlet pointing vertically downwards for ground floor use, or a P-trap with inclined outlet for upstairs use.

#### Types of Water closets

*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 2 to 3 inches 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.

**Testing closets.** This can be easily done by throwing into the pan two or three small apples and cork bungs, and some small pieces of paper crumpled up. If the discharge of the flushing apparatus carries all away right through the trap, the pattern is so far a good one. The efficiency of the flushing rim can be tested by powdering the sides of the pan above the water-line with lamp black.

**Flushing Cisterns for Water-closets.** Capacity is 2 to 3 gallons. (In America flushing tanks capacity is as high as 5 to 8 U.S. gallons.)

Height  $6\frac{1}{2}$  ft., from top of pan to bottom of cistern.

Water is brought in by  $\frac{1}{2}$ " pipe to the flushing cistern which is stopped by ball-cock when the cistern is full. Flushing pipe should not be less than  $1\frac{1}{2}$ " dia. ( $1\frac{1}{4}$ " min.) and preferably of lead weighing not less than 12 lbs. per yard for  $1\frac{1}{2}$ " pipe and 9 lbs. per yard for  $1\frac{1}{4}$ " pipe. The siphonic water-closets generally have the flushing cisterns a little above the seats and not high up. A flushing cistern should not be less than  $3/16$ " thick if of cast iron, and not less than 12-I.S.W.G. thick if of wrought iron.

The flushing cistern should have an overflow at least one size larger than the supply pipe, with a minimum internal diameter of  $\frac{3}{4}$ ". 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 w.c., urinal or slop sink, etc.

A reserve water tank of closed type, of not less than 75 gallons 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  $\frac{3}{4}$ " 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 this 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 in about ten seconds.

**Urinals.** Height of a porcelain urinal stall bowl type above foot level is 2'-6".

### Urinals in ranges

Centre to centre of partitions ..	24" to 27"
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(Prefer greater distance so that the users do not soil their clothes)

Depth of partitions ..	20" to 22"
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Depth of end partitions .. ..	24"
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Height of partitions .. ..	54" to 66"
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Height of screen or partition where desired to be fixed above floor ..	26"
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Depth of bottom slab .. ..	24"
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The average spacing of urinals in ranges is generally taken 2 ft. per stall.

Slab type, and Channel or Concave back type urinals are generally used for public places.

The foot rests or treads should be of the non-slippery type and slightly sloping towards the channels.

Any metal gratings in connection with urinals should be of gun-metal.

Ordinary brick walls cement plastered are not suitable for urinals. White glazed bricks 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 shall is 2" 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 14 ft. run of channel). The maximum run of channel in any one fall should not exceed 7 ft.

**Flushing Cisterns for Urinals.** A cistern of one gallon capacity is sufficient for a single stall. For public urinals in single stalls or ranges, automatic flushing siphons of 1 to 3 gallons (or more) capacity, should be installed. The usual allowance of water for each stall in a public urinal is  $\frac{1}{4}$  gall. 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 w.c. or a pan, except through proper flushing tank.

#### Lavatory Basins :

Height of top above floor level .. 31" to 33"

Projection from wall .. 18" to 20"

Usual sizes are 22"  $\times$  16" or 25"  $\times$  18"

When basins are fixed in ranges, there should be a space of at least 3 ins. between the basins.

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 3 ft. 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



5 ft. before entering the vented line, or where more than one fixture is connected to an unvented branch line.

**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	4"
Main waste pipes	2" to 3"

### Branch Soil and Waste pipes :

For each w.c.	4"	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.
„ slop sink	3"	
„ urinal	2"	
For bath tubs	2"	
„ kitchen sinks	2"	
„ laundry tubs	2"	
„ wash basins	1½"	

Galvanized and black wrought iron pipes should not be used for soil, waste, anti-siphonage, ventilating or drain pipes.

### Vent pipes : (main)

For w. cs.	..	..	3"
For other fixtures	..	..	2"

### Branch vent pipes :

For each w.c. or slop sink	..	..	2"
For other fixtures	..	..	1½"

### Sizes of Traps : (min.)

For w.cs...	..	..	3"
„ slop sink	..	..	3"
„ kitchen sink	..	..	2"
„ wash tubs	..	..	2"
„ urinals ..	..	..	2"
„ wash basin & pantry sink	..	..	1½"



*Flushing Pipes*

Height of cistern above pan ...	2'	4'	8'	12'
Diameter of flush pipe ...	2½"	2"	1½"	1¼"
Diameter of flush valve ...	2"	1½"	1½"	1¼"

(For urinals, the diameter of the flush valve is  $\frac{1}{8}$ " smaller)

**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 6 ft. 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.** 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 6 ft. 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 10 ft. 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 and proper test commenced. Permissible drops in a 6" funnel per 100 ft. length of pipe line are as under :—

4" to 6" pipe	$\frac{1}{4}$ "	Any leakage will be visible. 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.
9"	$\frac{1}{2}$ "	
12"	1"	
18"	2"	
21"	3"	
24"	6"	
30"	6"	

It is considered satisfactory by some authorities if the water level does not fall more than  $\frac{1}{2}$ " in a length of 300 ft.

The water put in the pipes for testing should not be drained out until the trenches have been filled in about 3 ft. 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.** To 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 C.I. 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 20 or 24 ft. 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 20 ft. 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 100 ft. to 300 ft. 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 in 2, 4 and 6 ft. lengths, with or without ears for fixing. The pipes should be blocked out at least 1' from the walls and securely fixed by means of short pieces of  $\frac{3}{4}$ " 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 3" 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."

#### Cast Iron Soil Pipes

Bore of pipe	Thickness of metal (not less than)		Weight in lbs. per 6 ft. length (inclg. socket)	
	A	B	A	B
2"	..	3/16"	..	28
3"	5/16"	3/16"	110	44
4"	3/8"	3/16"	160	54
5"	3/8"	1/4"	190	69
6"	3/8"	1/4"	230	84

"A" pipes are for use underground and "B" pipes for use above ground



## Weight of Lead and Gasket in lbs. for Cast Iron Soil Pipes

Dia. of pipe	2"	3"	4"	6"	8"	10"	12"	14"	16"	18"	20"
Lead-lbs.	2.5	3.5	4.5	6.5	9	13	15	18	22	26	33
Gasket-lbs.	.125	.170	.170	.200	.200	.250	.250	.375	.500	.500	.625

## Light Rainwater Pipes

Nominal size	2"	2½"	3"	3½"	4"
Internal dia.	1½"	2½"	2½"	3½"	3½"
Weight per 6 ft. length, lbs.	17	19	23	28	30

These pipes have "ears" for fixing them against walls.

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 ¼" to the foot for pipes 4" or larger in size, and of ½" for smaller sizes.

## Weight of Brass Pipes

Bore of pipe	1½"	2"	3"	4"
Wt. in lbs. per ft.	2.84	3.82	7.92	11.29

**Lead Pipes.** Should be used only for short branches of soil, waste or vent connections. The following weights are according to the British Standard Specifications :

## Weight of Flushing and Warning Lead Pipes in lbs.

Bore of pipe	½"	¾"	1"	1½"	1½"	2"
Min. wt./yd. lbs.	2.5	4	5.5	7.5	10	13

## Weight of Soil, Waste and Ventilating Lead Pipes in lbs.

Bore of pipe	1½"	1½"	2"	2½"	3"	3½"	4"	4½"	5"	6"
Min. wt./yd. lbs.	7	9	12	14.4	17.1	20	22.8	29.1	41	57

## SANITARY LATRINES

**Bore-hole or Earth-pit Latrines.** A vertical hole of size 9 to 12 ins. is drilled with an earth auger up to a depth of about 8 to 20 ft. or more, without reaching ground water level. Where an earth auger is not available, a pit can be made of size 2 to 3 ft. 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 latrine seat is at least 9 ins. 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 7 ins., for use. Size of the squatting plate should be 3'-3"  $\times$  3'-6" min. overall size and should normally extend to the superstructure walls. A cement concrete pre-cast slab 2ins. thick may be made which can be used for an unsupported span up to 2½ ft. Foot-rests should form an integral part of the squatting slab where possible. Cover for the hole is desirable. A temporary superstructure can be provided over the seat to afford privacy. When the hole gets filled up to about 1½ to 2 ft. 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 25 ft. 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 50 ft. should be taken.

**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 2'-6" diameter extending to about 3 ft. below ground water level, up to a depth of about 10 to 12 ft. Half of the well is covered up by the squatting plate and the other half



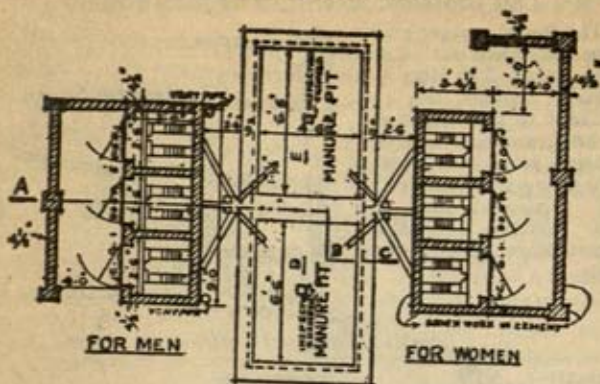
with a concrete or stone slab, which can be removed when it is desired to remove the sludge (after about 4 to 6 years), which can be used as manure.

**Service Latrines.** Special types of closets are now manufactured by some firms which are fixed in small compartments in place of the usual w.c. 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 6-in. 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 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 G.I. 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 6'-0"  $\times$  2'-6" and 5 to 6 ft. deep with 6 ins. to 9 ins. air space. The effluent is passed on to a soakpit or a filter chamber built adjoining the privy and then to the municipal drain.





*Manure Pit Public Latrines*

The set of six seats is meant for 100 daily users for six months. To start with, the excreta is allowed to flow into one pit and the inlet pipe of the second pit is closed. After six months the pit in use is closed and the flow of excreta turned to the second pit. The night soil of the first pit is allowed to cure into humus for about 5½ months after which it is cleared up and the pit exposed to sun for 15 days to dry up and then is put into use again and the second pit is closed down. The working is repeated.

## 5. PLUMBING &amp; INTERNAL FIXTURES

*Joints*

Joints between pipes of -	How made
Lead to lead	Wiped soldered joint. (Plumber's joint.)
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 fittings by socket and lock-nut.
Iron or copper and stoneware	Cement mortar joint as for stoneware pipes.

Joints between pipes of	How made
Lead and stoneware (Connection between earthenware W.C. trap and lead soil pipe)	A brass thimble (having socket at one end and spigot at the other) necessary to receive the trap outlet which have cement caulked joint. Wiped solder joint between brass spigot and lead pipe.
Cast iron and stoneware (Connection between earthenware W.C. 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.
Cast iron water pipes	Red lead or cement
Hot water cast iron pipes	Cement or red lead, or "rust cement" maybe 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 it not corrode. Brass and earthen ware take cement but lead will not.



Min. length of wiped soldered joint for pipes of different diameters :—

Pipe dia. ins.	$\frac{1}{2}$	$\frac{3}{4}$	1	$1\frac{1}{2}$	2	3	4	5	6
Min. length of joint, ins.	$2\frac{1}{2}$	$2\frac{1}{2}$	$2\frac{1}{2}$	$2\frac{1}{2}$	3	3	$3\frac{1}{2}$	$3\frac{1}{2}$	$3\frac{1}{2}$

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 Work.** 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 part	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.

Sanitary appliances are usually made of the following materials :—

Baths	Porcelain enamelled cast iron and sheet steel.
Lavatory basins	Earthenware, fireclay, vitreous china, porcelain enamelled sheet iron, stainless steel, porcelain enamelled cast iron.
Sinks	Fireclay, porcelain enamelled cast iron and pressed steel, stainless steel.
Draining boards	Asbestos cement, cast iron porcelain enamelled, fireclay, pressed steel sheet porcelain enamelled, stainless steel, wood.
W.C.flushing cisterns	Cast iron, pressed steel, lead-lined and copper-lined wood.
Hot water cylinders	Galvanized mild steel, copper.
Traps	Lead, other non-ferrous metals, cast-iron malleable cast iron.
Pipes for water, waste, soil and rainwater above ground and below ground	Lead, lead alloy, copper, steel, cast-iron, salt-glazed ware, aluminium, asbestos-cement.

## 6. STANDARDS FOR PUBLIC SANITARY CONVENIENCES

"Sanitary Conveniences" means closets (or latrines) urinals and wash basins, etc. The following scales may be adopted in the absence of any by-laws or prescribed standards :—

	Males		Females
	W.Cs.	Urinals	W. Cs.
Public Latrines	2%	0.2%	2½%
Cinemas, Theatres, etc.	1% up to 400 ½% above 400	5%	2% up to 200 1% above 200
Schools, Offices, Factories	4% up to 100 2% above 100	4% up to 100 3% above 100	6½% up to 100 3% above 100
Boarding Houses	10%	4%	12½%
Hospitals			
Indoor (beds)	12½%	2%	12½%
Outdoor	1%	1%	2%

A minimum of 1 w.c. seat should be provided in all buildings. It is good practice to provide 1 seat extra to the prescribed scale. No urinal need be provided in residences and in factories, offices, etc., where there are less than six users. In addition to the above, wash basins should be provided at about the same scale as closets except for cinemas where these may be 1 for every 200 persons or part thereof. Washing taps are required in factories. Each 50 sq. ft. of floor area in each room of offices is deemed to be occupied by one person.

## 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 R.C.C., 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. The human faeces consist of about 65 per cent of mineral matter and about 35 per cent of organic matter. Only about 20 to 40 per cent of the organic matter (solids) are liquefied or gasified, the mineral matter does not undergo any chemical change in a septic tank.

During the course of action in a septic tank the lighter matter rise to the surface and form a thick 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 6" 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 5 ft. 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 1'-6" from the floor, and from 9" to 1'-6" 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 few inches below the water level in the tank and at a distance of about 2 ft. from the outlet end. Instead of the second baffle wall 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 slab of slate, flagstone, R.C., or even wood, often suspended from the top into grooves in the sides of the tank to submerge about 9 ins. or 1 ft. into the liquid. The outlet pipe may be straight, instead of bent down, where a scum board has been provided.

**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. Additional tank volume should be provided for sludge storage on the basis of 1 c. ft. per capita per year. Dilution of the sewage between certain minimum and maximum limits is also essential for a complete septic action to take place and provided it lies between those limits, it does not affect the size of the septic tank.

It is considered that as long as the dilution per head is more than 5 gallons and less than 40 gallons, a septic tank having a content of about 2.5 c.ft. per user will do the work required. It has, however, been reported that septic tanks with as low a dilution as 2 gallons 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 5 gallons per user, there is not enough water for the necessary reactions to take place in the tank, while if it is weaker than 40 gallons per user it goes through the tank too fast and remains under-treated. He limits the capacity of the tank between 1.7 c.ft. to 2.4 c.ft. per user. Effluents from both the over and under-treated tanks clog the filters. In America as the sewage is much more dilute, higher capacity tanks are provided (av. 6.5 c.ft. per user) to ensure the organic matter remaining in the tank for the time necessary to bring about the essential changes. However, it is the general opinion that, for the proper operation of a house drainage system, a minimum water supply of 20 gallons per capita per day should be available.

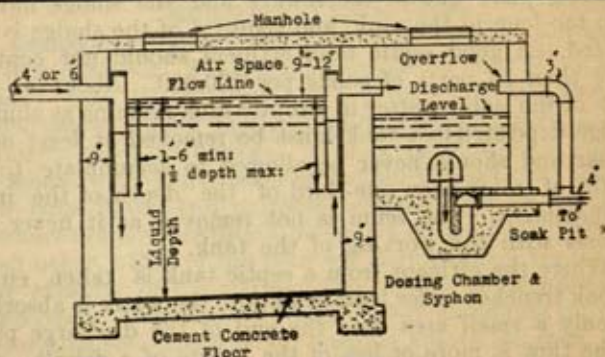
As it is not possible to estimate the exact number of users or the amount of dilution for any proposed installation, it will be safe to provide a capacity of about 3 to 4 c.ft. per user for small tanks and about 2.5 c.ft. for large tanks for Indian conditions where the bath-rooms are also connected with the system. Where only flush water flows into the septic tanks, a space of 1.5 c. ft. per user may only be provided for domestic tanks. For the above capacity tanks the number of users can be increased occasionally to  $1\frac{1}{2}$  times without over-loading. The minimum capacity of a septic tank should be not less than 250 gallons which is sufficient for 10 to 15 users; width should not be less than 1'-9" (prefer 2 ft. min.) for a man to enter in 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 9 ft. for any single unit as great width introduces the danger of uneven flow and local stagnation. Liquid depth should not be more than 7 ft., prefer 6 ft. max. at the inlet end which is considered as ideal depth by Col. F.C. Temple. Liquid depth should not, however, be less than 4 ft. 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 tank is considered to be  $72' \times 9' \times 7'$  which should be sufficient for about 2500 users.

The following table gives inside dimensions of domestic septic tanks where bath rooms are connected :—

No. of users	Length	Breadth	Liquid depth	The number of users given in the table if increased by 50 per cent will not over-load the tanks.
10	5'	2'	4'	There should be an air space of 6" to 12" above the liquid depth for the accumulation of gases.
15	5½'	2'	4½'	These sizes include for one year sludge storage. (Frequency of sludge removal is optional and may be removed at intervals of one to four years.)
20	6'	2'	5'	
30	7'	2½'	5½'	
40	8'	2½'	6'	
50	9'	2½'	6½'	
100	11'	3½'	7'	



The dosing chamber shown in the illustration need not be provided in tanks of less than 700 galls. capacity or where it is not considered necessary. The function of a dosing chamber has been explained later. An airtight manhole cover is provided of cast iron of size  $18'' \times 12''$ , or circular. A ventilating pipe is generally provided of 3 inches diameter, taken to about 12 to 15 feet height. Venting can also be done by house vents or sewers. (Some authorities are of the opinion that introduction of a vent pipe retards bacterial action). The floor of the tank should slope at 1 in 30 towards one side or the centre to facilitate cleansing of deposits, and the manhole should be above



this. The tank should be built as far away from the house as possible on the leeward side.

**Working and Care of a Spetic Tank.** A septic tank should be initially filled with water. Disinfectants 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. A good septic tank effluent should not contain more than 15 parts of solids per 100,000. About 41 per cent of the solid matter in a septic tank remains as sludge. 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 the 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" built 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  $2\frac{1}{2}' \times 3'$  or of the same width as the tank according to the flow expected. Depth may be 2'-9" to 3'-6" (floor to ceiling).

For a single house of ten to twelve inhabitants built on gravel or chalk or other permeable strata, 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 30 ft. deep and 2 to 3 ft. 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 50 ft. away from the building, and should be covered.

Use cast iron pipes for sewage up to a length of 4 to 6 ft. from the house to the septic tank and beyond that glazed earthen pipes; diameter is about 4 ins. to 6 ins. Give a slope of 1 in 40 to the sewage pipe to enable the solids to flow down. Provide manholes for lengths over 300 ft.

#### **Effluent from Septic Tanks**

**Cesspool or Seepage Pit.** The effluent may be disposed of in a cesspool. The cesspool should be built 50 ft. away (min.) from the building. It is a small circular chamber built below ground level. Depth is 5 ft. minimum from effluent 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 4 to 5 ft. minimum for rapid absorption soils and 6 ft. for medium absorption soils. 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 15 sq. ft. for rapid absorption soils, 25 sq. ft. for medium and 40 sq. ft. 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, which may be made conical with a cover at the top. The pit may be filled with gravel or brick-bats for about 1 ft. 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 4" dia. to pour its contents in the centre of the pit.

The bottom of a cesspool should not be less than 2 ft. above the sub-soil water. A soak-pit should not be less than 100 ft. away from a well or any source of water-supply.

Relative absorption	Type of soil	Approx. range of loading rates in galls./sq. ft. day
Rapid absorption	Coarse sand, gravel	2.8
Medium absorption	Fine sand, sandy loam	2.8 to 1.4
Slow absorption	Clay with sand or loam	1.4 to 0.64
Semi-impervious	Dense clay	0.64 to 0.46
Impervious	Rock	Below 0.46

## 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 outskirts and the 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 throughout the 24 hours.

Due allowance must be made for future expansions and increase of population as explained earlier.



Some times 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 10 to 18 feet 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 town 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 may settle on the invert. The screening chamber is made 2 to  $2\frac{1}{2}$  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 6 ins. sewer on a gradient of 1 in 100, a chamber of size 10 to 12 ft. long, 4 to 6 ft. high and 1 to 2 ft. 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; spacings of  $1\frac{1}{2}$ " to 2" will serve for most of the works. Where finer screening is desired, two racks can be fixed, coarse and fine which may be with  $\frac{1}{2}$ " to  $\frac{3}{4}$ " clear spaces. The bars are rectangular in cross-section, usually about  $\frac{3}{8}$ "  $\times$   $1\frac{1}{2}$ ", 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 minimize clogging and facilitate cleansing. Curving the bars at the 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 screening is recommended for flows in excess of five million gallons per day.

The quantity of screenings removed varies with the size of openings, and may be about 1 c. ft. per million gallons with openings larger than 1", and about 8 to 10 c.ft. with  $\frac{1}{2}$ " to  $\frac{3}{4}$ " openings. 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. Detritus 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 settling tank). It should be small and narrow in proportion to length to check the velocity of flow, which should not be more than 1 ft. 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  $1/125$  to  $1/100$  of the daily average flow with detention period of  $\frac{1}{2}$  to  $\frac{3}{4}$  minute. The area of a tank is determined by surface loading and the depth by the detention period. The grit is considered to settle 1 ft. in about 16 seconds, and the chamber should be about 1 ft. long for each inch that particles must settle. A travel of 30 ft., i.e., a length of 30 ft., is considered sufficient for small installations. The depth should be about  $1/16$ th of the length; the usual depth of a grit chamber is 3 to 4 ft. The quantity of grit is considered to be about 3 to 5 c.ft. per million gallons. 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 3 ft. deep and have a detention



period of about 3 to 5 minutes. There 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 4 to 6 inches below the scum surface for the liquid to flow under them continuously.

### **Sedimentation Tanks or Settling Tanks or Clarifiers**

Grit chambers, skimming tanks and screens are provided ahead of the 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 capacities 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 6 to 12 ft., usually about 9 ft. Detention period is 2 to 3 hours with a velocity of about 1 to  $1\frac{1}{2}$  ft. per minute, and overflow rate of 600 to 800 gall. per sq. ft. per day to bring about sedimentation of the suspended matter. Shallower the tank, the shorter need 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 a foot. The sewage should enter the tank quietly and at a low velocity and any turbulence 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 1 ft. below and 9 inches 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 a few feet 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 2 galls. per head of population. The usual method is to use the tanks continuously by drawing 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. By-pass channels and valves should be provided so that any one tank can be isolated and emptied 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 one gallon a yard super 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, 10 to 20



grains per gallon of sewage is the dose. If alum is used, 6 to 9 grains per gallon 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  $\frac{1}{4}$ " to 3". 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 6 to 10 ft. depth is generally considered suitable for sizes varying from  $1\frac{1}{2}$ " to 3", and of 3 to 5 ft. depth for sizes  $\frac{1}{2}$ " to 1". 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 larger material, and the secondary filter may be composed of much finer material, which gives the 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 4 to 6 ft. 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. The rate of treatment is about 40 to 60 galls./c. yd./day for average sewage, and 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 in 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  $\frac{1}{2}$  gall. per head of population per day. The sludge is discharged on to the beds to a depth of about 9 ins., 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 super yard of bed is required for every five to seven head 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 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 4 to 10 ft. and the size usually recommended is 70 galls./c. yd./day, with average sewage (and 35 to 40 galls. with very strong sewage) and 10 ft. depth, using 2" to 4" size of stones or bats; or an average of 150 galls./sq. yd./day, with 6 ft. depth and 2" 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 an 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 6 ins. 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, 3" to 4" size and about  $1\frac{1}{2}$  to 3 ft. apart, laid radially from the centre with suitable falls (say 1 in 100), led to an external peripheral channel, for collecting and disposing of the effluent. Unglazed earthen lay tiles are also commonly used.

**Simple Filters for Small Installations.** Make a circular platform of concrete, cement rendered, laid 3" higher at the centre than at the circumference, surrounded by a drain to carry off the filtered effluent. A 5 ft. high pillar is built in the centre of the platform the top of which is corbelled out to form a basin 2 ft. 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 6-ins. two layers of dry bricks or stones may be laid with  $\frac{1}{4}$ " spaces between them. Quantity of filtering material required is about 3 c.ft. per user 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 5 ft. 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 18 to 30 ins. Loading rate is 2 to 5 galls./sq. ft./day for preliminary treated sewage, 1gall./sq. ft./day for raw sewage.

and 10 galls./sq. ft./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 1" to 2", followed by smaller size up to about  $\frac{1}{4}$ " in 3-in. 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 Settling 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) *An erobic 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 a 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  $1\frac{1}{2}$  to 2 times the depth which is about 10 ft. in small plants and 15 ft. 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 necessity 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 3-in. 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 40 ft. 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 15 ft. 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 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 4 ins. and the velocity of flow not less than  $2\frac{1}{2}$  ft. and not more than 6 ft. 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 200 to 1000 gallons per minute, with lifts of about 20 ft. "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 5 to 6 ft. and the bed of sump sloped towards the suction side.

**Valves on Pumping Mains** should be located as follows:—

(a) *Reflux valves.* Immediately above 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 main where required to isolate sections.

(c) *Air Release 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 10 feet 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 under-drains 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 loam overlying a dry gravel or other porous sub-soil, about 5 to 6 ft. 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 gallons per acre per day generally varies from 4,000 to 30,000 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 acre of land per 100 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.

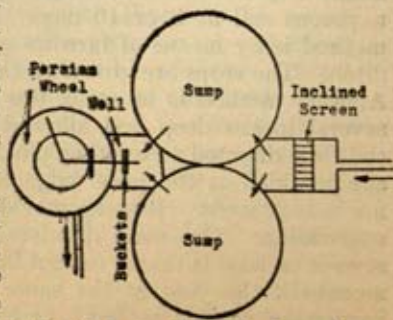
Where sewage is passed on to a land for irrigation, about 2 to 4 inches of sewage can be applied at a time on a porous soil at 8 or 10 days intervals. The common method is by means of furrows opening out from a header ditch. The crops are grown on the ridges between furrows. Another method is to apply the sewage on a plot of land several inches 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 4-ins. 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  $\frac{1}{2}$ " to  $2\frac{1}{2}$ ", which should surround the pipes with a minimum of 4" to 6" at the bottom and 2" at the top. The pipes may be laid to a gap of  $\frac{1}{8}$ " to  $\frac{1}{4}$ " 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 18" to 36" width at the bottom and 18" to 48" depth. The trenches should be laid at a distance apart equivalent to three times the width of a trench with 6 ft. 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 50 to 100 ft. apart and *kachha* side drains (or open trenches) made at right angles to the main drains, about 6 to 10 ft. 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 6 ft. min. to 10 to 15 ft. 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 50 to 100 c.ft. per million gallons, 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.")

When the subsoil water is within 10 ft. 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 2 inches. 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 2 ft. to 4 ft. is generally suitable for clay soils and 4 ft. for sandy soils. Distance apart in clay soils varies from 20 ft. to 25 ft. or even 30 ft., in strong loams 30 ft. apart and in light soils 40 ft. In sands, a spacing of 200 ft. may be used. Clay is a soil which is very retentive of moisture and one c.ft. of moist dry clay will absorb about one gallon 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 freer 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 200 ft. to 1 inch 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 330 ft. to 1 inch. 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 1/192 (16 ft. to 1 in.) 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.** For detailed levelling, reading on the staff



should be taken to hundredth of a foot. Levels should be taken at the following intervals :—

(a) 200 ft. along the centres of all roads and at summits and lowest points. (b) 100 ft. on the beds of all pucca drains and 200 ft. for kacha drains, or of the proposed drainage lines. (c) At every 400 ft. (should the ground allow of it) readings should be taken at 200 ft. and 400 ft. 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 9" 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 which should be not less than  $\frac{1}{4}$ " nor more than 3" apart, on a 330 feet to the inch.

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 100 ft. to 1 inch), 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  $\frac{1}{4}$  in. to 1 ft. 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

(Also see Section 20 "Estimating")



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 refuse of the existing arrangements.

## 11. 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 pipes.

*Benching* : The sloped floor of a manhole on both sides of and above a channel, on which a man can stand for cleaning the sewers.

*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.

*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.

*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.



*Drop Manhole* : A manhole incorporating a vertical shaft or pipe in which sewage falls from a sewer at a higher level to a sewer at a lower level.

*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.

The term is also used for the erosion formed on road shoulders by rain water.

*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 sub-soil 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.

*Interceptor Manhole or Interceptor Chamber* : A manhole incorporating an intercepting trap, and providing means of an access thereto.

*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.

*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.

*Lateral Sewer* : Is a street sewer into which sewage from house connection pours.

*Liquefaction* : The results of the action of bacteria for the decomposition and purification of sewage or waste matter.

*Night-soil* : A mixture of faeces, 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** : A mass of sewage solids, both mineral and organic, 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. (Also see "Sullage").

**Sewers** : Are underground pipes or conduits which carry the sewage 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 organic solid matter deposited at the bottom of a tank during treatment of the sewage, mixed with more or less water to form a semi-liquid mass.

**Sludge Digestion** : The biochemical process by which organic matter in sludge is converted into more stable organic matter.

**Soakaway** : A pit, dug in permeable ground to which the surface water is led, and from which it may soak away through the ground.

**Soffit or Crown** : The internal surface of a pipe at the upper end of a vertical diameter.

**Soil Sewage** : The sewage from water-closets, slop sinks and urinals, and a mixture of this sewage with any other drainage or waste water.

*Sullage* : Waives water from bath-rooms, lavatory basins, kitchens, snks, street and roof washings, etc.

*Sub-soil Water* : Water occurring 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.

*Water Carriage System* : Removal of sewage by a network of underground pipe line 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 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* : Same as Sullage.



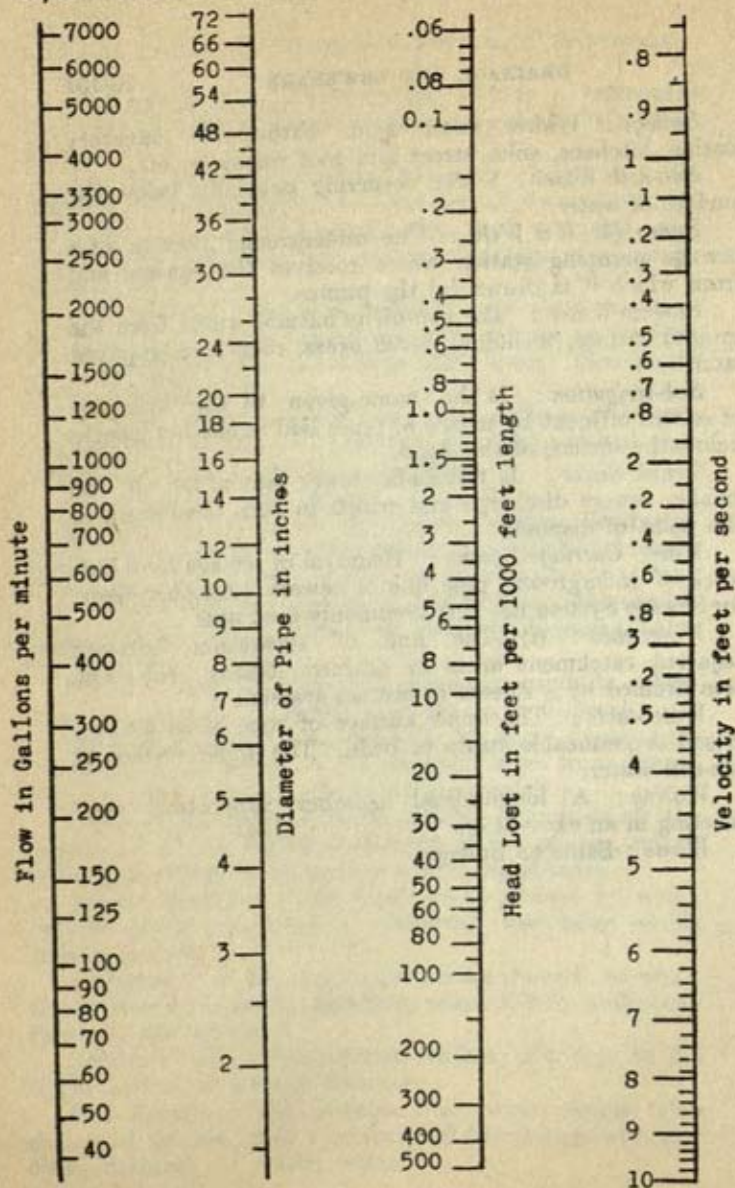


CHART FOR FLOW IN WATER PIPES  
(Based on Hazen and Williams Formula with  $C = 100$ )

## SECTION 17

## IRRIGATION

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## 1. GLOSSARY OF IRRIGATION TERMS

(Based generally on the Central Board of Irrigation & Power, India.)

*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. The capacity of canals in India varies from the 10,000 to 15,000-cusec main canals down to 100-cusec branches and laterals.

*Branch Canal or a Branch*—Taking off from a main or another branch and having a head capacity of not less than 300 cusecs. There is no direct irrigation through it.

*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. Under 300 and above 25 cusecs discharge.

(All channels with discharges less than 350 cusecs have been classified as distributaries in the Bhakra canal system.)

*Minor Distributary*—A channel taking its supply from a major distributary for supplying water to outlets. 25 cusecs and under discharge.

*A Lateral*—Name for a distributary in U.S.A.

*A Minor*—Is a small channel taken from a distributary where the water courses carrying water to the fields have to run longer than two miles.

*Water-Courses or Irrigation Channels and Field Channels*—Water-courses do not carry water at the head more than 3 or 4 cusecs and field channels generally carry less than one cusec. They receive water through outlets fixed in the banks of distributaries or minors for supplying to the fields. Water courses are owned and maintained by the cultivators.

*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).

*Perennial Canal*—A channel which is designed to irrigate 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.

*Non-perennial Canal*—A channel which is designed to irrigate during only part of the year, usually in the summer season and at the beginning and end of the winter season.

*Ghat-fed Canal*—Is a canal from a storage which derives its supply from monsoon rains in the "ghats".

*Tank-fed Canal*—Which derives (unreliable) supply from a storage fed from "non-ghat" catchments.

*Inundation Canals*—Channels excavated (during old times) directly from rivers, with or without some form of head regulator, not essentially 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.

*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 30 ft. for sub-branch and 50 ft. for main branch should be left between the two channels. These widths may be varied by  $\pm 10$  ft. to allow min. shift in the alignment of the ditch channel.

*Balance or Balancing Tank*—Is a subsidiary reservoir for storing excess water which is utilized during periods of short supply.

*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.

*Distribution System or Distribution Works*—Water from a river or reservoir carried through canals, distributaries or minors, etc. on to the fields for irrigation.

*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.

*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.

*Broad Irrigation*—Irrigation with sewage from a town (instead of with natural water), in which the disposal of the sewage is the primary object.

*Irrigating Head*—The flow used for irrigation of a particular tract of land.

*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 sometimes 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-grade*—Average diameter of the silt particles.

*Load*—It is the weight of silt in movement in a channel and is usually expressed as lbs. per second.

*Silt-charge*—Proportion of silt per unit volume by weight in water.

*Steady Flow*—Is that state of flow in a stream where the discharge remains constant across any defined section, at all times.

*Regime or Regimen Flow*—Is that state of stream flowing is self borne alluvium when its slope and shape have reached a stable form as the result of its flow characteristics and there is neither silt nor scour.

*Critical Flow*—The flow is said to be critical when for a certain discharge the total energy of flow i.e., the sum of the potential, kinetic, and pressure energies is a minimum for the discharge passing. It varies with the discharge passing.

*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*.

*Uniform Flow*—Is steady flow in a stream when the depth does not vary with constant discharge.



**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.

**Critical Velocity**—Critical velocity for a channel is that mean velocity which for a channel of a given depth will just keep the channel all the year round 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.

**Hyper-critical Velocity**—A velocity in excess of the critical velocity.

**Sub-critical Velocity**—A velocity less than the critical velocity.

**Sounding**—Measuring water depth of a river or a channel (with a rod or a string and weight).

**Critical Depth**—The depth of water in a channel corresponding to (one of the recognized) critical velocities.

**Drift**—The distance in feet a boat (used for measuring discharge) travels downstream with the current (whether anchored or not) during the time taken to make a velocity observation.

**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.

**Reach**—A comparatively short length of a stream or channel.

**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.

**Area Assessed**—The area irrigated on which water rates are levied.

**Gross Command**—Is the total area included within the irrigation boundary 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 or the village itself.

**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 *chak outlet*.

*Actual Command*—Is the area on which water will flow from a complete canal system as constructed or likely to be constructed.

*Culturable or Cultivable Commanded Area*—Is the portion of 'gross commanded area' which is cultivated, or culturable. (Gross commanded area minus the area of uncultivable land such as, roads and paths of a village.)

*Ayacut* is a Tamil name for culturable area.

*Culturable Irrigable Area*—The gross irrigable area less the area not available for cultivation.

*Irrigable Area*—Area within the command which can be irrigated (both flow and by lift).

*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.)

*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.

*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.

*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.

*Dry Crop*—A crop which is raised entirely with the help of rain-fall.

*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*".

*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.

*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.

*Base*—Is the period (in days) for which water is supplied to the crops, and on which duty is calculated.

*Duty*—Is the relation between the area irrigated and the quantity of water used

(These terms have been explained in detail later)

*Capacity-Factor*—Is the ratio of mean supply of the canal to its authorized full supply or capacity.

*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.

*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".

*Cusec Day or Day Cusec*—Is one cusec flowing for a day of 24 hours—the quantity of water equal to 86,400 c. ft. A cusec-day on one acre is roughly 2 ft. (1.98 exact) depth of water for the day.

*Open Discharge*—Is the total of the daily discharges in cusecs divided by the number of days the canal is allowed water for irrigation.

*Time-Factor*—Of a channel is the ratio of the number of days the channel is in flow to the days of crop period, or irrigation period. The value of the time factor is generally from 0.5 to 0.7.

*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 ft. It is 43,560 c.ft.



*Acre-inch*— $1/12$ th of an acre foot, defined above (and is almost equal to one cusec hour).

*Capacity Co-efficient*—Is the number of acres irrigated in one day per cusec of supply.

*Capacity*—The authorized or designed full supply discharge of a channel.

One cusec running continuously for 24 hours will give a volume of 86,400 c. ft. of water, or 540,000 gallons.

Or, a cusec a day = 1.983 acre-feet.

A million c.ft. = 22.96 acre ft. or 23 acre ft. (approx).

11.574 cusec-days = 1 million c.ft.

One inch of run-off per hour = 1 cusec per acre  
= 3630 c.ft. = 645 cusecs per sq. mile.

1 cusec per sq. mile = 13.56 inches yearly run-off.

In America, one c.ft. per second is sometimes called "second foot."

*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.

*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.

*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.

*Outlet Discharge Factor*—The 'Duty of Water' with reference to a suitable 'base' and the 'place' of its measurement as decided upon.

*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.

*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.

*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.

*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.

*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.

*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.

*Superpassage*—A work which carries one channel over another without lowering the bed level of the lower channel.

*Siphon or Syphon*—A closed conduit (pipe or tube)



for conveying water over an obstacle shaped like an inverted V by raising it above its 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).

*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.

*A 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 downstream 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.

*Chute*—(i) A high velocity conduit for conveying water to a lower level; (ii) An inclined drop or fall.

*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.

*Head Regulator or Head*—This term is usually applied to the control works constructed 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.

*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.

*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 a 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.

*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.

*Notch-Fall*—A fall the crest of which is usually at or near the bed level, generally without a glacis.

*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.

*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 hard 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.

*Submerged Weir*—A weir which in use has the tail-water level higher than the weir crest, by which the discharge is affected.

*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.

*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 reservoir or a dam. A pick-up weir serves the same purpose as an intake weir but is constructed as an adjunct to a reservoir.

*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*.

*Escapes*—Weirs (with or without 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 drainage 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.

*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. (See also Barrage).

*Contracted Weir*—A measuring notch with sides designed to produce a contraction in the area of the overflowing water.

*Parabolic Weir*—A measuring weir whose notch is



bounded on the sides by parabolas such that the flow is proportional to the head.

**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 overflowing water. A weir may be suppressed on one end or both ends.

**Waste Weir (or Spillway)**—Is an escape provided for the passage of surplus water from a tank or a reservoir.

**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.

**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.

**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.

**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.

**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 a barrage



is much more than that of a weir.

*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 waterway, etc., to prevent scour.

*Spillway*—A passage for the flow of surplus or waste water in a weir or conduit.

*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 ordinary 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.

*Head-wall*—A wall built across a small channel and provided with a regulating arrangement to head up water on the upstream side.

*Flank Wall*—The retaining wall in continuation of abutments both upstream and downstream.

*Breast Wall or Face Wall*—When applied to irrigation practice, a wall immediately above the face of a submerged orifice

*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.

*Toe Walls*—Longitudinal shallow retaining walls built near ground level (at the foot of the slope) for supporting the pitching on the face of earthen embankments or flared walls.

*Talus*—A protection at the downstream end of a weir or fall, consisting of blocks of concrete or masonry.

*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.

*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  $\frac{1}{2}$  : 1 as required. The flared walls may be straight or curved.

*Baffles*—A cross-wall or a set of vanes or some other device placed in a channel downstream of a hydraulic structure to effect a uniform distribution of velocities across the section, and to dissipate energy.

*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.

*Free-board*—The margin between a canal bank or the crest of a dam and the full supply level. (Also see under "Bridges").

*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".



**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 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 15 to 20 ft. per second is required to dislodge silt.

**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.

**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".

**Tail-Race**—(i) The channel between the silt extractor and the river through which the escape 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.

**Head-Race**—A channel leading water to a water-wheel.

**Fore-bay**—A reservoir or pond at the head of a pen-stock or pipe line.

**After-bay**—The tail race of a water-power plant; a pond or reservoir at the outlet of the turbines.

**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 Water**—The water just downstream of a structure.

**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.

**Bifurcation Gate**—A structure that divides the flow between two conduits or channels.

**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.

*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.

*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.

*Fender Piles*—Wooden or R.C.C. piles fixed 8 to 10 ft. centre to centre, at surplus sluices of off-takes for the canal to facilitate the movement of boats.

*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.

*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.

*Jetty*—A dike of piles, rock, or other material, extending into a stream or sea to induce scouring or bank building or for protection.

*Hydraulic-Fill Dam*—A dam composed of earth, sand, gravel, etc. Generally the fine materials are placed towards the centre for greater imperviousness.

**Gravity Dam**—A dam depending solely on its weight to resist the water pressure. It may be straight or slightly curved in plan.

**Crib Dam**—A barrier made of timber, built in compartments or bays which are filled with stone or other suitable material.

**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.

**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.

**Core Wall or Diaphragm Wall**—A wall of masonry, sheet piling or puddled clay built inside a dam or embankment to reduce percolation.

**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.

**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.

**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.

**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.

**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.

*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.

*Spur or Groyne*—Spurs are made to train the flow and reduce the velocity in a channel and cause silting. They are generally constructed of brushwood or wooden piles driven into the bed or berms.

*Bandalling*—A temporary spur composed of stakes (bamboos) driven into the bed of a river about 3 ft. 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 6 ins. 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.

*Dyke*—Is an earthen embankment built on each side of a river some distance away from its banks, to control floods. They are more or less like earthen embankment, kept about 4 to 6 ft. above the highest flood level. As far as possible, curves should be avoided in earthen 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.

*Arid*—A term applied to lands or climates that lack sufficient water for agriculture, without artificial irrigation.

*Semi-Arid*—A term applied to lands or climate, neither entirely arid nor strictly humid, in which inferior crops can be grown without irrigation.

*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.

*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.



*Capillary-Water*—Water held in the pores of the soil above the water-table by capillary force, and which is not drained by gravity.

*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.

*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.

*Capillary Fringe*—The water held by the forces of capillarity above water-table by the interstices of soil.

*Creep*—The movement of the water under or around a structure built on permeable foundations.

*Caving*—The erosion of a river or canal bank by the undermining action of water.

*Seepage*—The water which by the action of capillary attraction passes underground from channels or tanks through close soils and does not appear visibly in the vicinity but the area becomes "water logged". The percolation of water into or from the soil; infiltration.

*Percolation*—Flow of water through the particles of soil (porous substance) due to the force of gravity or pressure of head.

*Hygroscopicity*—Is the ability to absorb and retain moisture without necessarily becoming liquid.

*Hygroscopic-Water*—Water found on the surface of the soil, and which is not capable of movement either by gravity or capillarity and can only be driven off by heat.

*Hygroscopic Co-efficient*—It is the moisture, in percentage 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.

*Wilting Co-efficient*—Is the moisture content in soil above which water is available for plant growth. It is the ratio of the weight of water in the soil at the moment when (with gradual reduction in the supply of soil water)

the leaves of the plants growing in the soil first undergo a permanent reduction in their water content as the result of a deficiency in the supply of soil water, to the weight of soil when dry.

*Free-Water*—Water in soil which is in excess of the hygroscopic 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*.

*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.

*Deliquescence*—Is the ability of a material to absorb moisture from the air and thus to dissolve and become liquid.

*Conservation Works*—Works like dams and reservoirs built for storing water during the days of plenty of supply for use in adverse times.

(More terms are given under respective headings in the following pages, and Sections on "Hydraulics, Water Supply, Drainage & Sewerage, and Bridges.")

## 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 smaller 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 the 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 erode 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 *fertilizing 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 (C.V.R.)— $V/V_o$

It is the ratio between the actual mean velocity ( $V$ ) in a channel to the critical velocity ( $V_o$ ) calculated from any of the standard formulae. For a non-silting channel the C.V.R. 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 80 to 85 lbs. per c. ft. 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 50 ft. 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 2 to 3 ft. per second, with  $1\frac{1}{2}$  ft. 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	..	..	..	$3\frac{1}{2}$ ft./sec.
In branch canals	..	..	..	3 "
In large distributaries	..	..	..	$2\frac{1}{2}$ "
In small distributaries	..	..	..	2 "
In very small distributaries	..	..	..	$1\frac{1}{2}$ "
In water-courses and field channels	..	..	..	1 to $1\frac{1}{2}$ "

Since a canal in ordinary soil cannot be made with a greater velocity than about  $3\frac{1}{2}$  feet per second and the critical velocity is greater than that in canals which have depth of over 9 feet, it is not desirable to design canals in ordinary soil, with a greater depth than about 9 feet, unless the water is 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 5 ft. 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:

	<i>Ft. per sec.</i>
Soft earth or very fine clay . . .	0.25 to 0.30
Soft clay or fine clay . . .	0.50 to 0.75
Very fine or very light pure sand . .	0.75 to 1.00
Very light loose sand or silt . .	1.00 to 1.50
Coarse sand or light sandy soil . .	1.50 to 2.00
Average sandy soil and good-loam . .	2.00 to 2.50
Sandy loam . . .	2.50 to 2.75
Light ordinary earth or sandy bed . .	2.50 to 3.00
Average loam or alluvial soil . .	2.75 to 3.00
Firm loam, clay loam . . .	3.00 to 3.75
Firm gravel or clay . . .	3.50
Stiff clay soil ; ordinary gravel soil, or clay and gravel . . .	4.00 to 5.00
Broken stone and clay . . .	5.00
Grass . . .	3.00 to 5.00
Coarse gravel, cobbles, shingles, shale . .	5.00 to 6.00
Conglomerates, cemented gravel, soft slate, tough hardpan, soft sedimentary rock . .	6.00 to 8.00
Soft rock . . .	4.50 to 8.00
Hard rock . . .	10.0 to 15.0
Very hard rock or cement concrete . .	15.0 to 25.0

Bottom velocity may be about  $\frac{1}{3}$ th to  $\frac{2}{3}$ th 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 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—
ft.	ft. per sec.
1.0	0.4
2.5	0.7
5.0	0.9
10.0	1.5
1.0	0.9
2.5	1.5
5.0	1.75
10.0	2.25
1.0	1.75
2.5	2.25
5.0	3.0
10.0	3.5
1.0	2.25
2.25	3.0
5.0	3.5
10.0	4.5
1.0	5.0
2.5	6.0
5.0	7.0
10.0	9.0
1.0	15.0
10.0	23.0

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 1.30 ft. will carry silt at a depth of 2 ft. but it will fail to carry silt at a depth of 5 ft. for which a velocity of over 2.35 ft. will be necessary (see above table).

The following bottom velocities in a channel will just produce motion in the substances mentioned :—

Ft. per second	Material
0.25	Soft earth, fine clay, river mud or silt
0.50	Common clay
0.70	Fine sand
0.80	Coarser sand
1.00	Fine gravel and coarse sand
2.00	Pebbles 1 in. diameter
3.00 } to 3.33 }	Pebbles, egg size
5.00	Stones 3 in. diameter
6.60	Boulders 6 to 8 in. diameter
10.0	Boulders 12 to 18 in. diameter.

**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 ft./sec.	
	Shallow ditch	Deep canal
Fine sand or silt .. ..	0.50—1.50	1.50—2.50
Coarse sand or sandy loam	1.00—1.50	1.75—2.50
Silty or sand loam ..	1.00—1.75	2.00—3.00
Clayey loam or sand clay	1.50—2.00	2.25—3.50
Fine gravel .. ..	2.00—2.50	2.50—5.00
Well graded gravel ..	2.25—3.50	4.00—6.00
Pebbles, broken stone ..	2.50—4.00	5.00—6.50
Stone masonry ..	7.60—15.00	..
Solid rock or concrete ..	15.00—25.00	..

**Velocities to Move Stones :—**Chailly's formula :

$$V = 5.67 \sqrt{G \cdot d} \quad \text{or} \quad d = \frac{V^2}{85} \quad \left| \begin{array}{l} d = \text{dia. of stone in ft.,} \\ G = \text{specific gravity of stone} \end{array} \right.$$

The following velocities (V) of water, in feet per sec., in a river will move stones of diameter (d) :—

V	$\frac{1}{8}$	1	2	3	4	5	7	10	15	20	25	30	35	40
d	$\frac{1}{8}$ "	$\frac{1}{4}$ "	$\frac{1}{2}$ "	$1\frac{1}{4}$ "	$2\frac{1}{4}$ "	$3\frac{1}{4}$ "	7"	1.2'	2.7'	4.7'	7.4'	10.6'	14.4'	18.6'

**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 where 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 them 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 based his theory on the charge and grade of silt present in the Bari-Doab canal (Punjab). (Thus the formula for  $V_0$  given below depends upon the nature and quantity of silt present in the parent channel from which the canal takes off.) If the channels have different size and grade, they would run non-silting with a velocity different from  $V_0$ .

Kennedy's formula (when put in the general form) is :

$$V_0 = K d^m$$

$V_0$ —is the critical velocity in ft. per sec.,

$K$ —is a constant depending upon the nature of the silt, which varies with the reach along the river or the channel, being higher for reaches near the heads of canals or the source of river reducing towards the tails of channels (as is evident from the values given below). This was fixed as 0.84 for the Bari-Doab canal.

$d$ —is the depth of water in ft. over the bed portion of the channel (full supply depth.),

$m$ —the value depends upon the type of silt carried by the water. It is different for different rivers.

The original equation was :

$$V_0 = K d^{0.64} \text{ or } = 0.84 d^{0.64} \quad (\text{for Punjab canals}) :$$

The value of  $K$  for various grades of material may be taken as :—

0.63 for very fine silt (as in Sind canals),

0.82 for light sandy silt,

0.84 for fine sand-silt (as in Punjab canals),

0.90 for coarser light sandy silt,

0.91 for fine sand silt (as in Burma rivers),

- 1.00 for sandy loam (as in Madras rivers),  
 1.07 for coarse silt and coarse sand,  
 1.2 to 1.5 for sand and small bajri,  
 2.5 to 3.0 for bajri and gravel,  
 3.0 to 3.5 for gravel and boulders.

$V_0$  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 following table gives Kennedy's critical velocities ( $V_0$ ) for fine sand-silt (Punjab canals) for the values of  $K=0.84$ , for various depths, Side slopes  $\frac{1}{2}$  to 1.

d	$V_0$	d	$V_0$	d	$V_0$	d	$V_0$
0.5	0.54	3.7	1.94	6.7	2.84	9.7	3.60
1.0	0.84	4.0	2.04	7.0	2.92	10.0	3.67
1.3	0.99	4.3	2.14	7.3	3.00	10.3	3.74
1.5	1.09	4.5	2.20	7.5	3.05	10.5	3.79
1.7	1.18	4.7	2.30	7.7	3.10	10.7	3.83
2.0	1.37	5.0	2.35	8.0	3.18	11.0	3.90
2.3	1.43	5.3	2.44	8.3	3.25	11.3	3.97
2.5	1.51	5.5	2.50	8.5	3.31	11.5	4.01
2.7	1.59	5.7	2.56	8.7	3.36	11.7	4.06
3.0	1.70	6.0	2.64	9.0	3.43	12.0	4.12
3.3	1.80	6.3	2.73	9.3	3.50	15.0	4.75
3.5	1.88	6.5	2.78	9.5	3.55	20.0	5.71

For Sind canals multiply  $V_0$  by  $\frac{1}{2}$

In alluvial soils, depth does not affect the value of "n" (in Kutter's formula) to any appreciable extent, but in the case of boulders and gravel, the roughness co-efficient varies greatly with depth.

Max. ratio of $\frac{\text{bed-width}}{\text{water-depth}}$	3.5	4	4.5	5	6	9
Discharge—cusecs ..	10	25	100	200	500	1000

The following equations have been evolved (based on the above Kennedy's theory) for the channels in Godavary and the Krishna delta systems :

**Normal Data of Design for "Kennedy" Channels  
with Kutter's " $n$ " = 0.0225.**

Discharge cusecs	Bed Width	Depth	Gradient 1 in—	Critical Velocity Ratio	Mean Velocity
2	2.00	1.0	2500	1.00	0.80
4	2.70	1.2	2500	1.03	1.00
6	3.50	1.4	2857	1.00	1.02
8	4.00	1.5	2857	1.03	1.07
10	4.75	1.6	3333	0.99	1.13
12	5.25	1.75	3333	1.00	1.18
14	5.50	1.80	3333	1.01	1.22
16	6.00	1.93	3636	1.00	1.25
18	6.25	1.95	3636	1.00	1.28
20	6.60	2.00	3636	1.00	1.32
25	7.25	2.15	3636	1.01	1.39
30	8.00	2.25	3636	1.02	1.46
35	8.75	2.45	4000	0.98	1.48
40	9.25	2.55	4000	0.99	1.50
45	9.75	2.65	4000	1.00	1.53
50	10.25	2.75	4000	1.00	1.56
60	11.00	2.90	4000	1.00	1.67
70	12.00	3.00	4000	1.01	1.73
80	13.00	3.20	4000	0.97	1.76
90	13.50	3.35	4000	0.98	1.79
100	14.50	3.40	4444	1.00	1.82
125	16.00	3.65	4444	1.00	1.91
150	17.00	3.69	4444	1.00	2.00
175	18.50	4.05	4444	1.01	2.07
200	19.50	4.30	4444	1.01	2.15
225	22.00	4.60	4444	1.01	2.23
250	22.00	4.70	4444	1.01	2.30
300	24.00	4.80	4444	1.03	2.38
350	26.50	5.15	5000	0.97	2.33
400	28.50	5.30	5000	1.06	2.44
450	30.50	5.50	5000	1.00	2.49
500	32.00	5.65	5000	1.00	2.54
600	35.50	6.00	5000	1.00	2.60
700	39.00	6.10	5000	1.01	2.70
800	42.00	6.30	5000	1.01	2.81
900	46.00	6.40	5000	1.02	2.85
1000	50.00	6.50	5000	1.03	2.89



Godavary	$V_0 = 0.67 d^{0.55}$
Krishna	$V_0 = 0.93 d^{0.52}$

For Shwebo canal, Burma, derived by G.C. Stawell for fine sand :

$$V_0 = 0.91 d^{0.57}$$

Kennedy's theory does not take the width or the shape of the channel or slope into consideration which have to be assumed. The velocity worked out should give the required discharge for the assumed section and at the same time satisfy the Kennedy's equation. *The mean velocity of the channel should never be less than critical velocity.* This involves the calculation of mean velocity by the Chezy's formula after assuming Kutter's co-efficient.

*Lindley derived the following equation :—*

$V_0 = 0.95 d^{0.57}$  for standard silt in lower Chenab canal and also found the relationship between Velocity, Bed Width, and Depth as :

$$V = 0.57 B^{0.355} \text{ and } B = 3.80 d^{1.61}$$

When Kutter's or Manning's formula in combination with Chezy's is used, the section is to be selected as to give the mean velocity slightly greater than the critical velocity obtained from Kennedy's formula; or use Lacey's formulae.

System now recommended is to use the gradients given by Lacey's formula and work out the sections by Chezy-cum-Kutter's formula and test for critical velocity with Kutter's formula.

### 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 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.

Lacey's theory is more satisfactory than Kennedy's and his formulae are now used in preference to Kennedy's for the reasons that his equations give one section for a particular discharge with a fixed silt factor, and take into account all the features which determine the canal section for a given discharge. There is only one value of the velocity, the cross sectional area, the wetted perimeter and the hydraulic mean depth; and are also easier to work out.

#### **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 " $V_0$ " 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  $\frac{1}{2}$  to 1.

If Lacey's formulae are employed for calculating the dimensions of a channel, each dimension should be worked out from one of his formulae, never assume one dimension and then work out the remaining dimensions from his formulae. This will give erroneous results.

Lacey's formulae can be applied only up to certain discharge values below which they give indeterminate



results. These limiting values of discharge are given in a following table. Kennedy's formula may be used for small discharges.

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  $V_0^2/R$ . 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 :

$$f = 0.7305 \frac{V_0^2}{R} \text{ or } 0.75 \frac{V_0^2}{R}$$

A rough qualitative formula for determining "f" for the predominant type of silt transported, useful for new canal projects :

$$= 8\sqrt{d} \text{ or } 1.76\sqrt{M}$$

Corresponds to a max:

$$d = \frac{f^2}{64}$$

size of silt 0.01 m.m. to  
0.257 m.m.

where:

d is mean diameter of silt in inches,

M is mean diameter of silt in m.m.

If the channel is in regime, the value of "f" got from Lacey's formulae will be equal; any variation will indicate the extent to which the channel is out of regime.

(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:

0.40—very fine silt as at Ismailia 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 in head reaches and 0.80 in tail reaches.)  
Channels designed from Lacey's diagrams for  $f=0.8$  will be smaller than those designed with Kennedy's diagrams with  $N_0=0.0225$  for the same slope and discharge.

1.00—Kennedy's standard silt, Bari Doab canals,

1.25—Medium sand as at Griffith

				Size of grain in m. m.	$f$
Silt:	very fine	..	..	0.052	0.400
	fine	..	..	0.081	0.500
	"	..	..	0.120	0.600
	"	..	..	0.158	0.700
	medium	..	..	0.233	0.850
	standard	..	..	0.323	1.000
Sand:	medium	..	..	0.505	1.250
	coarse	..	..	0.725	1.500
Bajri & sand:	fine	..	..	0.988	1.750
	medium	..	..	1.290	2.000
	coarse	..	..	2.422	2.750
Gravel:	medium	..	..	7.280	4.750
	heavy	..	..	26.100	9.000
Boulders:	small	..	..	50.100	12.000
	medium	..	..	72.500	15.000
	large	..	..	188.800	24.000

"Regime Flow in Incoherent Alluvium"—G. Lacey. Central Board of Irrigation Paper No. 20.

**Notation :**

$V_0$ =actual mean regime velocity in a canal called "critical velocity",

$f$ =Lacey's silt factor,

$Q$ =discharge in cusecs,

$P_w$ =wetted perimeter of the channel,

$R$ =hydraulic mean depth. It is also affected by silt grade,

$A$ =area of the channel,

$S$ =bed slope of the channel,

$n$ =Kutter's co-efficient of rugosity,

$N_0$ =absolute rugosity co-efficient. (Also see further).

**Lacey's Standard Formulae and Derivations**

$$1) V_o = 1.17 \sqrt{fR} = \sqrt{\frac{4}{3} + R}$$

When the silt factor is unity, this expression gives on canals substantially the same velocities as Kennedy's equation.

$$2) P_w = 2.668 \sqrt{Q} = \frac{5}{3} \sqrt{Q} \text{ (wetted perimeter formula).}$$

Since the wetted perimeter depends not only on discharge but upon the silt charge as well, the co-efficient 2.668 is not a rigid constant (as was envisaged by Lacey) but varies from 2.10 to 3.20 (which may vary in individual channels as well). The expression gives the value of the minimum stable perimeter in channels in incoherent alluvium (pure sand). The upper limit is about 3.16 and the lower 2.20 for Punjab canals. This formula is being used for regime and non-regime conditions by taking suitable constant.

If the banks are tenacious, the width may be less, if the bed is tenacious, the width may be greater. If the wetted perimeter provided is greatly in defect, the channel will tend to widen in the course of time. Stiff clay banks may prevent it doing so, but wherever the banks are friable, the channel will give trouble. If the wetted perimeter allowed to the channel is too great, there will be a tendency for the channel to berm up and to narrow and deepen itself.

$$3) R = 0.4725 \left( \frac{Q}{f} \right)^{\frac{1}{3}} = \frac{0.734 V_o^2}{f}$$

$$4) \frac{P_w}{R} = 7.111$$

$$5) V_o = 0.141 \frac{P_w}{R}$$

$$6) V_o = \left( \frac{Q \cdot f^2}{3.8} \right)^{\frac{1}{3}} = 0.7937 Q^{\frac{1}{3}} f^{\frac{1}{3}}$$

} Regime velocity  
formula

$$7) A = 1.26 \frac{Q^{\frac{2}{3}}}{f^{\frac{1}{3}}}$$

$$8) V_0 = 3100 \frac{RS}{f} \qquad 9) S = \frac{f^{\frac{5}{3}}}{1844 Q^{\frac{1}{3}}}$$

$$10) S = \frac{f^{\frac{3}{2}}}{2686 R^{\frac{1}{2}}}$$

$$11) A f^2 = 4.00 V_0^5 \text{ or } Q f^2 = 4.00 V_0^6$$

$$12) V = 16.05 R^{\frac{2}{3}} S^{\frac{1}{3}}$$

$$13) V_0 = \frac{1.3458}{N_0} R^{\frac{2}{3}} S^{\frac{1}{3}}$$

The equation (12) applies to channels in perfect regime and has the decided advantage that it applies to all earthen channels in regime irrespective of any rugosity consideration.

The equation (13) applies to channels which are not in regime.

$N_0$  is Lacey's *absolute co-efficient of rugosity* which for a regime channel flowing in its own silt is constant for any particular grade of silt, it does not vary with the size of a channel as Kutter's or Manning's "n" does. Corresponds to "n" in the Manning's equation.

This equation can be used as a substitute for either Kutter's or Manning's formulae in non-regime alluvial channels or channels with rigid boundaries, rivers, etc., with better accuracy and greater facility by adjusting the value of  $N_0$ .

According to Lacey the co-efficient of rugosity, i.e., "n" is related to the silt factor "f" by the equation :—

$$N_0 = 0.0225 f^{\frac{1}{2}} \text{— average for Punjab canals}$$

$$\text{and } \frac{n}{N_0} \text{ will be } = 1.104 R^{\frac{1}{12}}$$

From this relationship the following results are obtained:

Value of "f"	Corresponding value of "n"
0.80	0.0213
0.90	0.0219
1.00	0.0225



$$\text{Manning's } V = \frac{1.4858}{n} R^{\frac{2}{3}} S^{\frac{1}{2}}$$

Values of  $N_0$ :—

0.010—for cement plaster.

0.013—for ashlar and good brickwork.

0.015—for rough brickwork or good stone work.

0.018—for stone work in poor condition.

(Generally used for brick lined channels to allow for weed growth)

0.020—for earthen channels in excellent order.

0.0225—for earthen channels in moderate order.

0.025—for earthen channels in poor order.

0.0275—for earthen channels in bad order.

0.030—for earthen channels in very bad order.

*Values of co-efficient "n" for canals in earth and rock:—*

(Taken from more recent experiments by Fortier and the U.S. Reclamation Service.)

0.015—For canals in indurated clay in excellent condition, constructed with well graded smooth surfaces, or worn smooth by the water, uniform cross section, regular alignment, free from sand, gravel, pebbles and vegetation.

0.0175—For canals well coated with sediment or in stiff tenacious clay soil.

0.0200—For canals in sandy and clay loam soils, in average condition, small variations in cross section, fairly regular alignment.

0.0225—For canals in earth in very good condition.

0.025—For canals in earth in tolerably good order or for canals in mixed compact gravelly soil or gravel ranging up to about 3 inches diameter.

0.030—For canals in rough gullied hard pan with eroded irregular cross-section and large gravels in bed.

0.040—For canals with rough scoured beds with cross-section about half filled with aquatic plants.

0.075—For canals in very poor condition, thick vegetation on the banks trailing in the water.

Channels above 1000 cusecs discharge are designed with 0.020 co-efficient; below 1000 with 0.0225; below 25 with 0.025, and for water courses the co-efficient may

be 0.030. The increase in velocity is very nearly proportional to the decrease in the value of "n".

Table Showing Values of  $N_0$  for Various Materials and Corresponding Values of n for Various Values of R.

Material of Channel	$N_0$	Values of "n"			
		R=1	4	10	25
Cement plaster ..	0.010	0.011	0.010	0.009	..
Ashlar and good brick-work ..	0.013	0.014	0.013	0.012	..
Rough brickwork or good stone work ..	0.015	0.0165	0.015	0.0135	..
Stone-work in poor condition ..	0.018	0.020	0.018	0.0165	0.015
Earthen channel in excellent order ..	0.020	0.022	0.020	0.018	0.017
Ditto. in moderate order	0.0225	0.025	0.022	0.0205	0.019
Ditto. in poor order	0.025	0.0275	0.0245	0.023	0.021
Ditto. in bad order	0.0275	0.030	0.027	0.025	0.023
Ditto. in very bad order	0.030	0.033	0.0295	0.0275	0.0255

$N_0$ —is Lacey's omnibus co-efficient of rugosity.

n—is for Kutter's or Manning's formula.

(This varies inversely as  $R^{1/2}$ )

Equation Nos. 2,3,6,7 and 9 enable the various elements to be determined directly from the discharge. Usually the discharge and side slopes of the section are given and the remaining dimensions are to be worked out. All the dimensions can be determined with the help of equation Nos. 2,3 and 9.

#### Limiting Discharge Values for Lacey's formulae

f	Rectangular section					Elliptical section
	R=0	$R=\frac{1}{2}$	$R=\frac{1}{\sqrt{3}}$	R=1	$R=1\frac{1}{2}$	
0.5	32.25	13.8	14.60	18.80	44.00	6.80
0.75	14.35	6.14	6.06	8.37	19.59	4.03
1.0	8.06	3.45	3.40	4.70	11.00	1.70
2.0	2.01	0.86	0.85	1.18	2.75	0.42
3.0	0.89	0.38	0.38	0.52	1.22	0.19

Classification of particle size (defined as "sediment") in m.m. according to the Central Board of Irrigation & Power, India :—

Clay	..	0	to	1/256	=	0	to	0.0039
Very Fine Silt	..	1/256	to	1/128	=	0.0039	to	0.0078
Fine Silt	..	1/128	to	1/64	=	0.0078	to	0.0156
Medium Silt	..	1/64	to	1/32	=	0.0156	to	0.0312
Coarse Silt	..	1/32	to	1/16	=	0.0312	to	0.0625
Very Fine Sand	..	1/16	to	1/8	=	0.0625	to	0.125
Fine Sand	..	1/8	to	1/6	=	0.125	to	0.167
Medium Fine Sand	..	1/6	to	1/4	=	0.167	to	0.25
Medium Sand	..	1/4	to	1/2	=	0.25	to	0.50
Coarse Sand	..	1/2	to	1	=	0.50	to	1.00
Very Coarse Sand	..	1	to	2				
Granule Gravel	..	2	to	4				

(This is the scale adopted by the International Assoc. for Hydraulic Structures Research and differs from the other scales given in "Soil Mechanics".)

The centre of the medium sand range is represented approximately by a Lacey's silt factor of unity.

*Lacey's Shock Theory*: Lacey introduced another factor in his slope formula for shock due to bends, and irregularities in the channel, which accounts for the energy destroyed due to such conditions. After taking shock into consideration the non-regime flow formula becomes :

$$V_o = \frac{1.3458}{N_o} R^{\frac{1}{2}} (S-s)^{\frac{1}{2}}$$

*Punjab Research Institute Formulae*: with  $f=1$

$$P_w = 2.800 Q^{\frac{1}{2}} \quad R = 0.470 Q^{\frac{1}{2}}$$

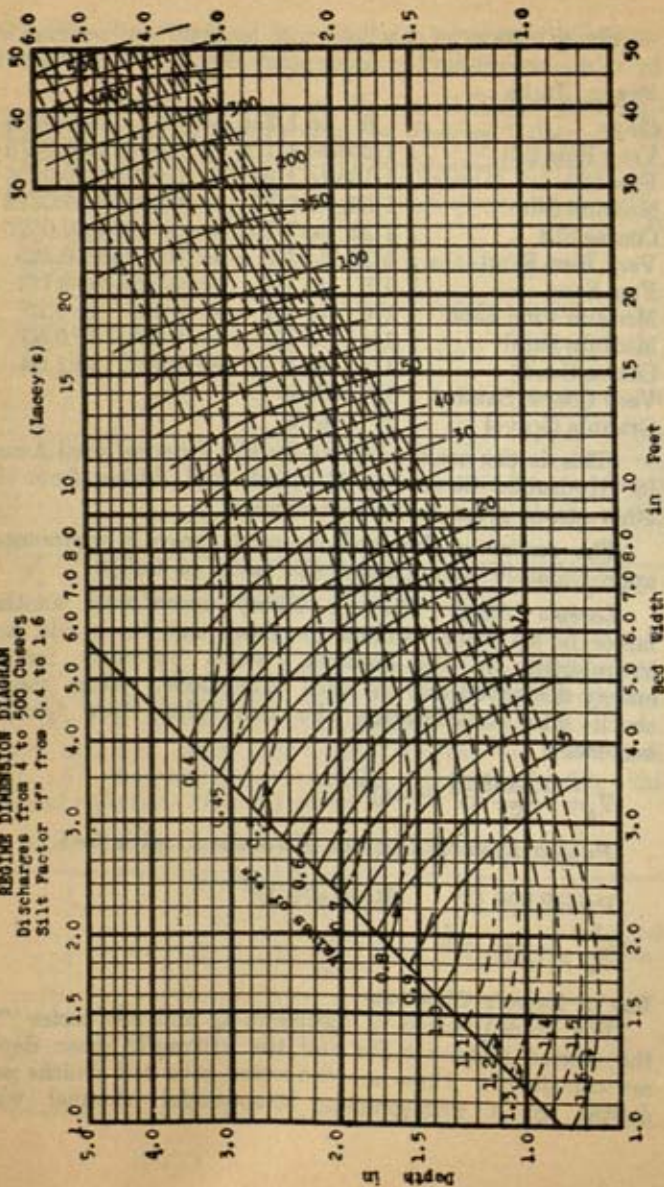
$$V_o = 1.120 R^{\frac{1}{2}}$$

### Use of Lacey's Diagrams

For known values of discharge  $Q$  and silt factor " $f$ " the wetted perimeter  $P_w$  and the hydraulic mean depth are calculated. These are converted into bed widths and depths for a corresponding trapezoidal channel with

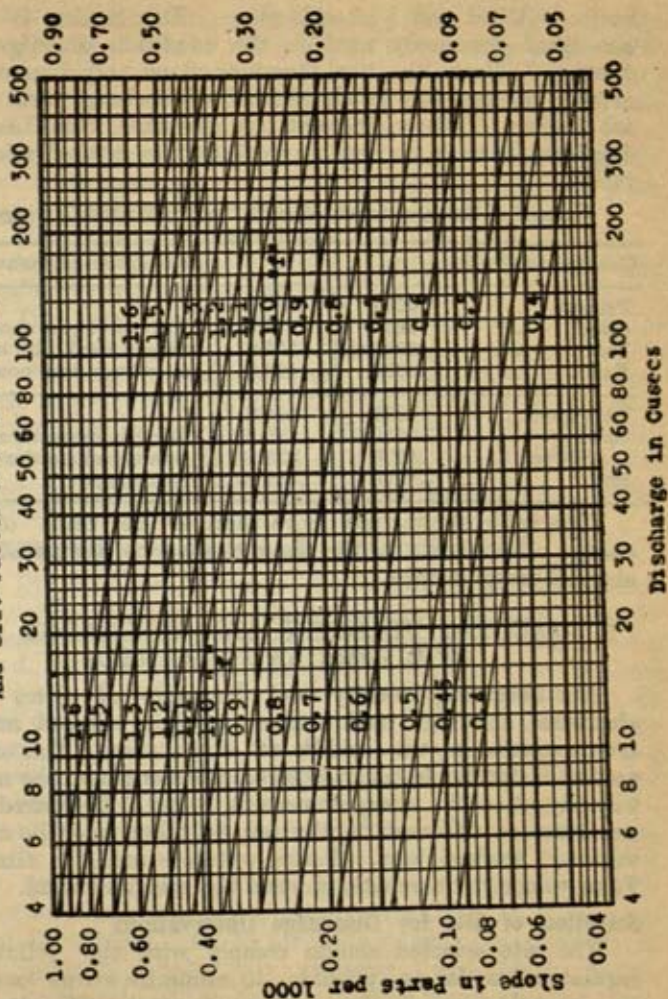


REGIME DIMENSION DIAGRAM  
Discharges from 4 to 500 Cusecs  
Silt Factor "f" from 0.4 to 1.6



(Lacey's)

REGIME SLOPE DIAGRAM  
For Discharge From 4 to 500 Cusecs  
And Silt Factor From 0.4 to 1.6





horizontal bed and  $\frac{1}{2} : 1$  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 velocity ratio).

Values of "s" corresponding to "n" in Kutter's formula :

Channel condition	n	s	Description of channel
Perfect ..	.0250	.000 S	Natural stream channels straight bank, full stage, no rifts or deep pools.
Good ..	.0275	.174 S	
Fair ..	.0300	.306 S	
Bad ..	.0330	.426 S	
Very good ..	.0225	.000 S	Earthen channels and canals under ordinary conditions.
Good	.0250	.190 S	
Indifferent	.0275	.331 S	
Bad ..	.0300	.437 S	

The table shows that more than 40 per cent. of the energy destroyed in the channel may be dissipated by channel irregularities.

### 3. GAUGING VELOCITY AND DISCHARGE OF RIVERS AND CHANNELS

For observing velocity 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 obs-



tructions 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 500 ft. 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 such 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 200 ft.

#### 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 1000 ft. 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 widths of the segments usually vary between 1 and 100 ft. 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 about 750 to 1500 ft.

segments are 50 ft. apart and beyond that width 100 ft. apart over the portion of the section passing 75 per cent. of the total discharge and 200 ft. apart for the remainder. A boat is useful for velocities of about 5 to 6 ft. only; for higher velocities, anchors should be used for 300 ft. 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  $1\frac{1}{2}$  minutes. The length usually taken is 25 to 200 ft. for canals and 500 ft. for rivers. In the case of large rivers the upper gauge is 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-sections 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  $2" \times \frac{1}{4}"$  size or a bamboo of 2" to 3" diameter instead of an oval wooden rod are also used. The rod is graduated in tenths of feet and bottom provided with a flat or round piece of wood or iron of size 4" to 6" 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 10 ft., but bamboo rods have been used even up to 30 ft. depths in low velocities.

**Log Line :** The weight and cord attached to it used for determining depths at observation points where it is impos-



sible 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 10 ft. It is usually wire or a hemp cord graduated into feet by tags of eather 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 4 to 12 lbs. is required for canals with velocities up to 4 ft. per sec. and in small depths. For higher velocities and greater depths, a weight up to about 50 lbs. may be necessary. The load or 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 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 a point  $0.15 D$  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  $2\frac{1}{2}$  per cent only.

In a branch canal of over 50 ft. 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  $1\frac{1}{2}$ .

### Empirical Formulae for Determining Mean Velocity

$$(i) \quad V = \frac{V_s + 2V_d + V_b}{4}$$

$$(ii) \quad \text{Prof. Von Wagner's formula :} \\ V = 0.705 V_s + 0.003 V_s^2$$

$$(iii) \quad \text{Morin's formula :}$$

$$V = V_b + 10.87 \sqrt{RS}; \quad V_b = V_s - 10.87 \sqrt{RS}$$

$$(iv) \quad \text{Bazin's formula :}$$

$$V = K V_s \quad K \text{ is a co-efficient}$$

#### Values of K for Different Surfaces of Channels

R	0.5	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10	15	20	50
a	.58	.65	.71	.73	.75	.76	.77	.78	.78	.78	.78	.79	.80	.81
b	.84	.85	.85	.85	.85	.85	.85	.85	.85	.85	.85	.85	.85	.86
c	.81	.82	.83	.83	.83	.83	.84	.84	.84	.84	.84	.84	.85	.85
d	.74	.77	.79	.80	.81	.81	.81	.81	.81	.82	.82	.82	.82	.83

a is for earthen channels. b is for smooth surface, fine plastered sides. c is for cut stone, brickwork etc. d is for rubble masonry, uneven surfaces.

where :

$V$  = mean velocity in ft./sec.,  $V_s$  = surface velocity,

$V_b$  = bottom velocity,

$V_d$  = mid depth velocity,

$R$  = hydraulic mean depth,  $S$  = sine of slope.

(v) The following co-efficients for working out mean velocity from surface velocity are given in the Trans.

Am. Soc. of C.E., Vol. 66, page 123 as worked out by Grunsky, and have been found satisfactory :  $V = C \times V_s$

Width of stream	}	5	10	15	20	30	40	50	100
Average depth									
C		1.01	.97	.94	.92	.89	.87	.85	.82

The above co-efficients are for sandy beds. For rough stony beds the co-efficients are reduced by 90 to 95 per cent.

(vi) 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	$0.70 \times$ central surface velocity
Minor "	$0.65 \times$ central surface velocity
Water-course "	$0.60 \times$ central surface velocity

Depth on verticals ft.	Values of No			
	0.016	0.020	0.025	0.030
0.5	0.87	0.84	0.82	0.78
1.0	0.87	0.84	0.82	0.78
5.0	0.90	0.90	0.89	0.85
10.0	0.91	0.91	0.90	0.86
20.0	0.91	0.91	0.91	0.88

The mean velocity on a vertical (not near the sides of a channel) relative to the surface velocity is shown in the table.

No—is Lacey's omnibus co-efficient of rugosity.

**Surface Floats.** Wooden discs 3" to 6" dia. 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. In the Punjab a value of 0.89 is used. 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 20 ft. above the starting line 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 100 ft. will be sufficient for most of the channels.

**Velocity Rods.** Are wooden poles of 1" to 2" 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 3" to 6" short of the bottom and about 1½" to 2" 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 0.06 is 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
V mean	0.954	0.961	0.968	0.975	0.981	0.986	0.989	0.992	0.996	1.00
V rod										

$l$  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 obtain 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. Lacey's telescopic discharge 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 2" above the water level. The length is adjustable and can be extended up to 14 ft.

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 the 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 5 ft. and for channels of regular sections. (Rods may be used for depths up to 15 ft. 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 1 or 2 ft. 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 2 to 20 ft. 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.

### Pitot Tube : (Not commonly used)

Is a vertical tube with lower end bent at right angles and is held (partly immersed) at a depth at which the velocity is required. Due to the force of 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 and  $h$  is the height of the water in the tube above the surface of the stream. Elaborate instruments based on the same principle are sometimes used in America.



#### 4. 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 500 ft. of distance and at every change of section. This profile should be a 10 ft. length of the actual completed channel or embankment with its 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 6 inches 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 parallel 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 4 to 6 ft., depth 3 to 5 ft., 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.

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 3 ft. 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 or 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 course, form a most stable structure, but such an ideal soil is seldom available. A dam of homogeneous material throughout 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 4 or 5 inches in diameter and immersing them in still water  $1\frac{1}{2}$  to 2 ft. 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 a small square tank say of about 10 ft. 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 rentive 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 m.m.)	..	60 to 80 p.c. by weight
Silt (0.002 to 0.02 m.m.)	..	12 to 25       "
Clay (below 0.002 m.m.)	..	8 to 15       "

The above proportions will "stabilize soil". If natural soils conforming 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 No. 36 B.S. Sieve). If 5 p.c. cement is added, blocks can be made (see under "Stabilized Soil for Building Construction in Section 7. )

It will usually make a good stabilized soil with 70 p.c. sand and 30 p.c. clay (or silt and clay together). If aggregate is used, the proportions are :—

Sand	50 p. c.	The aggregate should be $\frac{1}{2}$ " downwards well graded.
Clay	17 p. c.	
Aggregate	33 p. c.	

From 45 to 60 p.c. sand should be retained on No. 60 mesh B.S. sieve. The clay content should be more in dry areas and less in wet areas.

Coarse gravel	59 p. c.	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 p. c.	
Sand	9 p. c.	
Clay	12 p. c.	

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 as 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  $\frac{1}{3}$  to  $\frac{1}{2}$  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 the 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 9 ins. to 1 ft. (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 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 3 inches in thickness. Compaction of banks can also be done by camels or bullocks, in which case the layers of earth may be 6 to 8 inches.

**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 useless. 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 rammod 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 foot (of loose material) in height should be made for shrinkage or settlement of earthwork :—

Soft or loose rock, laterite or gravel	..	1"
Firm compact earth	..	1½"
Ordinary loose earth	..	2"
Black cotton soil ..	..	3"

Some authorities prescribe the following specifications for settlement of channel banks and earthen dams:—

Make an allowance per foot of vertical height of one inch on all works rolled and watered, two inches on all works rolled but not watered, and three inches on all



works neither rolled nor 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  $87\frac{1}{2}$  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 Methods of Embankment Compaction with Various Plants

Plant	Silty and heavy clays	Sandy soil	Gravel, shingle or loose sand	Fragmental rock (not over 4" size)
	Thickness of layers			
8-ton sheepsfoot roller or tractor, dumper, etc.	6" to 8"	...	Loose sand should be damped	...
5-ton smooth roller	..	8" to 12"		..
Tractor equipment	..	...	12"	
Heavy smooth rollers	..	...	...	12" to 24"

(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 3 ft. 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 the monsoons continuous longitudinal earth bunds about 9 ins. in height and 1 ft. 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 25 ft., 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 ease and rapidly 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 8 ft. 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 elsewhere). A bank is considered to be safe if the saturation line is covered by at least 2 ft. 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 3 to 5 ft. 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".)

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** have been tabulated elsewhere in this Section.

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, as shown in the illustration at page 17/75. 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 embankment 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 material,



the steeper will be the line of saturation).

The plane of the surface of percolation water is called the *plane of saturation or percolation* and if this cuts the outer face of the bank, visible flow will appear along and below the line of intersection. The hydraulic grade line must fall within the toe of the bank and be covered by at least 2 ft. of soil and which should be much more in the case of river embankments. Rear berms (10 to 15 ft. wide) may be provided to give adequate cover to the seepage line. The hydraulic gradient can be observed by installing a small pipe with strainer, of say, 2" dia., vertically in the embankment, it will have water rising up to this line. The relative permeability of different kinds of soil may be compared by observing the respective percolation gradients under the same conditions. The velocity of percolation is very small when compared with surface flow and varies with the permeability of the soil being proportionately greater in a coarse soil than in fine one.

Core walls of puddle-clay etc. reduce the seepage water and prevent the flow lines from cutting the downstream face of the embankment. Sometimes filter materials are placed on the downstream toe to provide free drainage, which will force the seepage lines down. Filter materials can be placed in several layers, each layer coarser than the last.

The hydraulic grade line is generally as follows :—

For good clay	...	...	...1 in 3
„ good compacted soils	...	...	...1 in 4
„ average soils (sandy loams)	...	...	...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 : (for canal banks)

Where F.S.L. is up to 4 ft. above natural surface—	1 in 4
„ „ 4 to 5 ft.	„ „ 1 in 5
„ „ 5 ft. or more	„ „ 1 in 6

If the foundation of a dam or an embankment is permeable, water will percolate under the structure, which will be steeper as the head is greater. To remedy this, the



base width of the work is made wider or a "cut-off" wall is built in the centre up to a hard bed where available, or a blanket of water-tight material is provided on the upstream side.

Percolation through or under a work does not effect its stability unless it is of sufficient pressure and velocity so as to disturb particles of any portion of the work.

**Free-board.** The height of free-board varies with the importance of the embankment and the provisions for safety essential and upon the height of waves which depend upon the depth, length or fetch of the lake and wind conditions. Violent winds may cause wave dashing to a height of 10 to 15 ft. Height of wave in a reservoir is worked out by the formula :

$$H = 1.5\sqrt{L} + [2.5 - (L)^{\frac{1}{4}}]$$

H is in ft. and L is length or fetch of the lake in miles.

The free-board is generally kept 5 ft. for heights of dams 15 ft. and under and 6 ft. for heights 15 to 25 ft. Above 25 ft. up to 50 ft it may be 7 ft. Free-board also allows for any accidental settlement of the dam.

For embankments of rivers the height of free-board is the same as found adequate for dams, and is generally kept about 5 ft. (and in exceptional cases up to 10 ft.) above the designed high flood level. Free-board should be able to provide for earthwork in emergency to close small leaks.

**Top Width of Dams and Embankments.** The minimum top width of an earth dam is 6 ft. up to 15 ft. height increased to 12 ft. up to 50 ft. height of dam. Empirical formulae for determining the top width in feet :—

$$(i) \quad W = \sqrt{2H} + 3 \quad (ii) \quad W = \frac{H}{5} + 5$$

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	Breadth of water spread	Top width of bund	Free-board
5' to 10'	Under 300'	4'-0"	3'-0"
10' to 15'	300' to 900'	5'-0"	4'-0"
15' to 20'	900' to 1500'	6'-0"	5'-0"
Over 20'	1500' and above	9'-0"	6'-0"

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. Similarly, 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 Site for Alignment of River Embankments.** The most suitable alignment is a matter of judgment 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 erosion 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 waterway 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 1 ft., 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 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 form 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 100 ft. in front of the toe of embankment as the existence of such plantations 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 intertwining twigs of bushes. As a temporary measure, gunny bags full of sand or earth can 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 50 ft., or even 100 ft., from the toe of an embankment, and not nearer than 30 ft. of the toe of a big canal bank, and 10 ft. of the toe of a small channel bank, but if the depth of the borrow pits exceeds 2 ft., the distance should not be less than 15 ft. 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 by 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 1 ft. deep in land acquired temporarily in cultivated areas, otherwise 3 ft. max., and no pit should be excavated more than 5 ft. deep within a distance of 100 yards from the front toe of an embankment. Borrow pits should not be continuous as otherwise they will form a channel. At least 10 ft. wide strip should be left unexcavated in every chain or so. A space of about 6 ft. should be left around all pits for the workmen to pass. Borrow pits should be in multiples of 10 ft. 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 6 ins. deep.

*Borrow pits in beds of channels :* In the case of large channels borrow pits can also be put in the bed leaving 5 ft. 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 1 to 2 ft. below the bed. These pits get silted up in 3 or 4 weeks' running and they form a partly watertight 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 4 ft.; length not more than 40 ft. leaving 10 ft. ridge between two adjacent borrows. Width 4 ft. less than the bed width of the canal.

Pits should not be dug near any masonry works or within 20 ft. of where footpaths or cattle tracks 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 10 ft. whichever is less. Spaces left between the pockets should not be of less than 5 ft. 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 than 1:1 but 1½:1 should usually be adopted. Rough stones are most generally used for pitching with a thickness varying from 9" to 24"

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 3" to 6" 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 2 or 3 ft. 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 inch per ft. 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 3 ft. higher than the High Flood Level and if possible, should not be carried up to a greater height than 10 ft. without giving a berm somewhere.

When concrete or kankar blocks are used they should not be less than one cubic foot 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  $1\frac{1}{2}$  to 1 or 2 to 1 for ordinary soils which may be increased to 3 to 1 for sandy soils. Thickness of revetment generally varies from  $1\frac{1}{2}$  ft. to  $2\frac{1}{2}$  ft. 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 8 to 12 inches diameter and about 12 ft. long bound with tarred ropes at intervals of 4 ft., 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 of 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.

(See also under "Stabilized Soils for Embankments".)



The puddle core wall is generally 4 to 8 ft. wide at the top and both sides batter outwards about 1 in 12 or 1 in 10 to the ground level below which the thickness is quickly reduced to about 2 ft. 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 1 ft. above the H.F.L. and 2 to 3 ft. 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 liable 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 moisture 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 3 to 4 ft. of ordinary earth must be placed over the top of the puddle core to prevent shrinkage and swelling due to exposure to atmosphere changes. Where the height of the dam exceeds 60 ft. a masonry core wall should be preferred to a clay wall. It is the compact 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 unreliable *kalrish* soil. It is keyed 3 ft. into the ground and carried up to the high flood level line, giving 4 to 6 ft. 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  $\frac{1}{2}$  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 constructions.

The clay should be dug up and left exposed to the air in layers not more than 12 ins. 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  $\frac{1}{2}$  in. apart so as to crush any stones or gravels present, before being mixed with water. The scoured clay should be 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  $1\frac{1}{2}$ " (preferably 1") and not less than  $\frac{1}{2}$ ", per linear foot, or it will probably not be sufficiently impermeable to water.

The clay puddle should be consolidated compact and deposited in layers not exceeding 6 inches 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 build-



ing 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 thoroughly 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 20 to 30 ft. (even 100 ft. below the surface is not uncommon and still deeper walls have been built in England) and width 6 to 10 ft., 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 3 to 5 ft., bottom width 4 to 6 ft., side slopes  $\frac{1}{2}$  to 1 or 1 to 1.

Where the cut-off trench is filled with concrete and puddle core-wall built over it, suitable grooves 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 (2" to 5" 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.



### 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 material and downstream of permeable material, 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 a longitudinal collecting drain will facilitate draining of rainwater away from the bank.

A mattress of rubble stones about 2 ft. 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+1)

H = total height of bund = height of water in ft. impounded (h) + foundation depth.

**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 2 to 3 inches diameter stakes or kilas driven in the river bed 2 to 4 ft. apart centres, single for short depths and 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 and 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, is 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 3 to 4 ft. 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 10 to 12 ft. 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 masonry 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.

#### 5. 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 (H.M.D.) for channels of same area and slope, those which have the smallest wetted perimeter will have the largest H. M. D. and the highest 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  $\frac{1}{2}$  to  $\frac{2}{3}$  power of the H.M.D.

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 a 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 :

$$b = 2d[\sqrt{n^2 + 1} - n] \qquad d = \sqrt{\frac{A}{2\sqrt{n^2 + 1} - n}}$$

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— <u>discharge</u> <u>velocity</u>
0 to 1	.707	1.414	1.414	.354	
1 to 1	.759	.938	1.697	.379	
1 to 1	.748	.675	1.996	.374	
1 to 1	.740	.613	2.093	.370	
1½ 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	$\frac{1}{2}$ to 1	1 to 1	$1\frac{1}{2}$ to 1	2 to 1	3 to 1
Bottom width } <sup>b</sup>	2d	1.237d	.828d	.606d	.472d	.325d
Depth ..	.5b	.809b	1.208b	1.65b	2.119b	3.077b
Top width ..			3.416b	5.95b	9.476b	19.46b
Area ..			2.667b <sup>2</sup>	5.734b <sup>2</sup>	11.11b <sup>2</sup>	31.55b <sup>2</sup>
R ..			.604b	.825b	1.059b	1.504b
Sides ..			1.414d	1.803d	2.236d	3.163d

## 6. 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 6 ins. 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 levels at the head of the off-taking channel are marked on the L 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 2 ft. is desirable between the full supply level of the distributary and the main canal or branch, and 1 ft. 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 kept 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  $\frac{3}{4}$ th of the full supply is carried. Some engineers recommend that a channel should command its entire area at  $\frac{1}{2}$  to  $\frac{5}{8}$  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 : 0.48 to 0.64 ft. per mile for main canals and branches and 0.8 to 1.6 ft. per mile 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 a mile and a half water course, may not exceed  $2\frac{1}{2}$  miles. Small size of distributaries are not economical as regards the maintenance cost; a bed width smaller than 3 ft. 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 2 miles 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 2 miles (max. 3 miles), subject to a min. command which gives a discharge of 3 cusecs. 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 6 ins. (min. 3 ins.) and the working head at the outlet is kept 0.75 to 1.0 ft. (min. 6 ins.) 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 10000 may be adopted. The full supply level required at the tail therefore, will be the natural surface level plus 3 ins. A free-board of 6 ins. is usually sufficient. As far as possible the water-courses should be aligned along the field boundaries. In a new irrigation system for undeveloped tracts the area is generally divided into square or oblong blocks of 10 to 15 acres 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 two-cusec outlet is generally the best when the cultivator irrigates flat fields of about  $\frac{1}{2}$  acre area. A discharge of less than one cusec or more than three cusecs is not generally adopted. For optimum conditions the discharge of an outlet should be about five times the area in acres 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 cusec will be higher than in the case of another carrying a discharge of two cusecs. 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 open, 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 cusecs	Bed width ft.	Depth ft.	Slope per thousand	
5.0	2.00	1.70	0.275	The changes in bed-width should occur as near to outlet heads as is feasible.
5.5	2.50	1.60	0.275	
6.0	3.00	1.55	0.275	
7.0	3.50	1.55	0.275	
8.0	4.00	1.55	0.275	
9.0	4.50	1.60	0.275	
10.0	5.00	1.60	0.275	

**Curves in Channels**

Minimum radii of curves on unlined channels :

Capacity of channel	Min. radii
Below 10 cusecs	300 ft.
10 to 100 "	500 ft.
100 to 500 "	1000 ft.
500 to 1000 "	2000 ft.
1000 to 3000 "	3000 ft.
above 3000 "	5000 ft.

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
1500 to 2499 cusecs	1000 ft.	7500 to 9999 cusecs	2500 ft.
2500 to 4999 ..	1500 ft.	10000 and above	3000 ft.
5000 to 7499 ..	2000 ft.		

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 6.5 ft. 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 8 ft. depth	8 ft. to 15 ft. depth
Firm rock .. .. .	$\frac{1}{2} : 1$	$\frac{1}{2}$ to $\frac{1}{2} : 1$
Soft or disintegrated rock .. ..	$\frac{1}{2} : 1$	$\frac{1}{2}$ to $\frac{1}{2} : 1$
Alluvial soil, firm gravel, hard compact earth, hard moorum .. ..	$\frac{1}{2} : 1$	$\frac{1}{2}$ to $\frac{1}{2} : 1$
Tough hard pan .. .. .	$\frac{1}{2} : 1$	$\frac{1}{2} : 1$
Stiff earth or clay well drained, soft moorum .. .. .	1 : 1	1 $\frac{1}{2}$ : 1
Ordinary gravel .. .. .	1 $\frac{1}{2}$ : 1	1 $\frac{1}{2}$ : 1
Ordinary earth, soft clay, dry sand, sandy loam, gravelly loam or loam	1 $\frac{1}{2}$ : 1	2 : 1
Loose earth, loose sandy loam ..	2 : 1	3 : 1
Wet sand .. .. .	2 $\frac{1}{2}$ : 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  $\frac{1}{2} : 1$  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  $1\frac{1}{2} : 1$  slope or with safe slope for the material. Estimates for excavation are made for  $1 : 1$  side slopes but discharge is calculated at  $\frac{1}{2} : 1$  (hor. to ver.) See Illustrations—"Typical Cross Sections for Channels." Lined channels are generally given slopes of  $1 : 1$  to  $1\frac{1}{2} : 1$ . 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 the maximum utility where the canal is in high embankments. It has been described before that the channels are usually excavated with side slopes of  $1 : 1$  which silt up to  $\frac{1}{2} : 1$  slope, this itself gives a minimum berm width of  $\frac{1}{2}$  the full supply depth of the channel.

The berm width usually prescribed is  $2D$  to  $3D$  or  $(\frac{1}{2} D + 4')$  for big channels and  $(\frac{1}{2} D + 2')$  for small channels. For Punjab canals when the N. S. is above full supply level, it is  $1.5$  to  $2D$ , 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  $3D$ . (See the Illustration showing "Typical Cross Sections for Channels"). Some engineers recommend much wider berms, varying from  $7$  ft. minimum for channels of  $50$  cusecs discharge to  $50$  ft. for channels of  $2000$  cusecs discharge. Berms may be omitted in small distributaries. The berms should be  $\frac{1}{2}$  to  $1$  ft. higher than the full supply level and slightly sloping towards the channel.

**Banks** Canal banks have to retain water and withstand the pressure. Banks have been described in detail under "Earthen Embankments and Dams". Top width and slope are generally governed by the hydraulic gradient and the nature of the soil. Bank width is sometimes determined from the following formula :

$$W = d + D + \frac{B}{10} \quad \text{or} \quad D + \sqrt{\frac{B}{D}}$$

where :

W = width of bank ; d = height of silted up bed above natural surface ; D = full supply depth of water, B = bed width of channel.

The following bank widths are generally prescribed for Punjab canals and on Tungabhadra project (South India) for channel discharge in cusecs :—

Punjab		Tungabhadra	
Capacity of channel	Width of bank	Capacity of channel	Width of bank
up to 49	4 ft.	up to 10	3 ft.
50 to 99	5 ft.	10 to 25	4 ft.
100 to 200	6 ft.	25 to 300	6 ft.
200 to 500	8 ft.	300 to 1000	6 ft.
500 to 1000	12 ft.	1000 to 5000	15 ft.

Bank widths prescribed for canals in U.P. :—

Channels carrying over 1000 cusecs	10 ft. (min.)
ditto. 200 cusecs	7 ft. (min.)
Other channels	5 ft. „

#### Service Roads and Inspection Banks

A service road to be motorable should have a minimum width of 12 ft., exclusive of the dowel, and that may be for canals up to 250 cusecs discharge. For higher discharge canals, up to 1000 cusecs, the width of the service road bank (in filling) may be 15, 17, 22 or even 25 ft. according to the cross section, and which may be increased up to 30 ft. for canals over 1000 cusecs discharge. On distributaries and canals 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 the 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 the road made during construction as an interim means of communication. The inspection bank is also called a "Petrol Bank". A *Boundary Trench* or a *Boundary Ditch* is made to mark the boundary of the 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 1 ft. to 1.5 ft. wide on top and at least 1 ft. high with  $1\frac{1}{2}$  : 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 1 to 4 ft. 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 6 ft. for canals and branches and 3 ft. for distributaries (inclusive of the banks). This has been explained in detail under "Hydraulic Grade Line". The top level of pushta should be 1 ft. below the full supply level.

**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.

The following free-boards are generally recommended:—

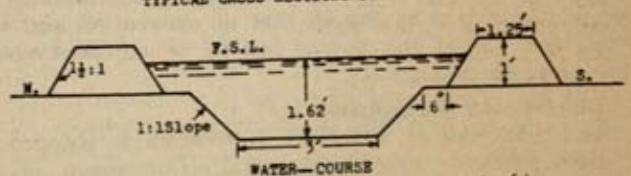
Water-courses	6 ins.	
Minors	1.25 ft.	} min: 1 ft. max: 2 ft.
Less than 200 cusecs discharge	1.50 ft.	
Less than 350 cusecs	2.00 ft.	
Less than 1000 cusecs	2.50 ft.	
Over 1000	3.00 ft.	

The following free-boards have been generally adopted for Bhakra canals:—

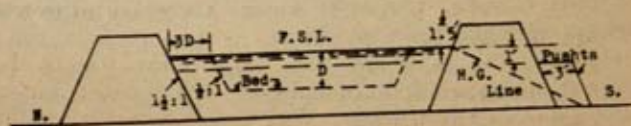
Lined main canals	2.5 ft.	to top of lining
Lined branches	2.5 ft.	"
Unlined branches	2.5 ft.	"
Unlined distributaries	1.5 ft.	"

The height of the free-board should not be less than 1 ft. plus  $1/10$ th of the full supply depth, for fluctuations.

TYPICAL CROSS-SECTIONS FOR CHANNELS



WHEN N.S. IS BELOW F.S.L. BUT ABOVE BED LEVEL



WHEN NATURAL SURFACE IS BELOW BED LEVEL

When N.S. is between F.S.L. and bed level, keep berm =  $2D$ . When N.S. is above F.S.L., keep berm =  $1\frac{1}{2}D$  on the road side and  $2D$  on the other side.

*The boundary ditch* along distributaries and minors should be 1 ft. wide and not more than 18 inches deep; care being taken that it is actually on the boundary.

## LINING OF CANALS

### Advantages of Lining

(i) Considerable saving in water losses due to seepage. (It has been estimated that the use of linings reduces seepage losses from about 10 cusecs per million sq. ft. of wetted area to less than one-tenth of a cusec.) 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 raising of the water-table and the rise of alkaline salts to the surface caused by seepage through unlined canals.

(vi) Prevents the growth of weeds.

(vii) 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 the 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 3 ins. to 6 ins. is spread on the bed and sides and covered with 9 ins. to 12 ins. 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.)

(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  $\frac{1}{2}$ " layer of 1 : 5 cement mortar laid on the consolidated and damped soil in 1 : 3 cement mortar. A layer of  $\frac{1}{2}$ " 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  $12" \times 6" \times 2"$  have been used, the bottom layer resting on  $\frac{1}{2}"$  or  $\frac{3}{8}"$  thick 1 : 5 cement-sand



plaster. The sand-wich layer consists of  $\frac{1}{2}$ " (preferably  $\frac{3}{8}$ ") 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  $\frac{1}{4}$ " 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  $5\frac{1}{4}$ ". One horizontal tile has been allowed in the top of the lining throughout the reach, which may be increased to 3 ft. 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  $\frac{1}{2}$ " layer of 1 : 10 cement-sand mortar and after curing for two days the first layer of 2" thick tiles is laid in 1 : 6 cement-sand mortar. The thickness of the mortar under the tiles is  $\frac{1}{4}$ " on the average. Joints between the two tiles are usually  $\frac{1}{4}$ " thick filled with cement mortar 1 : 6 1 : 3 cement-sand mortar  $\frac{3}{8}$ " 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 10 ft. depth, a mortar consisting of 1 : 2 $\frac{1}{2}$  (cement-sand) with 25 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 interposing a layer of 1/16" 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 1 gall. per 100 sq. ft. 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.)

*Method of Construction :* Success of lining is mainly dependent on the care with which the back-fill has been compacted. The compacted section should extend not

less than 2 ft. 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 penetrates to a depth of 12 ins. in sandy soils and 6 ins. in other soils. Alternatively, 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 25 ft. centres in the bed of the canal along the centre line, and similarly on the side slopes. To get the correct profile in the side slopes, three templates are erected 25 ft. apart. For lining the side slopes the masons work standing on 2 ft. 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 parallel 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 thickness of the sandwich 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 conditions 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 0.5 cusec per million sq. ft. 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	4.10 to 5.35 ft./sec.
Full supply depth	14.50 to 17.90 ft.
Free-board	2.5 ft.

(iii) **Concrete Lining.** This type is generally considered most suitable for big canals if cost is not prohibitive. Usual thickness is  $2\frac{1}{2}$  ins. to 6 ins. 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 8 to 12 ft. 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  $\frac{1}{4}$ " 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.

**Soil Cement Lining.** Stabilized soil with 5 per cent of cement to be compacted in a 3 in. layer and topped with  $\frac{1}{4}$ " to  $\frac{1}{2}$ " 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. Arrangements 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 toe 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.

**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 the F.S. 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.

### **Surveys and Plans for Canal Projects**

For canals, contour surveys to a scale of 1 or 2 inches to a mile, with contours at 5 ft. intervals, may be available from the Survey of India Deptt. Preliminary canal line may be surveyed by running a rapid contour falling at the rate of the 8 ins. to 24 ins. per mile, according

to the nature of the land. Detailed levelling can then be conducted on the final line by taking levels at every 100 to 300 ft. longitudinally according to the nature of the site and the work, and at every 20 or 30 ft. cross-wise. *Reconnaissance* is a preliminary field examination of a proposed project. For preliminary estimating for canal excavations and embankments the depth or height should be taken to the nearest 6 ins. and the distances at 200 to 400 ft. apart.

For the lay-out of water-courses and in the selection of a *chak* (field) the first requirement is an accurate closely contoured map of the area. In the Punjab I.B. a 1 ft. contour map to a scale of 4" to 6" (which may be up to 8") to the mile is usual. Water-courses are designed on contour plans.

Scale for longitudinal sections of a distributary is generally 2" to a mile although sometimes 1" to a mile is also 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 500 ft. apart in case of minors and distributaries and 250 ft. apart in the case of minor branches.

Trial pits should be made in the canal line, about 10 in a mile, to ascertain the nature of the soil and also at the site of the masonry works and canal crossings.

## 8. 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. Mr. Kennedy found that the rate of seepage in Punjab canals from April to September was  $15\frac{1}{2}$  inches depth of water per day., and from October to March it was  $10\frac{1}{2}$  inches per day. Certain canals with discharges from 400 to 450 cusecs were found to lose about 1 cusec per mile. The Bari Doab canal when 18 years old, lost about 12 to 14 per cent of its discharge in a length of 50 miles. On the Ganges canal the total loss sometimes amounts to as much as 56 per cent. The loss in the Irwin canal Mysore is estimated to be 10 cusecs per million square ft. of wetted area. In full scale experiments recently carried out in Northern India, the loss by seepage in an unlined channel was 22.4 cusecs per million sq. ft. of wetted area on the date of filling and reduced to 10.26 cusecs in six weeks. The percentage losses which include evaporation are usually taken as follows, at different stages :—

	<i>Per cent</i>
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	8.5 to 25
(due to un-economical methods)	

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 :

$$K = 5.5 Q^{0.0625} \quad K = \text{absorption loss per million sq. ft. of wetted perimeter; } Q = \text{discharge in cusecs.}$$

Absorption losses on all unlined earthen channels in the Punjab are generally calculated at a rate varying from 3 cusecs per million sq. ft. of wetted perimeter for a channel of 20 cusecs discharge, to 8 cusecs per million sq. ft. of wetted perimeter for a channel of 2000 cusecs discharge, and which may be as high as 15 cusecs. Some engineers now take at the rate of one cusec per foot depth per million wetted sq. ft. within the limits of 4 ft. to 12 ft. depths.

On some channel it is usual to add a further 3 ins. 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.

Losses from distributaries and water-courses with normal soil section, may be taken as follows :—

$$\text{Loss in cusecs per mile} = 0.16 \sqrt[3]{Q} \quad (\text{approx.})$$

When the outlet channel is fully in moorum cutting and the channel flows along a falling contour of say 1 in 1000, the losses are very heavy :

$$\text{Loss per mile per cusec} = 0.53 \sqrt[3]{Q}.$$

The following percentages are generally taken of F.S. discharge for Bombay Deccan canals for loss by seepage:—

F.S. Disch. in Cusecs	Over 100	100—50	50—25	25—15	Less than 15
	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 Madras the allowance per million sq. ft. of wetted surface is as follows :

(i) for rock	..	..	3 cusecs
(ii) for black cotton soil	..	..	5 cusecs
(iii) for alluvial or red soil	..	..	8 cusecs
(iv) for decayed rock or gravel	..	..	10 cusecs

For channels unaffected by fluctuations in ground water table the following average transportation losses are recommended by Shri K.B. Khushalani, in cusecs per million sq. ft. of wetted perimeter :—

Nature of soil	New Canals	Old Canals
Impervious clay loam	4.0	3.0
Medium clay loam with a hard pan layer at 2 to 3 ft. depth	5.0	4.0
Medium clay loam	6.5	5.0
Ordinary clay loam	8.5	6.0
Silty clay loam	9.5	8.0
Silt loam	10.5	9.0
Loam	11.0	10.0
Sandy clay loam. gravelly clay loam	11.5	10.5
Sandy loam	17.0	12.0
Loose sand	19	17
Gritty soil	23	19
Gravelly sand	29	23
Porous gravelly soil	35	29
Gravel	70	35

### 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. Total loss by evaporation in one year may be from 3 ft. to 8 ft. of vertical depth according to the temperature, altitude, wind velocity and vapour tension or humidity but rarely exceeds 0.4 in. in a day in the hottest and driest part of India.

Capt. Cunningham found the following evaporation losses on the Ganges canal :

Month	Feb.	March	May	June	Oct.	Dec.
Evaporation per 24 hrs.	0.14"	0.12"	0.15"	0.12"	0.13"	0.10"

This gives an average of 0.13 inch per day. The canal is 50 ft. wide and 5 miles long with a discharge of 600 cusecs and gave 1.65 cusecs as loss due to evaporation.

There is more of evaporation from an open water surface than from land; more in regions of less rainfall than those of heavier rainfall; more in dry and desert regions than in green and cultivated regions; more in mountains than in low lands (under same conditions). 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 0.12 inch 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—	3"	per month	12"
for 4 hot months—	10"	per month	40"
for 4 monsoon months—	5"	per month	20"

Total ...  $\overline{72''}$  or 6 ft.

Loss of water in tanks from evaporation and absorption in Southern India is about 6 ft. per year and from lakes in Bombay about 4 ft. at full lake level.

## 9. SUITABILITY OF SOILS AND WATER FOR IRRIGATION AND LAND RECLAMATION

(Also see under "Soil Mechanics.")

### Types of Soils

*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 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 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 in-

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 1 or 2 ft. to deep layers of 12 ft. and over. The sub-strata consists of *moorum* or heavy clay. (See "Black Cotton Soils" in Section 6.) The soil contains uncombined calcium carbonate, is rich and productive, except for the deficiency of nitrogen, and 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 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 the 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 salts 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 25 ins. of rainfall; a rainfall of over 3 ins. at a time in a day will wash away lots of surface salts. The application of sufficient quantity 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 low 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 crystalline 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 on 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 detrimental 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 5 ft. 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 limit of capillary action; both the surface and underground 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 3 to 4 feet of the soil by flooding it with some 6 to 9 inches 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 half acre 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 5 to 6 ft. 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 10 ft.

(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.

## 10. 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  $2 \times 1.35 \times$  average annual rainfall; and the minimum rainfall will be  $2 \times 0.69 \times$  average annual rainfall. For three consecutive years, the maximum rainfall of all the three years together will be  $3 \times 1.27 \times$  average annual rainfall; and the minimum rainfall will be  $3 \times 0.75 \times$  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 rainfall. This is called the driest year. The mean of yearly rainfall of about 10 to 12 consecutive bad years (average bad year) may be  $\frac{2}{3}$  to  $\frac{3}{4}$  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 rain-water, 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. It consists of a cylinder 5" in dia. with its base enlarged to 8". A funnel 5" 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 inches 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" 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 50 sq. miles, 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. Such rain gauges are useful for recording "intensity of rainfall."

## 11. 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 6 ins. 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  $\frac{1}{2}$  to 1, else they will fall in when the channel is running. If lowered continuously 3 ins. below F.S.L. 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 irregularities 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 1 ft. 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 6 ins. 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-belled 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 18 ft. 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 2 to 6 ft. 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 relatively clear. Velocities below 2 ft./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 weed growth. Thereafter the weeds tend to persist.

Temperature between 20° C. and 30° 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



1½ to 2 months time 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 sometime 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 2 inches to a mile and vertical scale 1/50.

## 12. WATER ALLOWANCE & 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) *Season 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 of outlet capacity, authorised per 1000 acres 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 channel 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 water-table 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 (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 mile of a channel is not ordinarily in excess of 600 acres and the percentage of Rabi to be irrigated is between 4 to 6 per cent. of the C.A.C. per week for max. period in which one watering is assumed to be completed. Depth of each watering is usually from  $2\frac{1}{2}$ " to 4" 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. It is the area irrigated during a base period divided by



the mean supply utilized in cusecs. *Base period or Base* is the period during which water is supplied to the crop—whole crop season. It is usually reckoned in days. In the case of irrigation from tanks the duty is the number of acres (av.) irrigated by a million c.ft. of water. Duty varies according to the kind of crop, nature of soil, climatic conditions, canal conditions and the method of cultivation, etc. *Full supply duty* of a channel is the area irrigated divided by the full supply capacity of the channel. 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. *Delta* is the total depth of water required by a crop to come to maturity. It is generally denoted by  $\Delta$ . A cusec day on one acre results in a delta of 2 ft. (approx).

From duty or delta for a crop we get an idea of the amount of water required by the crop.

**Relationship between Duty and Delta :**

$$\text{Duty} = \frac{86400 \times \text{Base}}{43560 \times \text{Delta}} = \frac{1.98 \times B}{D} = \frac{2 \times \text{Base}}{\text{Delta}}$$

$$\text{Delta } \Delta = \frac{2 \times \text{Base}}{\text{Duty}} \quad x = \frac{A \times \Delta}{2B} \text{ (approx.)}$$

$\Delta$  = average depth of water in feet poured in a crop,

B = base of duty (number of days during which supply of one cusec runs in order to mature a crop),

D = duty of water (number of acres of crop matured by one cusec),

x = discharge in cusecs necessary to irrigate a given number of acres (A) with a given duty (D) and base B.

86400 is the quantity of water flowing in 24 hours in c. ft.; 43560 is the quantity of water over one acre of land, 1 ft. depth.

A cusec of water running for a day = 2 acre ft. approx.

Acreage/Duty = Discharge in the channel.

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 crop 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 channel). 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 the 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 a few inches deep. 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 channels, and is suitable for crops grown in rows. Since the water is not applied to the entire area, the consumption of water is less.

#### **Advantages and Disadvantages of Tube Well Irrigation :—**

*Advantages :* Isolated areas can be irrigated ; Water is available 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 content.



**Classification of crops :**

(a) Heavy crops (perennial—12 months)—Sugarcane, Pan gardens.

(b) Medium crops (seasonal and two seasonal)—All fruit trees, flower gardens, ground nuts, all vegetables, wheat, potatoes, guinea grass, lucerne.

(c) Light crops (hot weather and seasonal)—Tobacco, cotton, rice, maize, mecca, barley, jowar, peas, hemp and fodder crops.

(i) *Bases of crops (or Base periods) in the Punjab 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.

Total depth of water required in inches for some of the crops (av. Delta on field in inches) :

Crop		Delta	Crop		Delta
Rice	...	48—72	Vegetables	..	18
Sugar-cane	...	48—60	Cereals (Wheat,		12—16
			Barley)	..	
Lucerne	...	36	Maize	...	10
Tobacco	...	30	Fodder	...	9
Garden fruits	...	24	Orchards	...	24
Cotton		20	Pern. Forage		36

The above figures are very approximate and variable according to the soil and climate.

The usual depth of first watering from a well is 3" in a ploughed field and the subsequent waterings are of 2" 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.

In the Punjab water allowance of 2 to 3 cusecs per 1000 acres of C.C. Area is enough except for sugarcane, rice and garden areas for which about double the quantity is provided. In U. P. a discharge of one cusec has been found to be sufficient for a culturable area of about 600 acres, or approx. one sq. mile. A discharge of less than one cusec, especially in sandy tracts, is not ordinarily an economical proposition for irrigation purposes. It follows that where tube-wells are provided they should be spaced about one mile apart. To meet the losses in transit from the head-works to the crops, provision at the headworks should be made for 10 to 20 per cent. extra. (See under "Absorption Losses.")

The intensity of irrigation may be kept about 40 to 60 per cent. in the case of village lands where some means of irrigation such as wells exist, otherwise the intensity of irrigation may be from 60 to 80 per cent giving about 20 per cent of land for rest during the year. Near the cities where vegetable crops are raised, the intensity of irrigation should be cent per cent. (Intensity of irrigation is the percentage of the culturable commanded area which is proposed to be irrigated.) Some land should be allowed to have rest during one year. Intensities of irrigation for different crops vary in different localities.

### 13. 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. 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 outlet 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-modular 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 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 upstream 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 variations 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 F.S.L. of the sill or bottom of pipe of an outlet to the full supply depth of the channel at that 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

minimum 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  $0.2 \times$  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 C.C.A. in the said chak based on the permissible intensity of the irrigation and the water allowance. The maximum discharge of an outlet is 3 cusecs and the minimum 1 cusec. The length of the water-course irrigating the chaks should not be more than 2 (max. 3) miles.

*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 cusec) of discharge of a semi-module for a tenth of a foot 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 17/9).

*Submerged Orifice*—An orifice which in use is drowned by having the 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 0.02 ft. 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 tale) 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.

Rigid modules are : Gibb's; Foote's, Khanna's ; Ghafoor's. Semi-modules orifice type are : Crump's (A.P.M.); Kennedy gauge outlet ; Kent's ; Jamrao outlet ; Harvey-Stoddard ; Inglis' standing wave pipe outlet. Notch type outlets are : Crump's open flume ; Jamrao type open flume. Most of these are not in much use now.

*Gibb's Module*—Is a device for obtaining constant discharge even if the water level varies in the supply channel up to a certain limit. 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 R.C.C. 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 3 cusecs. The gauge is provided on one side in feet and on the 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 1 to 2 furlongs 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 volume, not withstanding variations 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  $0.2 \times$  the full supply depth of the distributary; this is what would be required for an open flume outlet 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 the design of the outlet.

Only that much quantity of silt should be admitted in an outlet as it can carry; but an 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 outfall. 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 2.0 cusecs at the bed level of the channel. The width of the outlet is limited to a minimum of 0.2 ft. 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=0.20 ft. to 0.29 ft.	C=2.90
=0.30 ft. to 0.39 ft.	C=2.95
=over 0.4 ft.	C=3.00

*Standing Wave Flume* consists of a converging transitionally rising flume, a 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 (O.S.M.)*

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 *Crumpp's Adjustable Proportional Module (A.P.M.)* which was subsequently modified and named *Adjustable Orifice Semi-Module (A.O.S.M.)*. 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 A.P.M. and A.O.S.M. is given by the formula :

$$Q=7.3Bd\sqrt{H_s}$$

B is width of the flume (throat); d is elevation of the roof block above the crest (vertical height of opening);  $H_s$  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 3 ft. square on the downstream side of the bank. On the downstream wall of the tank a module is fitted which may be a pipe working free fall, an open flume, an A.P.M. or an A.O.S.M., 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 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 0.1 ft., 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 3 ins., and 6 ins. is considered standard size. The level of the pipe should be kept at least 6 ins. above the level of the highest land command under the pipe. Discharge through pipes is calculated as detailed in Section 14 under "Flow Through Pipes"; friction and entry losses



are taken into account. Total head lost  $h$  is :

(i) Velocity of approach loss =  $\frac{V^2}{2g}$        $d$  and  $L$  are in ft.

(ii) Entry loss =  $\frac{0.5V^2}{2g}$       (iii) Friction loss =  $\frac{4fLV^2}{d2g}$

$$\text{Total head lost} = \frac{V^2}{2g} \left( 1.5 + \frac{4fL}{d} \right)$$

Value of  $f$  is taken

$$= 0.005 \left( 1 + \frac{1}{12d} \right) \text{ or simply } 0.005 \text{ for clean iron pipes ;}$$

$$= 0.01 \left( 1 + \frac{1}{12d} \right) \text{ or simply } 0.01 \text{ for encrusted iron pipes. ;}$$

$$= 0.0075 \left( 1 + \frac{1}{12d} \right) \text{ or simply } 0.0075 \text{ for glazed earthenware pipes.}$$

$$V = \sqrt{2gh} \sqrt{\frac{d}{1.5d + 4fL}} ; \quad Q = AV$$

The co-efficient 0.5 in "entry loss" (or loss of head at inlet) is for an inlet with sharp entry, and this co-efficient is only 0.05 when the inlet is bell-mouthed.

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 distributary.

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 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 6-in. pipes of encrusted iron or concrete and of 12 to 15 ft. lengths. (Also see further). 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.

The following values for co-efficient  $c$  have been approved in Madras I.B. for concrete pipes.

Length/Diameter	Co-efficient $c$	Length/Diameter	Co-efficient $c$
8	0.78	56	0.58
12	0.76	60	0.57
16	0.74	64	0.56
20	0.72	68	0.55
24	0.70	72	0.54
28	0.685	76	0.53
32	0.67	80	0.52
36	0.655	84	0.515
40	0.635	88	0.505
44	0.62	92	0.50
48	0.61	96	0.49
52	0.60	100	0.48

(The same values may be taken for cast iron pipes, and multiplied by 1.11 for glazed stoneware pipes.)



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.

Dia. of pipe in inches	Area of pipe in sq. ft.	Dia. of pipe in inches	Area of pipe in sq. ft.
3"	0.049	10"	0.545
4"	0.087	12"	0.785
5"	0.136	14"	1.069
6"	0.196	15"	1.227
7"	0.267	18"	1.767
8"	0.349	21"	2.405
9"	0.441	24"	3.141

#### *Scratchley Outlet*

This differs from the pipe outlet only at its downstream end where the barrel opens into a 2 to 3 ft. 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 arrangements.

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 outlets 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.

#### 14. MISCELLANEOUS 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 upstream 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 the 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 divide 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. Sill of the head regulator is made at a higher level than the bed of the approach channel, or the sill 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 6 to 8 ft. wide with top covered with R. C. 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 escape 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 the piers 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, 2 to 3 ft. high, for the full width of the main canal which are covered with an R.C.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 regulator 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 tunnels.

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 bed 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 5 to 10 miles 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 F.S.L. provided with vents and gates, having the sill of the opening of the vents about a foot or two below the bed level of the channel.

*Silt Traps :* Pits or basins are formed in the bed at the canal inlets, 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 to 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 500 to 1000 ft. 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 4000 to 5000 ft. Inlets are provided 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 a 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 bed of 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 5 ft. 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 ad-



vantage may however accrue by reducing the width of a short aqueduct. The structure may be built of masonry arches, R.C. slabs or box culverts. Size of abutments and thickness of arch rings may be worked out as explained in Sections 7 and 19. Abutments should be designed to resist saturated earth pressure; the width may be  $1/5$ th 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  $= (\text{height of aqueduct opening} + \text{drop of water level in the aqueduct}) \times \text{weight of water}$ .

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 and this will permit free passage of seepage water. Where, however, the section has been somewhat restricted which does not increase the velocity more than 7 to 8 ft. 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.) R.C.C. 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 drop 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. It is essential to have self cleansing or scouring velocity in siphons, i.e., a velocity of 8 to 12 ft. 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 at page 14/30.

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, 10 to 15 ft., 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.

### 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 pucca 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 become 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 2 ft. 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 sub-soil water and also takes the dynamic action of the falling water. A thickness of at least 4 ft. is provided at its downstream end with an increase at the upstream end. The downstream apron is provided at or below the river 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 wall** increases the creep of the percolating sub-surface flow and decreases 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.



# SECTION 18

## ROADS AND HIGHWAYS

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## 1. GLOSSARY OF HIGHWAY ENGINEERING TERMS

(Based generally on the Indian Roads Congress and British Standards Institution.)

**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."

**Highway**—An important road in a road system.

**Roadway**—That portion of a road (included within the construction limits) ordinarily used for traffic. It includes carriageway and shoulders.

**Street**—A road in built-up area.

**Carriageway**—That portion of the roadway designed and constructed for vehicular traffic.

**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.

**Trunk Road, Arterial Road**—A main channel or traffic route which forms an essential part of the highway system of the country.

**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.

**By-pass Road**—A road to enable through traffic to avoid congested areas or other obstructions to movement.

**Loop Road**—A route formed by a road or a series of roads to avoid an obstruction or provide an alternative way for traffic.

**Ring Road**—A circumferential road built around an urban area to enable free flow of traffic.

**Radial Road**—A road which provides direct communication between the centre of an urban area and the outer districts.



*Detour*—An alternate circuitous road for traffic.

*Fair-weather Road*—A road that can be used by traffic during dry weather only and not during monsoons.

*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.

*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.

*Drive-way*—A way to secure access from a road to private property.

*Blind Alley*—A way or road open at one end only.

*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.

*Island*—A central or subsidiary area in a carriageway, generally at road junctions, shaped and placed so as to constrain and control traffic movement.

*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).

*Weaving Length*—The length of carriageway between adjacent radial-routes around a traffic roundabout.

*Transition Length*—The length of the transition curve connecting a straight length of road with another main curve which may be circular or transitional.

*Fly-over*—A junction so designed that traffic streams are divided to enable them to pass over or under each other.

*Subway*—An underground passage or tunnel to permit the movement of traffic, or to accommodate service pipes, cables, sewers, etc.

*Stock Subway or Cattle Creep*—A shallow subway constructed to permit passage of cattle underneath a road or a railway.

*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.

**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.

**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.

**Base Coat**—An intermediate course between the base course and the wearing coat.

**Sub-crust**—An intermediate layer acting as a cushion between the foundation and the pavement.

**Black-top Surface**—A general term applied to wearing coats or surfaces of roads in which tar or bitumen is used as a binder.

**Carpet**—A surfacing obtained by laying bitumen or tar concrete to a thickness of usually more than an inch.

**Pavement**—Is the hard crust placed on the soil formation after the completion of the earthwork.

**Paving**—Separate blocks or units (usually stone, cement concrete or wood blocks) fitted closely together over a road to serve as a surface.

**Trackways**—A carriageway in which wearing surface is provided on the wheel tracks only (usually of bricks, stone or concrete slabs).

**Creteways**—A carriageway in which a cement concrete wearing surface is provided for the wheel tracks only.

**Median Strip**—A dividing strip in the middle of a roadway.

**Edging**—**Bricks** (or blocks of concrete or stone) embedded along the edges of a pavement to protect the pavement from damage caused by traffic.

**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.



*Catch Drain*—Is a drain provided in the slope of a cutting to intercept the water flowing down the cut slope.

*Side Ditch*—Is a roadside drain or channel provided at the toe of a road bank.

*Gutter*—An open drain constructed along the sides of a carriageway (in town areas) to carry away the water drained from the surface of the pavement.

*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.

*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*.

*Crossfall*—The fall given to the surface of any part of a roadway, at right angles to road length.

*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.

*Ramp*—A short steeply inclined way connecting surfaces at different levels. Generally made for repair platforms. Max : rise 1 in 6, prefer 1 in 7.

*Sag*—The hollow or depression formed by the junction of two falling gradients.

*Summit*—The peak formed by the junction of two rising gradients.

*Lockspit (Dagbel)*—A narrow continuous V shaped cut made in the ground surface along a defined line of demarcation.

*Benching, Stepping*—The formation of a series of small level platforms or steps upon an incline or slope.

*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.

*Embankment*—An earthwork raised above the natural ground by the deposition of material to support construction at a higher level.

*Spoil*—Surplus excavated material.

*Spoil Bank, Tip, Shoot*—An earthwork bank formed by depositing spoil (outside the limits of the works).

*Cut*—The material excavated to make a cutting.

*Cutting*—That portion of the site of a road where the formation has been excavated below the ground level.

*Lead, Haul, Run*—The distance over which excavated material is transported (or carried) for use as filling or to the spoil bank.

*Dragging*—The operation of smoothing out and re-shaping irregularities in surface earthwork by means of a drag. (See further, under "Plant and Machinery Terms")

*Grading, Trimming*—The operation of excavating and shaping the surface of earthworks. The final shaping of earthworks.

*Grubbing*—Uprooting and removing the stumps and roots of small trees, plants, hedges, etc. from the site of the works.

*Stripping*—The preliminary operation of clearing the site of the works of turf, grass, weeds, brushwood and other extraneous material.

*Turf*—The surface of grass land consisting of earth or mould filled with matted roots of grass and other herbs.

*Sod*—A rectangular piece of turf.

*Rubble*—Pieces of stone or broken brick or concrete of irregular size and shape.

*Hardcore*—A consolidated layer of broken stone, brick, slag or concrete in sizes of about half a brick, with some proportion of smaller material.

*Macadam*—Broken stone, road stone or road metal crushed or broken stone of regular size below 3-in. for road construction.

*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.

*Grouted macadam*—A consolidated wearing surface form-



ed 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.

*Bituminous Macadam*—Bitumen or tar macadam.

*Bitumen Macadam*—Consists of bitumen only.

*Tar Macadam*—A mixture of macadam-type aggregates with a tar binder, laid down in a cold state.

*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.

*Aggregate*—For concrete works the word 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 (bituminous) pavements it is meant to include angular pieces of hard crushed stone. The word coarse aggregate is usually employed for material coarser than  $3/16$ " and fine aggregate down up to sand.

*Chippings*—The term is generally intended to include uncrushed gravel as well as crushed rock, of a gauge finer than  $\frac{3}{4}$  inch. (1 inch to  $\frac{1}{8}$  inch according to B.S.S.)

Angular fragments of crushed or broken stone, of size between 0.2 inch and  $\frac{3}{4}$  inch. (I.R.C.)

*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 sharp-edged aggregate or coarse sand.

The word "grit" can be used equally well for broken stone or small stone found in natural state. It gives a suggestion of roughness in the stone and of roughness to the work. Grit includes sand and sharp fine gravel.

*Bajri*—Is a term largely used to denote stone screenings ranging from fine stuff to about 1 inch 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.

*Shingle*—Consists of coarse rounded or water-worm stone, detritus or pebbles larger than gravel and smaller than boulders and is available in hill streams.

*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 inch and which when consolidated yields a coherent layer.

(ii) Stone or gravel of irregular unscreened sizes which may contain smaller material and also sand.

*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.

*Pitch or Coal Tar Pitch* is the black or dark brown solid or semi-solid residue from partial evaporation or distillation of tars.

*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 bituminous 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  $\frac{1}{4}$ ") 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".

*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 275° F. before use to ensure sufficient fluidity and adhesion to stone, as opposed to heating to about 350° F. for bitumens.

*Binder*—Is a term applied to tar or bitumen used for binding road metal.

*Matrix*—The binding material in which the larger particles of mineral aggregate are embedded.

*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.)

*Bitumen*—Is by-product of the distillation or evaporation of crude petroleum either by natural processes 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.

*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.

*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.

*Bituminous Cement*—A general term for bitumenous materials which bind together adjacent solid substances of a suitable nature.

*Rock Asphalt*—A natural rock formation, usually of lime-stone or sand-stone, impregnated with bitumen throughout its mass.

*Lake Asphalt*—An asphalt which as found in nature is in a condition of flow or fluidity.

*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 1 to 2 inches 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 325° to 350° F. on a prepared surface. Chippings are spread over the laid asphalt where the thickness is over  $\frac{1}{4}$ -in or under heavy traffic to reinforce the mastic, and compacted.

*Rolled Asphalt*—A dense mixture of stone, sand, filler and bitumen mixed and laid hot, and consolidated by rolling while still warm.

*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.

*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  $\frac{3}{4}$ " to  $1\frac{1}{2}$ ".

**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 products.

**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.

**Crude oil**—Unrefined petroleum.

**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).

(i) **Blindage**—A material, such as moorum, earth, used for covering the top of a water-bound macadam surface. (I.R.C.)

(ii) **Blindage—Gritting or Dressing**—(Term used for surface dressing)—Spreading stone chips on a road surface after painting it with bitumen or tar.

**Blinding**—(a) The process of applying a loose layer of fine material to fill the voids or interstices in a water-bound macadam surface. (b) The material so used.

**Gritting**—The operation of spreading small broken stones, chippings, or gravel.

**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 200 mesh 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 200 mesh 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 visco-

sity 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 surfacing-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.

The quantity of filler used is about 3 to 4 lbs. per c.ft. of the aggregate. As large a quantity as mix. will carry, may be used, and yet be workable.

*Primer*—A binder of low viscosity which on application to a surface, other than a black-top surface, is completely 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. (I.R.C.)

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 a bitumen 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 1" to 1½".

*Prime Coat*—The initial application of a binder to an absorbent highway surface prior to the construction of a wearing coat.

*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. (I.R.C.)

*Seal Coat 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 ½" and the size of grit used varies from ½" 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.

*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.

*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.

*Creep*—The slow plastic movement of the material in a surface layer in the line and direction of traffic flow or gradient.

*Crazing*—The breaking up of a surface layer through cracking into some irregular shaped areas.

*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.

*Abrasion*—The removal of material from the surface of a solid by grinding or rubbing action. (I.R.C.)

*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. (I.R.C.)

*Rut*—A groove or depression formed in a surface layer longitudinal to the road by the wheels of travelling vehicles.

*Pot-holes*—Marked local depression in a surface layer, roughly circular in plan, arising from the wearing away of material by traffic or by some other agent.

*Flagstone*—A flat and relatively thin slab of natural or artificial stone for pavements subjected only to foot traffic.

*Fender Kerb or Wheel Guard*—Kerb so placed as to prevent the encroachment of, or to secure the constraint of wheeled traffic.

*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  $3/16$  in. size.

*Picking*—The loosening of the top surface of a road by pick axes or similar tools.

*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 0.2 inch prescribed for chipping sizes.

*Blotter*—A covering of a suitable material to absorb excess binder or to overcome bleeding.

*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.

*Ignition point*—(Burning point)—The temperature at which the vapour of a substance takes fire and continues to burn, under specified conditions of test.

*Fascines*—Bundles of grass tied and laid across a sandy track for passing temporary traffic.

#### *Plant and Machinery Terms*

*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")

*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.

*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.

*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 towed by a tractor.



*Scraper*—It consists of a large scoop with cutting edge. It excavates, transports and dumps the material where required. The cutting blade maintains a constant digging depth.

*Drag*—A machine fitted with two or more oblique blades, generally of steel, for scraping off and reshaping irregularities in the surfaces of earth or similar low type roads.

*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.

*Dumper*—A vehicle for transporting excavated material, so designed as to be capable of discharging its load by forward tipping.

*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.

*Jumper*—A heavy bar chisel or drill worked either by hand or by means of a hammer, used in making blasting holes in rock.

*Hopper*—A funnel-shaped storage receptacle, through which material can be measured or periodically discharged.

*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.

*Screed*—A strip of wood or metal, used as a guide for a template or 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.

## 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 and 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 which directly receives the traffic loads from the pavement. The sub-grade is 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.

**Pavement.** The function of pavement is to : (i) Distribute the traffic loads over the soil formation sufficiently to prevent the soil from being over stressed ; (ii) Protect the soil formation from adverse affects of weather ; and (iii) Provide a smooth riding surface. The thickness of the road crust or pavement is related to the traffic and the sub-grade conditions.

**Foundation, Soling or Bottoming.** The function of this course is to spread the traffic loads and the weight of the roadway above (base) sufficiently to prevent over-stressing of the sub-grade. 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

#### **Water-bound Macadam Roads**

(a) The sucking action of pneumatic tyred fast moving vehicles which loosen the road metal enhanced by the abrasive or crushing action of solid tyred bullock carts, the road metal is pulverized and blown off by the winds. A partial vacuum is created under a fast moving vehicle.

(b) The action of wind and rain in destroying the surface of the road.

(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 50 ft. 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.

#### 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 foundations of the road and the traffic. Due to heat, asphalt gets semi-plastic 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. (Also see under "Bleeding".)

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.)

The following are some faults which occur in treated surfaces :—

*Fatting up* (same as Bleeding)—Due to excess binder.

*Striations*—Due to non-uniform distribution of binder.

*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.

*Scabbing*—Due to dislodgment of stone for insufficient binder.

*Bleeding*—The exudation as a liquid of some of the binding material (bitumen or tar) from the surface layer of a treated road. Generally occurs on surfaces that have been given a seal coat. It is due either to an excess of tar or asphalt or to an insufficient quantity of blinding material. Excess of tar or asphalt is probably due to the binder not being hot enough at the time of application and not properly spread out.

To stop bleeding more blindage (or coarse sand, should be applied and continued till the bleeding stops.

*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 of 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  $1\frac{1}{2}$ ". Then roll, spread metal and apply a seal coat. Or alternatively, a premixed carpet of about  $1\frac{1}{2}$ " 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 may result in a surfacing being brittle and having a tendency to fretting or crumbling. Too much binder may cause smoothing and softness. This is checked by binder content determination.

*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, and may be shown by tests on the consistency of the recovered binder.

#### 4. SUB-SOIL DRAINAGE AND MOISTURE CONTROL

The removal of water from road surface 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. Below 3 ft. 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 parallel 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  $\frac{3}{4}$ " to  $1\frac{1}{2}$ ". (Also see in the later 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 3 inches. 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, waterproof 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 15 ft. 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 four feet below the formation level. In the case of concrete roads the base of the slab shall not be less than 12 inches above the anticipated water level in the side drains or the surrounding country. If the water-table stays below six feet, 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 of 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  $\frac{3}{4}$ -in. to 3-in.

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 18" to 4 ft. The drains should be laid not less than 18" and not more than 4 ft. below the formation in sandy soils, not less than 24" in silty soils in wet locations and 3 to 4 ft. in dense soils. Soils with high degree of capillarity 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

6 to 8 ft. below the natural surface. The depth is taken to be at the invert of the drain. Pipes (laterals) should not be less than 3-in. in diameter and need not be more than 4-in. except where the ground is water-logged, or where the fall is less than 1 in 200, or the lateral is more than 500 ft. long when it may sometimes be 5-in. Sizes of mains will be bigger according to the laterals joining them, and may be 6 to 8 ins. 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 feet :—

Nature of Soil For Properties of soils, see "Soil Mechanics"	Depth of invert of drain	
	2 ft. to 3 ft.	3 ft. to 4 ft
Clay ... ..	25 to 30	30 to 35
Sandy clay .. ..	35 to 45	40 to 45
Clay loam .. ..	45 to 55	55 to 65
Loam .. ..	60 to 90	85 to 100
Sandy loam ... ..	85 to 100	100 to 150
Sand .. ..	150 to 200	200 to 300

A thumb rule is known that a tile or drain line will draw 1 ft. on each side for each inch of depth.

Mr. C. G. Elliot gives the following guidance for the spacing of sub-soil drains :—

Close dense soils, largely clay	.. 30 to 40 ft.
Coastal plain lands composed of mixed clays with fine sand and of uniform structure	.. 60 ft.
Alluvial grounds or heavy soils but with granular structure	.. 70 to 80 ft.
Alluvial glacial drift and sandy loam soils with clay sub-soils	... 100 ft.
Sandy lands and soils containing considerable quantity of vegetable matter	.. 150 to 200 ft.

## 5. THE SUB-GRADE AND ITS PREPARATION

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 upon the particular type of binder used where as 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 these 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.	

(Also see under "Stabilized Soil".)

Compaction 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  $\frac{1}{2}$  ton per sq. ft. are not usually suitable for sub-grades under heavy traffic loads. (See under "Design of Pavements".) Patches of soft soil, if any, should be excavated and removed and substituted by good granular soil. Sometimes about 6 inches 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 to 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 soils in cuttings, the best method is to excavate the soil to a depth of 2 ft. below the final formation level and then replace it and compact it in 6-in. 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 1 ft.

It is considered inadvisable to compact cohesive soil sub-grades below their optimum moisture content in cases where they are likely to be subjected to the ingress of moisture. (See under "Rolling").

## 6. ROAD ROLLERS

Rollers vary in weight from 2 tons for bullock rollers to 15 tons for steam rollers. (Cylindrical hand driven rollers are lighter than 2 tons). 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 rollers are generally used in India :—

**The 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 3 ft. in dia. and 4 or 5 ft. long. Ground pressure is about 100 lbs. per sq. in.

**The Sheep-foot Rollers :** These rollers consist of a hollow cylindrical steel drum on which projecting feet like truncated pyramids or staggered teeth are mounted. Various makes are available having different diameters



and widths of drum and different lengths and shapes of feet. The most common type is the one having two drums 4 ft. wide and 3 ft.-6 ins. in diameter with feet 8 ins. 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 9 ins. thick. The soil is supposed to be consolidated when the impression by the projecting teeth is not more than  $\frac{1}{2}$ " 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. When loaded they can weigh up to 75,000 lbs. for a 10-ft. width. Pressures on the feet vary from 60 to 100 lbs. per sq. in. for light rollers and up to 400 to 1000 lbs. per sq. in. for giant rollers.

**Pneumatic-tyred or Wheeled Rubber-tyred Rollers :** Not much used in India. These rollers are generally 6 to 10 tons weight and fitted with a number of wheels (up to about 15). Used for compacting cold laid bituminous pavements, soft base course material or layers of loose soil. These rollers are 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. These rollers are particularly efficient when used to finish off the embankment compacted by sheeps-foot roller or on loose sandy soils. Motive power is provided by a tractor.

### Smooth-Wheel Rollers

**The 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 suit-

able for compacting base courses of hard material. Ground pressure is 140—240 lbs. per sq. in. Usual weight is 4 to 8 tons.

**The 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 10 to 15 tons.

The two types (steam and diesel) are very much alike, the difference being mainly in power unit. These rollers are also manufactured to work on kerosene or petrol. Adjustable weight devices are available which can be fitted to the wheels so that the rolling pressure per inch width 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. Diesel rollers are cheaper in running cost.

#### **Working and Care of Steam Road Rollers :**

**Labour**—One trained driver and one cleaner are required. The driver should be able to do minor repairs such as packing glands, making joints, tightening up of bolts and nuts and even the adjustment of water in brasses.

**Fuel**—If wood fuel is used instead of coal, the box and fire bars should be specially adapted. The fire should be banked up at night when the roller is in work. This is done by damping down the fire with small coal and half burnt ashes, closing the damper and placing a cover of some sort over the chimney top. The fire will then smoulder during the night keeping the water in the boiler



hot and save fuel required in raising steam in the morning.

**Water**—The presence of salts in water is readily noticeable by slight incrustations appearing at the glands, cocks or joints. Muddy and salty waters damage the valves and cylinders.

**Boiler**—The boiler tubes should be kept clean inside by being swept once or twice daily with a tube brush. If kept clean they will not crust up with scale which requires special tube cleaners to scrape out, and which interferes with the steaming capacity of the boiler, if allowed to remain. If the tubes leak at their junction with the tube plates as a result of bad feed water or from working for long periods on a steep gradient, and not keeping sufficient water in the boiler to cover the ends, they should be expanded with a tube expander, but repeated use of this should not be encouraged. Leaky tubes invariably indicate defective working in some direction.

Weekly or fortnightly, according to the cleanliness of the feed water used, the boiler has to be washed out and half-a-day should be allowed for this work including filling the boiler again, packing glands, etc., and raising steam. The plugs or mud-hole doors at the bottom of the fire-box must all be removed and all mud and deposit thoroughly washed out from the four sides. If this mud is allowed to accumulate above the level of the fire-bars the fire-box place will become overheated and will bulge or crack and the stays will leak or be broken. The man-hole door in the boiler barrel should be removed once a year and oftener if the feed water is bad, and as much cleaning and scaling done as possible. The fusible plug which is provided in the fire-box crown plate of every boiler should be taken out and, if encrusted, carefully cleaned at least once a year, oftener if the feed water is saline. If there is slightest doubt about the condition of the *fusible plug*, a new one should be fitted and the old sent to be refilled. An encrusted fusible plug which may then not melt when the boiler is short of water is a serious damage. Always keep a spare fusible plug.

**Steam Pressure**—Most compound rollers work at 180 lbs. per sq. in. and most single cylinders at 140 lbs. The

boiler should work at or nearly at its full working pressure continuously. There is no excuse for repeated blowing off from the safety valves which is wasteful and noisy.

*Reversing under Steam*—The engine should not be reversed without first shutting off the steam. There is no harm done in reversing a roller when under steam provided it is travelling quite slowly, but it causes slight wear and tear, and may possibly be responsible for a breakage, if done at high speeds.

Great care should be taken to see that the water does not get low in the boiler. When going on a climb see that the water level is maintained, this can be done by adjusting the feed water pump valve before starting. Neglect of this precaution often leads to melting out of fusible plugs.

The gauge cocks must be kept perfectly tight and clean and the test cocks should be blown through occasionally, when testing the water in the gauge glass. Sometimes the gauge glass is not a true indication of the quantity of water in the boiler because of the passage being choked. Empty gauge glasses are also often mistaken for full ones. This all should be guarded against.

*The Max. grade a roller can climb is 1 in 5.*

When raising steam in the morning in a cold boiler, the lower cock in the funnel should not be opened and used unnecessarily to hurry up the process as it will only cause the joints in the boiler to leak. The blower cock is intended for occasional use only and is useful to lighten a dull fire when the engine has been kept standing. Raising steam will take 1 to 1½ hours depending on the fuel and whether the boiler is cold.

When first starting the engine the cylinder cock should be opened to get rid of the condensed steam (water) from the cylinders.

The setting of the safety valve should not unnecessarily be tempered with by the drivers. The driver may slightly ease the valve from time to time, by lightly tapping the safety valve lever, to ensure that the valve is free.



List of Stores required for a day's work for a 10-ton Steam Road Roller working 9 to 10 hours. (Compound Roller)

1. Steam Coal	...	10 to 18 c. ft. (6 to 11 mds.) 14 to 20 maunds		A single cylinder roller will need 20 to 30 per cent more fuel.
(b) Wood fuel (if used instead of coal)				
2. Fire-wood	...	7 to 10 seers		Lighting fire
3. Kerosene oil	...	$\frac{1}{4}$ bottle	...	Lighting fire and cleaning
4. Cotton waste	...	$\frac{1}{2}$ lb.		
5. Cylinder oil	...	$\frac{1}{2}$ gall.		Lubricating cylinder
6. Engine oil (heavy)	...	$\frac{1}{2}$ gall.		
7. Gear oil	...	$\frac{1}{2}$ gall.	...	Main driving gear
8. Grease	...	$\frac{1}{2}$ lb.	...	Gear water and pump.
9. Water	...	300 galls. for compound and 400 galls. for single cylinder rollers.		Only clean water free from mud and salt should be used

In cold or wet climates grease is used for the gearing. but in dusty country a heavy engine oil is preferable.

A steam road roller travels about 8 miles per day (while working).

Compound rollers cost more than single cylinder rollers, but are more economical in fuel and water consumption and are easier to handle.

List of Stores required for washing out of a Steam Road Roller once a week:—

1. Kerosene oil	...	$\frac{1}{2}$ bottle	...	For cleaning brass mountings
2. Gear oil	...	$\frac{1}{2}$ gallon	...	
3. Cotton waste	...	1 lb.	...	
4. Common oil	...	$\frac{1}{2}$ bottle	...	Cleaning brass fittings
5. Boiled oil	...	$\frac{1}{2}$ bottle	...	For lead joints
6. Wool	...	1/32 lb.	...	For eill cup syphons
7. Copper wire	...	1/32 lb.	...	For joints and trimmings
8. Red lead	...	$\frac{1}{2}$ lb.	...	For boiler joints
9. White lead	...	$\frac{1}{2}$ lb.	...	do
10. Flax	...	$\frac{1}{2}$ lb.	...	For packing glands
11. Bar soap	...	4 pieces	...	For washing and cleaning
12. Special grease	...	$\frac{1}{2}$ lb.	...	For steering gear
13. "Sticky" grease	...	$\frac{1}{2}$ lb.	...	In grease gear for steering and pump gears

Asbestos jointing mill board for mud holes will be required and a sheet 4 ft. sq. and 1/16 in. thick will be sufficient for about one year. 1 lb. assorted sizes of packings for glands should be sufficient for one month. Fine copper wire for oilers (oil cup syphons) 2 or 3 oz. per year will be required.

### Dimensions of Steam Road Rollers Old British Manufacture Types

Particulars	6-ton	10-ton	12-ton	15-ton
Diameter of front wheel	...	3'-9"	3'-9"	3'-9"
Diameter of each rear wheel	...	5'-3"	5'-3"	5'-3"
Width of front wheel	3'-0"	4'-0"	4'-0"	4'-3"
Width of each rear wheel	1'-3"	1'-2"	1'-4"	1'-8"
Distance between rear wheels c/c	4'-0"	5'-4"	5'-0"	5'-9"
Wheel base c/c	10'-0"	9'-8"	10'-0"	12'-0"
Load on or weight of front wheel	2.6 tons	5 tons	5.5 tons	5.5 tons
Load on each rear wheel	3 tons	4 tons	4 tons	6 tons

The weight of the roller in working in about 2 to 3 tons (varying from 20 to 30 per cent) above its normal weight. Width of contact between roller wheel and road surface is 3 inches.

### Particulars of 8-ton Three-wheel Smooth Roller

Rolls	Weight	Diameter	Width	Load (lbs. per in. width)
Front	2.9 tons	42"	34"	188
Rear (2)	5.4 tons	47"	20"	300

### Descriptions of Indian made Steam and Diesel 10-ton Road Rollers

Particulars	Steam	Diesel
Wheel base (distance between axle centres)	9'-10½"	10'-8½"
Overall length	17'-1½"	20'-3"
Overall height	10'-8½"	10'-1½"
Front roll diameter	3'-8"	3'-11"
Front roll width (each)	2'-1"	1'-5"
Front roll rim thickness	0'-1½"	0'-1½"
Rear roll diameter	4'-9"	5'-3"
Rear roll width (each)	1'-2"	1'-6"
Rear roll rim thickness	0'-1½"	0'-1½"



Total rolling width		
(without extensions) ... ..	6'-3½"	5'-4"
(with extensions) ... ..	7'-1½"	5'-4"
Turning circle (min. dia.) ... ..	20'-9"	38'-0"
Max. grade roller can climb loaded	1 in 5	1 in 5
No. of speeds ... ..	2	4
Weight empty (approx.) ... ..	93/20 tons	96/20 tons
Weight in working order ... ..	10 tons	10 tons
Ground pressure exerted per inch		
width of rolls : Front rolls ... ..	168 lbs.	150—280 lbs.
Rear rolls ... ..	370—500 lbs.	350—470 lbs.
Horse Power (at normal R.P.M.)	24	36
No. of cylinders ... ..	1	4

The grade of Fuel used in the above Diesel roller is High Speed Diesel Oil and consumption is about 16 to 18 lbs. per hour under normal working conditions.

### 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 top of 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 10 to 15 lbs. 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 120° and 180° F., and the lubricating oil pressure between 10 to 15 lbs.

(For more details see I.R.C. Journal Vol. XV-3, Jan. 1951 from which the above extracts have been taken).

## 7. 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-ton roller is good; for brick aggregate, 6-ton; and for highways and roads near towns, 10 to 12-ton rollers are satisfactory for hard stone metal up to about 2" 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 6 to 9 inches and that of stone metal 3 to 4½ inches. If more material is to be consolidated it should be done in layers.

The rolling should start longitudinally at the sides and work towards the centre of the pavement overlapping on successive trip by at least one-half of the width of roller



while starting, and the overlapping may be reduced to about one foot while finishing. This eliminates uncompacted strips in the surface. In super-elevated curves rolling should commence from the inner edge and proceed towards the outer.

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 shoulders should be rolled first to lock the stones firmly at the edges and then rolling should gradually progress towards the centre.

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 2 miles per hour for smooth-wheel roller and between 3 and 4 miles per hour with pneumatic-tyred and sheepfoot rollers. For lighter work the speed of a smooth-wheel roller may be increased to 3 to 4 miles per hour. (A steam roller travels about 8 miles 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 10-ft. wooden straight edge should be kept for checking the surface during rolling.

#### Methods of Base Compaction

Material	Weight of Roller	Max. thickness of loose material to be compacted in ins.	Moisture content (per cent)
Hard brickbats or ... crushed rock ...	10 tons or more	8"	Not im- portant ]
Ashes, clinker ...	8 tons	6"	12—20
Stable clayey gravels...	8 tons	9"	6—10
Clayey sands ...	6—8 tons	9"	10—18
Lean-mix concrete	6—8 tons	9"	...

**Recommended Plant for Particular Soils and the Number of "Passes" for Sufficient Compaction :—**(Also see similar table in the Section on "Irrigation" under "Embankments").

Plant	Clayey soil	Silty soil	Sandy soil	Sand
Smooth-wheel roller 7-ton ...	...	8—16	8—16	8—18
Pneumatic-tyred roller ...	...	...	8—18	8—16
Sheepfoot roller	8—16	...	...	...
Smooth-wheeled roller ...	4—8	...	...	...

(Central Road Research Institute, Delhi).

Increasing number of "passes" (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 Steam Road Roller :—(Approx.)**

A steam road roller should be able to consolidate about 700 to 900 c.ft. of hard road metal, well rolled to full compaction, in a day (working nine hours). For soft metal hard rolling is inadvisable and in such cases 2000 to 3000 c.ft. may be consolidated. (If more work is pushed through it may mean, consolidation not fully done.)

For rolling soling, about  $\frac{1}{4}$  mile of a 12-ft. wide road should be done in a day.

For rolling on re-metalling work, 3000 to 4000 sq. ft. are consolidated in a day.

In the case of rolling on pre-mixed metal, the rate of consolidation should not exceed 1400 to 1800 sq. ft. per hour.



**Rolling Surface Dressings : (Approx.)**

16000 to 18000 sq. ft. per day	On $\frac{1}{4}$ " chips, spread at about 5 c. ft. per 100 sq. ft.
20000 to 22000 sq. ft. per day	On $\frac{1}{4}$ " chips, spread at $3\frac{1}{2}$ to 4 c. ft. per 100 sq. ft.
10000 sq. ft. per day	On two coats of surface dressing done together.
5000 to 6000 sq. ft. 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. For steam rolling short lengths are no objection and are less impediment to traffic;  $\frac{1}{4}$  to 1 furlong should be the limit, 6 to 8 trips of the roller are generally sufficient for surface dressing work.

**8. SOLING OR BOTTOMING**

The primary function of soling is to distribute the load over a soft sub-grade in such a way that there will be no sinking of the road crust into the sub-grade.

Soling is generally laid in a trench made in the road and kept one foot wider than the finished road surface. The trench should be dug to have a profile of the same camber as the finished road surface and this trench should be well rolled before laying the soling. The soling should also preferably be rolled before laying metalling, weight of roller depending upon the material used.

All soft places in the sub-grade should be excavated and filled in with firmer spoil and well rammed before laying the bottoming or other foundations. No soling or metal should be laid on a new road 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 unevenness developed made good.

**Stone Soling.** If rubble is used, for sections in good soil, 6" of rubble is specified to be increased to 9" if the sub-grade is of poor soil. No stone should be less than 4" thick or less than 8 lbs. in weight or more than the depth of the soling in thickness and not to exceed in length or breadth twice its thickness. Stones should be hand packed as close as possible and bedded firmly with their broadest face downwards and greatest length across the roadway, and voids filled in with chips or small stones. Gauge pegs are driven in to indicate the thickness of the stones to be laid. The interstices are filled up to the level of the gauge pegs before rolling and any hollows formed are filled in during the process of consolidation, so as to conform with the gradient and camber and leave an even finished solid surface.

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 thickness should be laid over the rubble to fill the interstices in it and to give the necessary depth of soling.

Some engineers are of opinion that if a layer of sand or small size gravel, about 4" 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 half to one inch of sand is recommended under brick soling. Where brick-soling is specified, the bricks are laid flat or on edge with close fine joints. 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. 9-in. (one brick) profiles are made at right angles to the length of the road at 8 to 9 ft. intervals and one in the centre lengthwise. Edging is made with single bricks laid on edge or on end, parallel to the road. The profiles are first made and then cross bricks filled in. Soling with flat bricks may be laid in single or double layers, breaking joints, with a layer of sand between the courses to act as a



cushion. In places however, where the soil is loamy, the soling may be brick-on-edge as this sub-grade affords good support to the bricks. After laying bricks, earth or sand should be spread over the soling to a thickness of 1 inch so that the joints of the soling may be filled up by the earth or sand working in. This earth must be allowed to remain as a protective covering to the soling until such time as the road is metalled.

The soling bricks should be fully burnt or a little over-burnt. The 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". Brick soling should be rolled with a light roller of say 6 to 8 tons and brought to camber.

The bricks are laid either to project 6" beyond the edge of the metalling or with a row of bricks placed upright on end to project from the face of the soling by an amount equal to the thickness of the metalling so as to act as a kerb to it and prevent the metal from spreading. Alternatively, two parallel bunds of clay puddle, or bajri or metal mixed with clay, 9" wide and of the height of metalling are made along the outer edges of the area to be metalled. A 12" width of the berms should be made up at the same time to act as backing for the bunds.

*Hard-core* soling may be of overburnt brick-bats, kankar (4" size), laterite, moorum containing pebbles etc., well consolidated into the ground in a trench, and of the required thickness. This type of soling serves well over hard and firm sub-grades.

Sometimes extra thickness is provided at the edges of the road (called haunches) for a width of about 1'—6" and a depth of 6 inches below the bottom of the soling with extra stones, to strengthen the edges which are the weakest part of a road.

## 9. METALLING

### Water-bound Macadam

Anything above 3" of hard metal and 4½" of soft metal should be laid in two layers and each layer rolled separately,

the lower layer being nearly consolidated, not completely so, before the next layer is put on. No blindage to be used on the first layer. The metal should be screened to remove fines from  $\frac{1}{2}$ " to dust which is spread over the surface (top) after first wet rolling as blindage, not exceeding  $\frac{1}{4}$ " thick.

If the water-bound macadam is laid in two distinct sizes of aggregate to be applied in two layers, each to be laid separately and one to fill the voids in the other, (the lower layer rolled before the upper layer is put on), it will make a much better and smoother surface.

The metal is rolled dry until well compacted and there is no appreciable movement (in the metal) when walked upon, or no appreciable wave in front of an advancing roller, and no lines of roller are left on the surface. Dry rolling should preferably be done with a light roller (tandem roller is considered best) as the object of dry rolling is to obtain interlocking of the adjacent pieces of stones by setting up a vibratory motion and not by forcing the stones to fit in as in compaction. Excessive dry rolling should be avoided as it is apt to cause corrugations in the surface.

The metalling is then moderately watered and kept saturated and rolling continued until the consolidation is completed. Just enough watering should be done so as to flush the metal slurry into the interstices and excess water that will soften the sub-soil should be avoided. A rough test for finding if the consolidation has been fully done is : a piece of metal about the size of a walnut is put on the surface and roller passed over it, it will be driven in if the consolidation is incomplete, but will be crushed if the surface has been well compacted. Or, a loaded bullock cart (not of the heaviest type with iron-tired wheels) going over it makes no impression. There should be no creeping of the stone ahead of the roller. Till this is done, no blindage or surfacing material is to be put on. No rolling shall be done where signs of metal crushing are noticed or rolling causes wavelike motions in the base course or sub-grade. Over-rolling should not be done. About 20 to 30 trips of the roller would be sufficient.



Before starting rolling the metal is dressed accurately to camber. No fresh metal should be added once dry consolidation has started, as it is merely crushed and serves no useful purpose. If new metal must be added once consolidation starts, the part of the road must be fully raked up so that the metal is thoroughly incorporated into the body of the road.

When rolling the surface in two halves, a strip of about 9 to 12 inches 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. Where no kerbs exist, one-half the width of the rear roller wheel should overlap the shoulder sufficient times to compact the shoulder firmly against the pavement.

When the first consolidation has been completed, the binding material (bajri or screenings) is spread over the surface and brushed backwards and forwards to fill in the surface voids, and rolling and watering continued to such an extent that the binding material is formed into a slurry and is grouted into interstices. After the road has been fully consolidated, the surface should be covered with  $\frac{1}{2}$ " layer of either sand, fine gravel or bajri and road opened to traffic, preferably four days after. The road is kept watered so that it remains in a damp condition, under traffic for 14 days. 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. After 15 days a light watering and rolling should preferably be done. "Blindage" has been described in detail in the following pages.

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. A thicker

coat of metal is also necessary since consolidation of a thin coat cannot be effected. But, for the road which is to be 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 spreading a thick layer of shale or bajri, and leaving the traffic to do the rest. A road can also be made and consolidated without water with "pre-mix".

If the metal is properly graded the particles interlock by wedging with small size stones and form a well bonded surface. The voids are also filled with clay and stone dust which is forced up from underneath during consolidation. This grouting material has a far better bedding effect than stone dust alone. The bedding of each piece of metal is the most essential feature of the wearing properties of a road as it is the gradual wear of one particle against another which finally accomplishes its destruction. This wear will not occur if the metal is bedded in properly.

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.

**Thickness of stone-metal for water-bound macadam roads:** The thickness of metal depends upon the intensity of traffic, bearing capacity of the soil and the hardness of the metal. Thickness of loose metal generally specified is : 2—4", 1—6", 1—4", which will consolidate to about 6½", 4½" and 3" respectively. Thickness of consolidated metal should not normally be less than 4½" except where the road is to be subsequently treated with bitumen or tar, where it may be 3 ins. with hard metal. (For more details see under "Design of Pavements.") Where iron-tyred cart traffic predominates, a thicker coat and with hard metal should be specified. The intensity of load of a laden iron-tyred cart is more than that due to a 12-ton road roller, per inch width of the wheels.



The maximum size of stone metal should not exceed :—

2½"	for	3"	consolidated	thickness	of	each	layer
2"	for	2½"	"	"	"	"	"

Some engineers recommend that the maximum size of coarse aggregate should be equal to the consolidated thickness of each layer (or pavement), this provides stability, because the larger stones engage the base course and the surface has no tendency to "roll" or "wave".

**Grading of Metal for Water-bound Macadam :**

2"	50%	2"	60%	2"	60%	1½"	60%
1½"	25%	¾"	30%	1½"	30%	1"	30%
¾"	15%	¾"	10%	¾"	10%	½"	10%
½"	to dust 10%						

The sizes are "nominal size"—see under "I.R.C. Standard Sizes of Broken Stone."

The quantity of metal required will be about 32 c.ft. for 4" loose thickness consolidated to 3", plus about 5 c.ft. of blindage material, per 100 sq. ft. of road surface.

**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 metal coat 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 gauges. Three templates should be provided and used with a distance of about 25 ft. between each, but not exceeding 50 ft. A spirit level shall invariably be used with the template to ensure that the edges of the metalling are truly level.

## 10. RE-METALLING

### 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 soil, climatic conditions,

and maintenance. 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 :*

- 4 years for traffic up to 250 tons per day (150 vehicles),
- 3 years for traffic up to 500 tons per day.
- 1 to 2 years for traffic up to 1000 tons per day.

A water-bound macadam road is not considered suitable for traffic above 1000 tons (combined).

*Bituminous treated :*

12 to 15 years for traffic up to 1200 tons 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 about 30 years.

**Scoring**—Consists in roughening the old surface with pick axes prior to spreading the new metal.

**Scarifying**—Consists in digging up the old metalled surface to a depth of from  $1\frac{1}{2}$ " to 3" preparatory to re-metalling. Where the new metal added is less than 3" the work is sometimes termed *Re-sectioning*.

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 4 inches. The life of a remetalled road is only about  $\frac{3}{4}$ th of the life of a road given full new coat.

The old surface is scarified to a depth of about  $1\frac{1}{2}$ " for hard stones and up to  $2\frac{1}{2}$ " or 3" 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  $\frac{3}{4}$ " gauge upwards to the surface. (The rakes should have prongs giving 1" clear between each prong.) New metal of  $1\frac{1}{2}$ " gauge is added at the top of the old to make a total thickness of at least  $4\frac{1}{2}$ ", and formed to required shape and then consolidated and surface dressed. Fresh metal



of the same kind should only 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  $1\frac{1}{2}$ " deep and 12" apart for thin coats and 15" apart for thick coats of metal to provide the necessary key. The lines should be at  $45^\circ$  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 3" wide and 2" deep, at intervals of 2 ft., 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  $\frac{1}{2}$ " to  $\frac{3}{4}$ " gauge and mixture of dust and fine grit are sifted separately. The chippings 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 2" to  $2\frac{1}{4}$ " 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 9" 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 12 ft. wide road, per mile
3" and under	12,000 c. ft.
4"	9,400 c. ft.
5"	7,500 c. ft.
6" and over	5,000 c. ft.

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 down not sooner than 14 days and when the road has thoroughly dried, all the fine stuff must be cleaned out for a depth of  $\frac{3}{4}$ " before the application of surface dressing.

**Blindage** (Water-bound Macadam Roads.—Blindage is used for filling the interstices of metal and forming a smooth running surface. Brick kiln rubbish and screenings are useful. Red gravely moorum and kankar are also good and they have the additional advantage of holding the metal if they penetrate in the voids between metal pieces. A final topping of sand while the surface is damp after the moorum blindage will give satisfactory results. Spreading and brushing of blindage should proceed simultaneously so that local excesses or deficiencies of material can be corrected at once.

The quantity of blindage on metal roads is about



$\frac{1}{4}$ " to  $\frac{1}{2}$ " of thickness, or about 5 c.ft. per 100 sq. ft. Blind ing 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. Blindage is not usually required on moorum roads which, however, may be finished off with a top dressing of sand. The binding material should be free from any admixtures of clay or dust. If the stone is completely devoid of binding properties and no small stuff is obtained from scarifying, earth may be used 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.

When a surface is to be painted, no material should be used for binding or for blinding which is of such a nature that it cakes hard and is, therefore, difficult to remove. Many moorums must be avoided for this reason.

## 11. SELECTION OF STONE METAL

### General Properties of Road Metal

In order to resist successfully weathering, abrasion and fracture due to stresses brought on by traffic, a road metal should be of a close, tough, durable and hard texture. ("Toughness" is opposite to brittleness and is the property which enables the stone to resist breaking when struck with a hammer, and is essential in a road metal to withstand the impact blows caused by traffic.) In addition it should possess good cementation properties so that it may retain a well-locked surface under the repeated shocks of traffic. The last quality is very important for water-bound macadam roads and it is this quality which gives kankar, 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. Each piece should be square and sharp; no round or oblong pebbles, or angular chips or flakes should be accepted. Stones with rounded edges have less interlocking properties than broken stone.

Aggregates with rough surfaces offer greater resistance to displacement under the shearing action of the wheel

load than those with smooth surfaces. Sharp angular particles will be more resistant to displacement than rounded ones.

The worst kind of metal is that derived from round boulders of small dimensions and varying kinds of long exposed rocks. Such metal soon wears into pockets and it should be avoided where possible. When boulder metal has to be used, the diameter of the boulders must not be less than 5", and the gauge of the metal not more than  $1\frac{1}{2}$ ", or proper consolidation will be impossible. In such cases a binding material will have to be used. Maximum possible uniformity of quality should be insisted on. Soft sandstones should never be used. Hard uniform limestone is easily consolidated and gives a smooth surface.

The maximum gauge of metal varies with the hardness and the nature of the material and the weight of the rollers available for consolidation. The maximum size is not usually more than two-thirds of the consolidated layer thickness. The normal max. gauge for stone metal is from  $1\frac{1}{2}$ " to  $2\frac{1}{2}$ ". As a general rule, the harder the material the smaller must it be broken. Soft stones like laterite, kankar, limestone and also vitrified brick ballast where used, should be broken up to 2" to  $2\frac{1}{2}$ " gauge. The harder stones like quartzite, granite, diorite, basalt should be broken smaller, that is, from 1" to  $1\frac{1}{2}$ " size, so as to allow the clay or moorum binder to hold it better. Coarse-grained rocks are not suitable for use in small size.

The greater the individual sizes of the aggregate, the greater will be the stability and strength, because large aggregate have fewer voids than small aggregate. A larger size of metal tends to produce a rougher surface than smaller size but it bears iron-tyred wheel traffic better. It is now recommended by some engineers that the maximum size of coarse aggregate should be equal to the consolidated thickness of the pavement.

It is occasionally necessary to use metal which without the addition of some kind of binding material will not consolidate at all. If the consolidation of such metal is carefully watched, it will be noticed that after a compa-



relatively 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.

#### **Field Tests for Suitability of Road Metal**

The following tests are generally prescribed :—

(i) Crushing; Abrasion or hardness; Attrition or wear; Toughness (Page Impact test); Cementation; Absorption; Specific gravity. Since these tests are carried out in laboratories equipped with the necessary apparatus they have not been described here. The following tests can however be made in the field :—

Examination should be carried out on a freshly broken surface and not 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 greenish tinge is generally indicative of poor quality of stone. The stone should be free from patches of glassy material and from large cavities.

Hardness of 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 with a sharp blow a piece of road metal with a small hammer of about half-pound weight. If the stone breaks readily, especially into several pieces, it is too brittle for road work.

For bituminous surface treatments two qualities 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 loads without crushing.

The first quality varies with different varieties of stones,

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 rocks 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 quartzites 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 sand-stones 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. A piece of stone should not absorb more than one per cent of its weight of water.

#### Size of Materials for Test Sieves-Square Holes

Size of Opening	Wire cloth Wire dia. SWG	Perforated plates, Plate thickness BG	If perforated sheet having circular holes is used for screens, the diameter of the holes should be $1\frac{1}{2}$ times the side of the square specified. Screens shall not be set at a slope steeper than $45^\circ$ to the horizontal.
3"	0	12	
2 $\frac{1}{2}$ "	2	14	
1 $\frac{1}{2}$ ", 2"	4	16	
1", 1 $\frac{1}{2}$ "	6	16	
$\frac{3}{4}$ ", 1"	8	16	
$\frac{1}{2}$ ", $\frac{3}{4}$ "	10	16	
$\frac{1}{4}$ ", $\frac{1}{2}$ "	12	18	
3/16", $\frac{1}{4}$ "	14	20, 18	
$\frac{1}{8}$ "	15	—	



### I.R.C. Standard Sizes for Broken Stones and Chippings for Road Works

Size Standard or Nominal	Wholly passing sq. mesh of size	Wholly retained on sq. mesh of size	
2½"	3"	2"	(a) In the standard sizes of 2½", 2" and 1½", not more than 20% of any sample shall exceed, in its greatest length, the standard sizes plus one inch.
2"	2½"	1½"	
1½"	2"	1"	
1"	1½"	¾"	
¾"	1"	½"	
½"	¾"	¼"	
¼"	½"	1/10"	
Sand	1/10"	200 meshes to the inch	(b) In the standard sizes of 1", ¾" and ½", not more than 20% of any sample shall exceed, in its greatest length, the standard size plus half inch.
Medium coarse sand	1/10"	80 meshes to the inch	

The shape and texture of a stone are defined according to the following scheme :—



### Use of Brick Metal

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 tons per day. Painting considerably increases its strength of crushing under iron-tired traffic. Brick metal when soaked in High Viscosity Tar and placed in 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 2" 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 metal after soaking in road tar, and the inferior classes of metal (slightly under burnt) by absorbing a greater quantity of tar, acquire approximately the same strength as those of superior quality (well burnt or over-burnt).

It has been observed that overburnt brick-metal absorb about 4 per cent, 1st 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, deep red in colour with some proportion of deep blue or black veins. Spongy or vitrified material, as a result of excessive overburning, is useless and should be rejected. (Some over-burnt bricks which are specified as vitrified, Jhama or Khanjar are quite suitable.) "Pajawa" kiln burnt bricks are not considered suitable. The size of brick ballast should be from  $2\frac{1}{2}$ " to  $1\frac{1}{2}$ " well graded for water-bound macadam and down to  $\frac{1}{2}$ " for carpets. For a good mixture efforts should be made to obtain 70 to 80 per cent of over-burnt (suitable quality) bricks.

For a water-bound macadam road a minimum thickness of  $4\frac{1}{2}$ " of brick metal should be consolidated over a kankar or brick soling. The roller used should not exceed



8 tons. Brick ballast should not be dry rolled. 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 2-in. Thick brick metal carpet :—

Brick metal (1" to 1½" size)	16 c.ft. for 100 sq. ft.
H.V. Tar for brick metal	6 lbs./c.ft. of metal
Sand (dry, medium coarse,	8 c.ft. for 100 sq. ft.
H.V. Tar for sand	5 lbs./c.ft. of sand.

(Most of the above information is based on the results of experiments carried out as reported in Technical Papers Nos. 154, and 172 of the I.R.C.)

## 12. USE OF ASPHALTIC BITUMENS AND ROAD TARS

**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.

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, 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 condition 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. 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".

#### **Tar versus Bitumen**

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, generally greater deterioration is produced in tar than in bitumen when exposed equally to weather action, and open textured are more liable to fail by disintegration than close textured. Road tars are more susceptible to temperature changes than are bitumens and hot weather will soften a tar surfacing more than a similar (of the same viscosity) surfacing made with bitumen, and consequently tars can be brought to a spraying condition at lower temperatures than bitumens. Surface dressings made with bitumen are more prone to failure by water displacement than those made with tar; tars generally adhere better than bitumens on wet aggregate. Bitumens fluxed with tar oils have better adhesion than those fluxed with petroleum oils.



Road tars have higher specific gravity and lower viscosity or greater fluidity, as compared with bitumens and these properties give them greater penetrating power and which are more marked during summers. Higher viscosities can generally be used with tar than with bitumen; higher viscosities being used in warmer weather. A primer is not generally needed with tars. The bitumens have a tendency to stay at or just near the surface and the result is a very rich and fat surface. When a primer is used this tendency of the bitumens is less marked and the use of a primer ensures better penetration. Tar is more suited for dense fine-grained surfaces and bitumen for more open or absorbent surfaces. The volume of tar required is about 10 per cent less than that of bitumen for the same type of road work.

The tars make harder surfaces (but such surfaces are brittle) than bitumens and should be preferred for roads in areas where bullock-cart or other hard-tyred traffic predominates. The bitumens make more elastic surface and are better suited for pneumatic tyred traffic. Bitumen is a very homogeneous material and its hardening is very gradual.

Bitumen and tar can also be mixed together and used. An addition of 10 to 20 per cent. of bitumen in tar will increase the consistency of tar.

**Cold Emulsions** have the advantage that they can be laid in cold and damp weather although they are not always successful if laid in wet weather. They are not so efficient, especially under heavy traffic, as hot tar or bitumen and should be used with discretion. They are not also cheap in real cost as half the volume is water.

#### **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 of asphalt at 77°F. 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.

There is a large difference in the temperature of the bitumen, which is at about 350° F., and that of the aggregate which at atmospheric temperature is at about 90° to 100° F. 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 (20/30 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 a 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 shall 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 of which is raised at the rate of 9° F. per minute. The temperature at which the bitumen softens sufficiently to allow the ball to pass through the ring and touch the bottom of the bath which is 1 in. below the ring, is called 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.

**Specific Gravity.** Petroleum Asphalts have specific gravities of 1 to 1.02; Cut-backs have generally 0.93 to 0.98 and Emulsions about 1.0. In practice it is near enough to take 1 gallon to weigh 10 lbs. for all asphaltic materials made from petroleum asphalts. Tar too is about the same.

**Ductility**—It indicates the binding power and adhesiveness. Measures brittleness; if too low, the asphalt will crack. It is the property which permits the permanent distortion of a material without rupture.

**Viscosity**—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

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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

which are too viscous to be tested in a viscometer, are tested by the so called "Float Test".

The physical properties of coal tar upon which the success of the material as a binder in road surfacing depends are, its power to adhere to the stones and its viscosity. If adhesion is satisfactory the strength of the surfacing depends largely on the viscosity.

The higher the viscosity the greater is the strength of the surfacing up to a limit; when viscosity reaches some high value it ultimately imparts brittleness to the surfacing material. The viscosity of the tar 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; higher viscosities being used in warmer weather. A more viscous tar 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. The binder for the first dressing of a water-bound macadam should be of a lower viscosity than that normally applied to tar macadam. Tar macadam (pre-mixed surfacings) to be used cold requires tar of relatively low viscosity in order that it may be spread and rolled at the prevailing atmospheric temperatures. Mixtures made with high viscosity tars remain for a shorter time at a consistency suitable for rolling.

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

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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.



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 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 stones, 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.

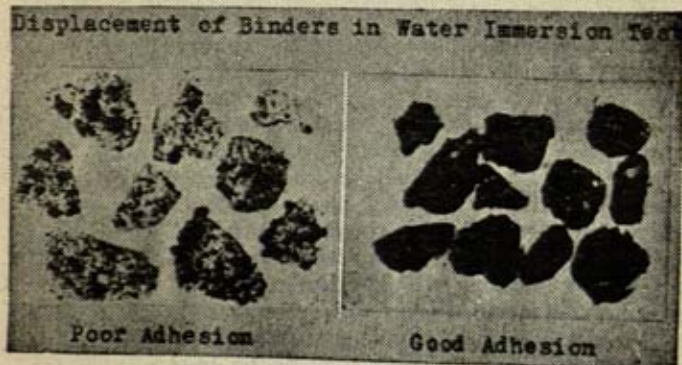
From experiments carried out in India, the following results have been obtained for relative adhesivity of different binders (with certain types of stones):—

**Bitumen**—Better than Road Tars Nos. 3 and 3A.

Road Tar No. 3—Poorer than with bitumen but better than road tar No. 3A.

Road Tar No. 3A—Poorer than all above.

**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.



### Properties of Road Tars

#### Correct Grade of Tar for Surface Dressing Work :

Shalimar Road Tar No. 3 is more fluid than Tar No. 3A and should be used on dense and compact surfaces under normal conditions.

Use Tar No. 3 A (a more viscous grade) where :

- (i) The base is of open texture, or of brick or kankar water-bound macadam.
- (ii) The aggregates show a low affinity for binder.
- (iii) The road is subjected to fast and heavy traffic.
- (iv) The area is subjected to heavy rainfall.

3 A is used for tack coats and pre-mixing of thin carpets. High Viscosity Tar (H.V.T.) is recommended for general pre-mixing.



## Properties of Hot Application Bitumens

Penetration	Suitable for work
8/15 (240°F. melting point.)	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.

Trade Name	Manufacturer	Application Temperature Degrees F.	Penetration at 77° F./25° C.
Mexphalte	Burmah-Shell	350-375	20/30 to 80/100
Spramex	"	"	180/200
Stanvac	Standard-Vacuum Oil Co.	"	30/40 to 100/120
Paving			
Hotfix	McLeod & Co.	350—370	30/40 to 80/100
"			40

Latest specifications should be obtained from the manufacturers as they sometimes change the nomenclature or specifications of their products.

## Cut-Backs

Trade Name	Manufacturer	Application Tem. Deg F.	Penetration at 770 F.	Suitable for work
Shelspra B.S.	Burmah-Shell	325-340 300	300/350	Surface paintings, semi-grout, pre-mixing (low viscosity); will coat cold chippings in drum mixers.
Shelmac B.S.	"	275-300	120/150	Pre-mixing as above.
Shelmac R.C. 3	"	Cold application		Surface dressing and pre-mixing.
Shelmac D	"	275-300		Surface dressing and pre-mixing with wet aggregate.
Socofix	Standard Vacuum Oil Co.	Cold application		Surface dressing, pre-mixing for thin carpets, and patch work.
Socosol	"	Cold fluid solvent for hot asphalts, mixed at the rate of 15 per cent. by weight.		For thinning out or retarding setting of hot asphalts. (Forms an asphalt into a cut back).



## Emulsions (Cold Application)

Trade Name	Manufacturer	Suitability for work
Colas	Burmah-Shell	For surface painting and grouting. Can be used with damp stone. Surface to be damped with water before application.
Colasmix	"	For pre-mixing carpets and ditto.
Stanvac Emulsion No. 3	Standard Vacuum Oil Co.	For surface painting.
Emulsion No. 6	"	For pre-mixing carpets up to 1½" thickness, semi-grout, seal coats, wet weather patch repairs.
Colfix	McLoad & Co.	For surface painting, pre-mixing, seal coat, repairing potholes, etc.

## Primers

Trade Name	Manufacturer	Application Tem. Deg. F.	Suitability for work
Shell Primer No. 1	Burmah-Shell	150	Penetrates less than Shell Primer No. 2. May be used as a combined primer and surfacing material for open textured roads.
Shell Primer No. 2	"	100 or sun warmed	For priming laterite, kankar, earth and similar types of roads.
Socofix Primer cut-back	Standard Vacuum Oil Co.	Cold application	For priming water-bound roads or for water proofing sub-soil when water-table is high.
Liquid Asphalt No. 2.	"		For dust laying, low cost painting, etc.

### 13. GROUTING OR PENETRATION

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.

Surface dressing provides only a thin water-proof and shock absorbing film of the binder to the surface of the road, but it does not reinforce the structure of the road. The impact and shear forces set up by heavy bullock-carts and similar traffic are often sufficient to dislocate the structure of a water-bound macadam road. Surface dressing is capable of giving good service under medium traffic conditions only.

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**. In the full-grout, the consolidated thickness is generally about 3", according to traffic conditions. On most district roads in India a 3" grouted thickness is quite sufficient provided there is at least 5" of good hard metal (including soling) in the base. In the semi-grout the consolidated metal thickness usually is about 2". Semi-grout is mid-way between a full-grout and a surface-dressing and can be used on roads where the traffic is not very heavy. An effort is made to cut down the amount of the binder and to reduce the interstices between the stones which can be done by either :—

(i) Hard rolling the metal, thus decreasing the interstices between the stones : (ii) using graded aggregate - (iii) brushing sand and gravel or stone dust into the interstices to fill them about half way up, to prevent drainage of the binder; (iv) 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 grouting. Emulsions which are more fluid than hot binders penetrate between the faces of stones better, especially in cold climates. The 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".

For grouting with a viscous binder (hot asphalt or tar) and to enable the binder to penetrate easily, it is essential that the interstices between the stones should be fairly open. For this it is necessary to use stone of uniform size without much percentage of smaller material which might get into the interstices and block the easy flow of the binder although a graded stone surface is more dense and more stable. For this reason the stone should be rolled only very lightly (with a 10 to 12-ton roller) just sufficient to give an even surface conforming to the required camber and this rolling must not crush any metal and block the interstices; 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.

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  $\frac{1}{4}$ " to 1" (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 surface of the road as "blindage" to fill the surface voids between the large stones. A thorough rolling should be given with a 10-ton 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. After 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. After an interval of about 10 days (some engineers prefer the next day, especially when the weather is uncertain) the surface is swept clean and a seal coat provided if necessary and surface rolled again. Three or four trips of the roller will be sufficient. No seal or wearing coat need be given until necessary to prevent wearing down of the grouted crust to penetration of water. A seal coat may not be required if the blindage material contains a proportion of smaller gauge chips ( $\frac{3}{4}$ " to  $\frac{1}{8}$ ").

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

Since emulsions can penetrate through small interstices as has been said before, advantage is taken of this quality and graded aggregate is consolidated to a considerable degree before the application of emulsion, instead of uniform aggregate and very little consolidation as with hot binders. As emulsions are not suitable for heavy traffic conditions they are used only for light traffic roads and for which a 2" grouted thickness is considered sufficient.



Table of Quantities for Grouting Work per 100 sq. ft. of Road Surface

3" Grouted thickness			2" Grouted thickness		
Stone metal		Hot Binders	Stone metal		Hot Binders
Gauge	Quantity		Gauge	Quantity	
Uniform gauge 2" nominal size (2 1/2" to 1 1/2")	27 to 30 c. ft. lightly rolled	165 to 190 lbs. 30/40 penetration Bitumen or Tar No. 3A	Uniform gauge 1 1/2" nominal size (2" to 1")	18 to 20 c. ft. lightly rolled	110 to 140 lbs. 30/40 penetration Bitumen or Tar No. 3A
Blindage 1/4" nominal size (1" to 1/2")	5 to 6 c. ft.	80/100 penetration	Blindage 1/4" nominal size (1/2" to 3/8")	5 to 6 c. ft.	80/100 penetration
Seal coat (where necessary) 1/4" nominal size (1/2" to 1/10")	3 1/4 c. ft.	25 to 30 lbs.	Seal coat (where necessary) 1/4" nominal size (1/2" to 1/10")	3 1/4 c. ft.	25 to 30 lbs.

Grouting with Cold Emulsions		
Graded aggregate	220 lbs.	160 lbs.
2" — 60% 1 1/2" — 30% 1/2" — 10%	30-32 c. ft. consolidated to 3"	20 to 22 c. ft. consolidated to 2"
Seal coat (for blinding) 1/4" nominal size or coarse sand	3 1/4 c. ft.	3 1/4 c. ft.
	40 lbs.	40 lbs.

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.

Two coats of heavy type surface-dressing can be given instead of the grouting method mentioned above, which will stand heavy cart traffic. Details about this will be found under "Surface Dressing".

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 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 six-inch 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.

#### Semi-grout Work

Chips for Blinding	Hot Binders	Cold Emulsions
$\frac{1}{2}$ " nominal size 5 to 6 c. ft. (lesser quantity for emulsions)	45 to 55 lbs. (or more) 80/100 penetration (spread at 55 to 60 lbs. per inch depth of penetration desired)	85 lbs.

A seal coat where necessary may be given of  $\frac{1}{4}$ " nominal size chips or coarse sand.

- (i) Lesser quantity of bitumen is required with harder



grades. (ii) Hard aggregates require more of binder than soft aggregates.

Chippings for blindage should be clean broken stone of hard and tough quality. Gravel is not suitable for grouting work.

#### 14. PRE-MIXING

The process of mixing mineral aggregates with bitumen or tar 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 avoided. 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 1 inch to 3 inches 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. Graded aggregate should be used so as to produce a dense surface, when consolidated. The denser the mix, the greater is the surface area of the aggregates composing it and, therefore, a greater quantity of the binder is required to coat it. Sand is used with the graded aggregate to make the surface more dense. Where sand is not used, it requires a seal coat to make the surface water-proof. (See under "Bituminous Concrete.")

**Edge or Hauch Supports** to give lateral support to the metal—

Before laying the premix grooves are cut along both the edges of the road (metal), 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 9-ins. 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.

The macadam should be laid as soon as possible after mixing in cold weather and rolling started soon after a length of about 200 ft. 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. The premix is spread on the road by means of rakes to the desired depth making a 20 per cent allowance for shrinkage during rolling, and is consolidated with a 10 to 12-ton roller. (Some engineers prefer to give first rolling with a light roller, say of 4 tons, 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 above.

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. A well made surface should not have a variation of more than  $\frac{1}{4}$ -in. in 10 ft. width. Any depressions or bumps in the surface should be corrected by concentrated rolling and by the addition of pre-coated chippings carefully raked into position or by skin patches. Fat spots should be cut out and refilled and tamped. The wheels of the roller should be



kept moistened to prevent them from picking up the coated material. Immediately after consolidation sufficient  $\frac{1}{4}$ " dry clean chippings are brushed over the surface to fill all the surface interstices, and well rolled with a light roller.

On dense mixes if a coating of either cement or stone dust is swept over the surface just before the final rolling, it will seal the surface and add to the finished appearance of the work. Or alternatively, a light seal coat of 80/100 penetration asphalt using 1 gall. 5 sq. yds. blinded with coarse sand may be given where heavy traffic is not expected. Chippings mentioned above need not be then spread.

Keep all traffic off the road for 24 hours, and bullock carts for 10 days. After 10 days all loose chippings should be brushed off and surface prepared to receive a seal coat over it. Wheel marks appearing should be removed by spreading blinding over them for at least 14 days. The period between consolidation and seal coat should not be more than one month.

Rounded aggregate make the surface smooth and slippery, especially during rains. Therefore, graded angular aggregates, mixed with sufficient of smaller size, to produce a dense and water-proof surface, should be used in rainy regions. Fine sand also makes the surface slippery. Gravel is not suitable for premix work.

**Joints in Bitumen Roads.** At shut downs and end of day's work, transverse joints are formed by rolling over edge and then cutting back a vertical joint at full depth.

For joining the new work with the old work, cut back the end portion of the old work by 2 to 3 ft. 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.

**Mixing.** Premixing should be done in a power rotary mixer where available. For hand mixing, a barrel mixer can be made of sheet iron, of size that of a tar drum in which 3 blades are welded inside of the shape somewhat like a screw and a removable lid is provided on the circumferential surface. This drum is supported on a stand about 4 ft. high and a 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 handles. If premixing is done in barrels made of tar drums, one such barrel can be used for mixing about 60 to 80 c.ft. of metal per day. Mixing can also be done with shovels or spades on a sheet iron platform. Bajri is best premixed by shovels.

Much bigger outputs and better coating of the metal is obtained in hot weather and when the stone metal has been exposed to the sun. (See also "Mixing" under "Carpets").

For pre-mixing with emulsions  $\frac{1}{2}$ " to  $\frac{3}{4}$ " metal is used which is coated by filling into perforated buckets which are then dipped into tanks containing emulsion. The mixture should be spread on the road and let to remain for 24 hours before rolling to let the water content of the emulsion evaporate.

### CARPETS

(i) A term applied to the wearing surface or top course of a bituminous or tar surface laid in two or more coats. (Refers to Surface Dressing.)

(ii) Pre-mixed materials laid over road surface is also termed carpet. The following descriptions relate to this term :—

*Paving*—Paving may be said to comprise the wearing and base course. *Sub-grade*—The sub-grade is the foundation for the paving. *Base Course*—The base course is a prepared material of hardcore whose function is to act as a weight distribution layer. *Wearing Course*—Protective layer laid on a base course or on any paving, which is applied to withstand the abrasive effect of traffic, usually providing a non-skid surface. A wearing course is made with smaller aggregate than used for the base course. Its composition and thickness will depend largely upon traffic conditions.

Carpets are laid either in a single course, called "thin carpets", and from  $\frac{1}{2}$  in. to  $1\frac{1}{2}$  ins. thick, or are laid in double courses up to a thickness of 5 ins. The cost of a 1 in. carpet is only slightly, if at all, above that of a two-coat surface dressing, and the surface becomes more stable as excess of the binder which results in bleeding and cor-



rugations in two-coat surface dressing is avoided. Where pneumatic tyred traffic is of medium intensity but hard-tyred (bullock-cart) traffic is fairly heavy, light surface dressings wear out rapidly. In such cases it is preferable to have a premixed carpet. 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.

All depressions exceeding one-third the proposed finished thickness should be patched in advance.

### **Tack Coat**

It is essential to give a tack coat over the old road surface if it is smooth or of cement concrete, before laying a thin carpet of say, 1-in. thickness or under, to ensure a proper bond. The tack coat should be applied just ahead of and keeping pace with spreading of the pre-mix. A tack coat may be of bitumen, tar or emulsion, the quantity depending on the texture of the surface and viscosity of the binder. Generally 15 to 30 lbs. per 100 sq. ft. is sufficient. The lesser quantity is required on smooth surfaces and under light sand carpets while more is needed under carpets of stone aggregate and on open-texture surfaces. 15 lbs. will be sufficient over existing surfaced portions and about 25 lbs. over water-bound macadams. The surface of the road should be thoroughly cleaned before applying the tack coat. 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.

A tack coat must not be applied over a surface which is rich in binder from accumulation of surface dressings as it will very likely find its way through the carpet and spoil the surface. Where a tack coat is considered necessary it must be applied thinly and uniformly.

**Priming Coat**

A coat of primer is necessary over dusty, porous or soft roads before applying bitumen as a bitumen will not bound to a dusty surface. (See under "Glossary of Terms") A coat of primer is not usually required with tar. Tack and priming coat are not given together. A priming coat is given with about 10 to 20 or even 30 lbs. per 100 sq. ft. of surface. The quantity of the primer and its viscosity varying with the porosity of the road material and the blindage to be used on it—10 lbs. on bituminous surfaces and 20 lbs. on water-bound. A more viscous primer should be used with porous road material and open textured roads and a thin primer with close textured roads. In all cases a primer should be allowed to soak into the road for 24 to 48 hours before the final surface dressing of bitumen is applied.

A priming coat is necessary on the following types of roads before laying a premixed carpet or surface dressing with bitumen :—

- (a) A water-bound macadam surface with good hard metal but with blindage of sandy soil or loam.
- (b) Gravel roads; gravel has hardly any mechanical interlock, priming makes it more stable.
- (c) Where the bituminous wearing coat over a newly laid water-bound macadam road has to be deferred for some reasons, a priming coat with sand blindage will preserve the surface for sometime.
- (d) Roads which have a highly capillary sub-grade; the application of a suitable primer is said to be effective in checking the rise of sub-soil water. A thin and slow curing primer applied to the sub-grade before laying the stone metal gives the best results.

The medium and close textured (or dense) carpets  $\frac{3}{4}$ " to 2" 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 vary from  $2\frac{1}{4}$ " to 5" according to the traffic and the condition of the road bed. The top course can be  $\frac{1}{2}$ " to  $1\frac{1}{4}$ " thick, but



it is generally kept about  $\frac{1}{2}$ " thick and made as a seal coat. The essential principle of two-course paving is that the base course acts as a stable but flexible structure designed to absorb the stresses imposed by traffic. By reason of its comparatively flexible nature it is capable of conforming to a somewhat varying contour in the road bed and will take up reasonable movement occurring in the sub-grade. The function of the top course is primarily to waterproof the pavement and to provide a better running surface. Two-course pavement is preferable when weaker foundations are experienced; the support value of the base largely governs the minimum thickness required.

When a single course is used, the material contains the large size aggregate of the base course, but fine aggregate is incorporated so as to produce a closer textured material to act as a wearing surface. Although this method 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 single-course than with two-course work. The incorporation of fine aggregate increases the durability of surfacing by closing its texture.

"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, nor 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.) Large size stones contribute markedly to stability.

The nature of the aggregate as well as its grading affects the quality of the carpet. Crushed rocks are preferable to gravel aggregate and to ensure good results with gravel it is necessary to use a greater proportion of fine aggregate which should not be less than 30 to 35 per cent, passing  $\frac{1}{2}$ " sieve. A dense carpet can also be made

with large size stone aggregate and 50 per cent sand (2:1) which will stand heavy cart traffic. This is sometimes called *Bituminous or Asphaltic Concrete*. Fine sand produces a slippery surface, therefore coarse sand should be used. Stability of bituminous mixtures increases as density of the mixture increases since intergrain friction is greater when particles are forced into closer proximity.

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 1" or 1½" thick.

#### Filler

Graded aggregate should be used where available with sufficient quantity of sand to form a dense surface. Where no fine sand has been used a small quantity of either fine sand or stone dust should be added to the mix at the rate of 4 to 6 per cent of the weight of aggregate passing 200 mesh if sand is used, or 4 lbs. per c.ft. if stone dust is used. Addition of filler creates resistance to flow of the binder, giving rigidity and body to the mixture. If the amount of filler is too much the mixture becomes too rigid for workability, and which might lead to early cracking. Sometimes cement is used as a filler at the rate of 1½ to 2 lbs. per c.ft. of stone. When chippings get thoroughly coated during mixing (as explained on the next page) filler is added and further mixing carried out until the filler is completely absorbed. With limestone less of fine material is required than with granite. Filler is also required with sand carpets.

#### Grading of Filler

	Percentage by weight	
	min.	max.
Total passing No. 52 B.S. sieve	100	—
Total passing No. 72 B.S. sieve	98	100
Total passing No. 200 B.S. sieve	85	100

**Requirement of Binder**—Will depend upon the type and the size of aggregate. The rougher the texture, the greater the porosity, and the lower the specific gravity,



the greater is the amount of binder required. The denser the mix, the greater is the surface area of the aggregates composing it and, therefore, a greater quantity of binder is required to coat it. Increase of sand content also needs more binder.

*Tar:* 4 lbs. per c.ft. for stone metal of sizes 1-in. and above (high viscosity tar).

5 lbs. per c. ft. for stone metal of sizes  $\frac{3}{8}$ -in. and above (tar No. 3A).

7 lbs. per c.ft. for sand (high viscosity tar).

*Bitumen :*

2½"—2" metal	2½	lbs./c.ft.	Hot Bitumens
1"—1" "	2½—3	"	30/40 penetration
"—" "	3—3½	"	
"—" "	3½—4	"	
"—" "	4	"	... Cut-backs
"—" "	6	"	... Emulsions

*Sand :*

Passing 10 mesh retained on 40 "	4—6 lbs./c.ft	Cut-backs and emulsions will be proportionately more.
Passing 40 mesh retained on 80 "	6—8 "	
Passing 80 mesh retained on 200 "	8—10 "	
Passing 200 mesh	10—15 "	

**Mixing.** Use drum mixers or concrete mixers. First mix the stone with approximately two-thirds of the total quantity of binder required for the whole batch. Add sand and finally the balance of binder. Stone aggregate and sand should not be mixed together with binder. If mixed together it will usually be found that the larger stones will not be well coated as the fine aggregate absorbs most of the binder. With aggregate containing more than about 10 per cent of material passing a  $\frac{1}{8}$  inch 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. A mixer with a vigorous action is essential for wet aggregate mixes. A double-paddle type has been found most suitable. (See also "Mixing" under "Pre-mixing" page 18/75).

**Consolidation.** Where an emulsion or cut-back has been used for a pre-mixed 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.

With hot binders, rolling of the carpet should be commenced as soon as about 50 ft. of the pre-mix has been laid, and when the bitumen is still hot. Coated chips are spread over the surface by means of rakes and templates and consolidation carried out with a 6 to 8-ton power roller. Too much rolling of coated chips is injurious. Rolling should commence at the edges and progress towards the centre. Traffic may be allowed 24 hours after completion of rolling. Fill in deficiencies with pre-mixed grit or sand while rolling. Base course and top 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.

The surface of the paving should be lightly dusted immediately after consolidation and before traffic is allowed on it with grit of a grading not exceeding  $\frac{1}{8}$ " to dust, or sand, which is spread by bass booms, at the rate of about 2 c.ft. per 100 sq. ft. The grit can either be dry or coated with 2 to 3 per cent of binder. After such dusting it is usually desirable to roll the surface lightly before opening to traffic.

#### **Thin Carpets**

Thin carpets of thickness  $\frac{3}{8}$ ",  $\frac{1}{2}$ ", or even up to  $\frac{1}{4}$ ", can be made of sand pre-mixed with bitumen or tar. The sand to be fine gritty sand all passing a 10 mesh screen (B.S. sieve); filler is added to the mixture. The carpet is laid on the road surface with a float and rolled with a light roller of about 4 tons weight and a very light sprinkling of stone dust or cement is given to the surface just before the final rolling to give it a non-tacky finish.



Open textured carpets may have a high proportion of  $\frac{3}{4}$ " or  $\frac{1}{2}$ " aggregate with about 5 to 20 per cent of fine aggregate. Medium textured may have graded aggregate from  $\frac{1}{2}$ " or  $\frac{3}{8}$ " down, and fine textured carpets  $\frac{1}{4}$ " down aggregate or sand.

If after preliminary compaction (light rolling), pre-coated chippings of size  $\frac{1}{2}$ " to  $\frac{3}{4}$ " are sprinkled over before the mixture has set, and embedded into the sand carpet by rolling light, wheeled or heavy traffic can be allowed over it.

A stone filled sand carpet in which the aggregate is composed of material smaller than  $\frac{1}{2}$ " with a substantial percentage of sand between 10 and 200 mesh (B.S. sieve), laid about 1" thick, gives a very smooth surface. Inert filler (limestone dust) or fine sand passing 200 mesh sieve should be added to the mixture. Traffic to be allowed 24 hours after laying.

Such 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. Open textured carpets are not as a rule sufficiently durable.

A cheap carpet suitable for footpaths etc., can be constructed as follows :—

The sub-grade to be of gravel, brickbats or kankar, well consolidated and cambered.  $\frac{1}{4}$ " hard stone chippings, 7 c. ft. per 100 sq. ft. for a  $\frac{3}{4}$ " thickness are uniformly spread over the surface, well watered and consolidated with a light roller to press the chippings into the existing surface. Dry coarse sand is sprinkled over the chippings at the rate of about 1 c. ft. per 100 sq. ft. Cold emulsion is applied over the surface at 40 lbs./100 sq. ft. after the surface has been moistened with water. Coarse sand is then spread at the rate of 2 c. ft./100 sq. ft. 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 it is 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 clippings of size  $\frac{1}{4}$ " to  $\frac{3}{4}$ " are sprinkled over a fine sand carpet before the mixture has set, as detailed before, it will remedy slipperiness.

*Table of Quantities of Material Required per 100 sq. ft.*

TABLE I—Open Textured Carpets

Size of aggregate (nominal)	Consolidated thickness of carpet									
	3"		2½"		2"		1½"		1"	
	c.ft.		c.ft.		c.ft.		c.ft.		c.ft.	
2"	*		*		*		*		*	
1½"	20	—	—	—	—	—	—	—	—	—
1"	—	22	18	18	—	—	—	—	—	—
¾"	—	8	—	7	15	15	—	—	—	—
½"	—	—	—	—	—	5	11	—	—	—
⅜"	—	—	—	—	—	—	—	6	—	—
⅓"	—	—	—	—	—	—	—	—	—	7½
⅔"	12	—	9	—	7	—	5½	4	—	—
Filler—lbs.	120	120	100	100	80	80	64	40	30	
Bitumen—lbs.	110	100	85	70	80	65	65	45	35	
Tar—lbs.	145	135	120	105	100	85	80	60	45	

*Note:* Carpets marked with \* can be used single-course carpets with seal coat on top, others are base course carpets for two-course work and require a top course. On thin carpets a seal coat of sand is given.

If cut-backs or emulsions are used, quantities can be computed as given earlier under "Requirement of Binder".

*Notes on Tables :*

(i) The size of aggregate given are nominal or standard size according to I.R.C. classification and as tabulated before under "Selection of Stone Metal."

(ii) The quantities given are only approximate for estimating purposes (which in fact, are very variable and depend upon many factors), and to these 5% should be added to the aggregate and 2½% to the binder for wastage



TABLE II—Dense Carpets for Single-Course Work

Size of aggregate (nominal)	Consolidated thickness of carpet														
	3"		2½"		2"		1½"		1"		¾"		½"		
	c.ft.		c.ft.		c.ft.		c.ft.		c.ft.		c.ft.		c.ft.		
2"	18	—	—	—	—	—	—	—	—	—	—	—	—	—	
1½"	9	26	13	—	—	—	—	—	—	—	—	—	—	—	
1"	—	—	8	20	11	—	—	—	—	—	—	—	—	—	
¾"	—	—	—	—	6	16	8	—	—	—	—	—	—	—	
½"	—	—	—	—	—	—	4	12	7	5½	—	—	—	—	
¼"	—	—	—	—	—	—	—	—	3	—	7½	2	6	6½	
Sand—c.ft	12	13	9½	10	7½	8	6	6	3	5½	1	2	3	6½	4½
Bitumen—lbs.	155	160	130	135	110	115	80	85	60	65	40	45	45	50	35
Tar—lbs.	190	195	150	155	120	125	90	95	70	70	45	50	52	45	30

Sand  
Carpets

(iii) The quantities for aggregate are for the stones before premixing, coated chips will increase slightly in bulk.

(iv) Stone dust has been considered for the filler in Table I. For Table II, filler may be added where necessary.

(v) If an emulsion is used the quantity will be about double that of bitumen.

(vi) The sand should be medium coarse sand as per size given in the table mentioned in (i) above.

(vii) The aggregate should not contain more than 15% by weight of material passing a 200 mesh B.S. sieve.

(viii) Excess of void filling material (sand) should not be used as that will tend to separate the stones and prevent their interlocking. 1 part of sand to 2 of stone will give a dense mixture and for lean mixtures the proportion can be increased up to 1 part of sand to 4 of stone.

(ix) For dense carpets no top coat is required except "dusting" (with stone dust or cement) after rolling.

## Seal Coat per 100 sq. ft. of surface

Topping course	Seal coat of sand
Stone $\frac{1}{4}$ " (nominal size) 5 c.ft.	Coarse sand $2\frac{1}{2}$ to 3 c.ft.
Binder 30 lbs.	Hot binder 18 lbs.
	or
If pre-coated chips are used, the quantity will be about 6 c.ft. $\frac{1}{4}$ " size.	Cut-back 20 lbs. Pre-coated sand can also be used.

For lighter applications the quantities of chips and binder may be reduced by about 20%. For seal coats with cold emulsions, see under "Grouting". Add for wastage.

**Grading of Coarse Aggregate :** (Additional to already given in the earlier tables)

Base course of pre-mixed carpets :

$1\frac{1}{2}$ " to $1\frac{1}{4}$ " {exact size } 70% } Top coat	$\frac{1}{2}$ " to $\frac{1}{4}$ " 75%
$\frac{1}{4}$ " to $\frac{3}{8}$ " {size } 30% } $\frac{1}{4}$ " to $\frac{3}{8}$ "	$\frac{1}{4}$ " to 10 mesh 20%
	passing 10 mesh 5%
$1\frac{1}{2}$ " to $\frac{3}{4}$ " " 35% } For dense	1" sieve 100%
$\frac{3}{4}$ " to $\frac{1}{2}$ " " 30% } thin	$\frac{1}{2}$ " sieve 50-70%
$\frac{1}{2}$ " to $\frac{3}{8}$ " " 30% } carpets	10 mesh 35-60%
Filler-stone dust 5%	200 mesh 7-14%
$1\frac{1}{2}$ " to $1\frac{1}{4}$ " " 30% } For dense carpets	
$1\frac{1}{4}$ " to $\frac{3}{4}$ " " 35% } $1\frac{1}{4}$ " to $2\frac{1}{2}$ " thick	
$\frac{3}{4}$ " to $\frac{1}{2}$ " " 30% }	
Filler-stone dust 5%	

**Grading of Fine Aggregate for Bituminous Pavings :**

No. 10 mesh B.S. sieve is generally considered as the dividing line between coarse and fine aggregate for road metal. The sand recommended is generally medium coarse graded sand with all passing No. 7 mesh sieve and not more than 3 per cent passing a 200 mesh sieve. For wearing-course mixtures, a size up to  $\frac{1}{8}$ " is sometimes specified with about 20 to 30 percent passing the 10 mesh sieve.

B.S. sieve size	7-mesh	25-mesh	72-mesh	200-mesh
Percentage by weight passing	100	75—98	15—58	0—3



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. (See also under "Filler.")

The sand shall consist of hard non-absorbent, but not necessarily sharp grains. Grains of very rounded shape such as some wind-blown deposits or highly polished grains should be avoided as generally good adhesion is difficult to achieve with these conditions. Impurities of any type, clay, loam or such fines as iron 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, aggregate grading is good.

#### 15. SURFACE DRESSING OR SURFACE PAINTING

Painting or spraying a road surface with a thin film of binding material (tar, bitumen etc.,) in a liquid state and subsequently applying over it sand or fine stone is called surface dressing.

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. It is therefore suitable for roads on which the traffic is mostly of rubber tyres.

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 roads. (A primer is necessary on dusty and water-bound macadam roads before the application of bitumen as bitumen will not adhere to a dusty surface). 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. Small size stone chips and coarse sand according to the thickness of the carpet, as already explained can be used. For specifications, see under "Thin Carpets." This method is commonly adopted in England for surface dressing and renewal coats, but it may not be generally practicable in India for want of necessary equipment at all the places needing repairs. 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.

### Patch Repairs

If the surface is not so bad as to warrant re-sectioning, it should be patch repaired a week or two before the surface dressing is to be started, and left open to traffic. Potholes should be cut as nearly square or rectangular as possible, and scarified to a depth of  $1\frac{1}{2}$ " to  $2\frac{1}{2}$ ", the sides being cut vertical and all loose material cleaned out. The bottom and sides of the trench should then be painted with hot tar or bitumen and filled with pre-coated metal, the size of the metal depending upon the depth of the potholes. The metal should be rammed in layers of 1" at a time (the hand rammer being dipped in water from time to time so that the coated metal may not stick to it.) The finished filling should be "proud" on the surface by about  $\frac{1}{4}$ " 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 wheels. Gauge of the patching metal should never exceed  $1\frac{1}{2}$ ".

Where the hollows are slight (up to 1" 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.

### Quantities of Metal Required for Patch work :—

For estimating purposes the following quantities of metal may be taken for patch repairs annually, per mile of road length, 12 ft. width of metal :—

Light traffic	..	.. 700 to 900 c. ft.
Medium traffic	..	.. 1000 to 1500 c. ft.
Heavy traffic	..	.. 2000 to 4000 c. ft.

For portions on gradients, add about 25 per cent extra.

### Preparation of the Road Surface

If a newly-metalled road has been well shaped and rolled smooth, it is advisable to allow traffic on it for a year before surfacing. During that period the crust will have become firm and consolidated and the 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.

It is very essential to examine the water-bound macadam carefully before deciding to surface dress it. The road must be bone dry before painting is commenced and this dry condition must extend right down to the earth sub-grade (or at least 2 to 3 inches in case of emergency). It is most important that the surface of the road shall be thoroughly cleaned before the application of bitumen or tar. It shall be swept clean and free from dust, dirt or other deleterious matter by hand brushing with wire brushes, bass brooms and finally by fanning with gunny bags to remove all loose dust. Stone metal of the old untreated road surface should be exposed to a depth of  $\frac{1}{4}$ " to  $\frac{1}{2}$ " 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. If the existing surface shows any depressions, potholes, ruts or irregularities exceeding  $\frac{1}{2}$ " or has a steeper camber than 1 in 60, it must be re-conditioned or patched a week or two in advance before commencement of the surface dressing.

#### **Rate of Spread of Binder**

The rate of spread of a binder is probably the most important factor affecting the quality of the surface dressing and special 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. For best adhesion, the binder film must rise up the sides of the chippings so that each chipping is held to its neighbours as well as to the road surface, the finished surface should show the stone chips just imbedded but not covered with the binder. The quantity of the binder depends on the size and shape of the chippings used and the condition of the road surface. Rounded gravel requires more binder than angular or cubical



stones. The thickness of the film shall be increased with increase in size of the aggregate, and when gravel is employed it shall be thicker than with granite (crushed or broken stone). Flaky or flat stones will require less binder than cubical stones. A relatively small difference, in the rate of spread, of the order of 1 sq. yd. per gallon, can have a marked effect on the results.

The more intense the traffic, as the compaction of the surface is increased, less binder is required (within the recommended limits). 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. The binder should not be too much or too little but just right all over the road.

The following are the recommended rates of spread of tar for most traffic conditions, as given by the British Road Tar Association, London :—

For Impervious Road Surfaces or Subsequent Dressings :—

Nominal size of chippings	Rates of spread of tar	
	Angular chippings (crushed rock)	Rounded chippings (gravel)
	sq. yds./gall.	sq. yds./gall.
1½—2½	4—5½	3½—4½
2—3	5—6	4—5
3—4	6—7	5—6
4—5	7—8	7—8

The thinner rates are more applicable to roads rich in binder, the thicker rates to roads appearing dry and deficient in binder. Open-textured pervious surfaces require more binder than is indicated by these rates. A thicker film of binder and of lower viscosity than that suggested above (or normally applied on previously treated roads) is desirable on a tar-macadam road and also on a water-bound macadam road that are being dressed for the first time. 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.

For 1st dressing on water-bound macadam or  
tarmacadam :

Rate of spread of tar sq. yds/gall	
Water-bound macadam (tightly bonded)	Tarmacadam (open textured)
3½—4½	4—4½

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, therefore ¾-in. size is recommended.

### Use of Emulsions

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 "fatting up" with emulsions, but a deficiency may bring about premature wear of the road surface. This is a useful property of emulsions and is important in the resurfacing of bitumen and tar surfaces where too thick an application of bitumen might result in bleeding.

If the road is very dry and dust is coming out from between the stones, it will be found advantageous to damp the road surface with water before pouring emulsion. The chippings shall be applied before the emulsion has "broken". (Where sand is used instead of chippings it should not be applied to the surface until after the emulsion has "broken".) Emulsions do not spread a thick layer of bitumen and cannot take bigger chippings than ½-in. for dressing on a previously treated surface.



### 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 boilers otherwise the tar will froth up and overflow. A full tar drum must always remain suspended over the tar boiler so that the contents will never be reduced to less than  $\frac{1}{4}$ th of its 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.



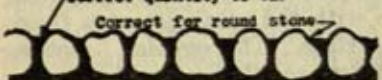
Insufficient tar (gives scabbing)



Excess tar (gives bleeding)



Correct quantity of tar



Correct for round stone

cannot be thus heated as it has to be raised to a much higher temperature which may not be possible.

The application of a binder is more efficiently and quickly done with pressure sprayers 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 paint 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 tar heater shall have a capacity in keeping with the daily output required and shall be so designed and operated that the temperature of the tar discharged to the road surface shall be within the range prescribed. A suitable thermometer (reading up to 600 deg. F.) shall be provided to enable the temperature to be determined. For small jobs tar can be heated in drums with a portion of the side cut out. Bitumen

the tar. The rate of application is controlled by maintaining a constant reading in the pressure gauge, where provided. Pumping may only cease when there is nothing but hot air coming out of the nozzle. Spraying shall 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 tar on the road caused by stopping and strating the sprayer, or by leakage, should not be allowed.

The operation is carried out by one man spraying the tar and two men equipped with long handled soft brushes. The tar 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. It is essential to draw the binder in one direction only as far as possible and to avoid painting it backwards and forwards.

The quantity of tar 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, however, hand pouring pots are used and two coats are to be done simultaneously, the lines 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 day'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 heat-



ing and applying tar should be provided with boots, putties and goggles.

(These specifications apply equally to hot Bitumens)

For the application of cold emulsions special pouring cans fitted with baffle mouthpieces are used. These cans should have a capacity of not less than cialns.

### **Shape, Size and Rate of Spread of Chippings**

The aggregate should be of non-flaky, hard, tough, clean, crushed or broken rock. Good results may be obtained with gravel if the methods of laying described are strictly adhered to, particularly those relating to the rate of spread of the binder and the interval before the traffic is allowed over the new work. In general the harder stone 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. Stones providing elongated particles generally suffer some weakness in the short axis and do not provide the interlocking necessary for maximum stability under the forces exerted by traffic.

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 (tar or bitumn) 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 than shoulder-to-shoulder cover. The usual practice is to allow roughly one cubic foot of chips to each gallon

(10 lbs.) of the binder and in the case of hot binder, add 10 to 20 per cent more. (See tables at pages 18/102, 103.)

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  $\frac{3}{4}$  c.ft. chips and  $\frac{1}{8}$  c.ft. of sand to each gallon (10 lbs.). 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 weather 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,  $\frac{3}{8}$ " or  $\frac{1}{2}$ " 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, larger sizes  $\frac{3}{4}$ ", are suitable. Where the problem is to deal with iron-tyred bullock-carts, chippings of size 1" to  $1\frac{1}{4}$ " may be used. For a surface dressing on tightly bonded (smooth surface) waterbound macadam road, the size of the chippings should be  $\frac{1}{2}$ " to  $\frac{3}{4}$ " according to the surface.  $\frac{3}{8}$ " or  $\frac{1}{2}$ " crushed rock is considered most suitable for chippings for normal roads.

The following rate of spread of chippings is recommended by the British Road Tar Association, London :—

Nominal size of chippings	Rate of application (sq.yd./ton)	The aggregate to weigh not less than 78 lbs. per c. ft.
$\frac{1}{4}$ "	70—80	
$\frac{3}{8}$ "	90—100	
$\frac{1}{2}$ "	100—120	
$\frac{3}{4}$ "	140—170	

**Spreading of Chippings.** It is desirable that clean, dry chippings be used, properly screened. The 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 aggregate shall be uniformly distributed and precautions shall be taken so that the whole of the sprayed surface is covered uniformly without any accumulation of surplus chippings at any point. The hand distributed 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.

**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 roller should be passed over the whole surface and given a thorough rolling. The weight of the roller must be governed to some extent by the character (hardness and size) of the chips and 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 tons for  $\frac{1}{4}$ " size of chips and 8 to 10 tons for bigger size. 6 to 8 trips of the roller should suffice for most of the works. 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-ton) 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 or rollers. The rubber tyre 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  $\frac{1}{4}$ -in. 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 surface. When an interval is to elapse between each application, the method of construction is exactly the same as in the single-coat work.

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 following show the quantities of materials usually recommended for the different types of works.

Some engineers recommend the use of medium sand instead of stone chips for the first coat. This is especially beneficial where the water-bound macadam is loosely compacted; 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 road tar and the stone chips. Cushioning formed by the sand mat increases the load bearing capacity of the stone chippings of the second or wearing course. Normally 40 to 45 lbs. of road tar and 3 c. ft. of sand are required per 100 sq. ft. of surface; the quantity of tar may be reduced for dry climates. 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. 30 to 35 lbs. of tar blinded with 5 c. ft. of stone chippings will be sufficient.

**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 starts 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 consolidation of the 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.

Where bituminous asphalt has 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 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.

In order to promote coating of wet 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.)

For the binder, it is not necessary to add any special reagent to tar and the tar specified is usually of the same grade for use in wet aggregate mixes, but for improved results with bitumen, 2 per cent by weight of the bitumen of Turkey red oil (fatty acid), and many other compounds have been found effective. Burmah-Shell recommend "Shelmac D", a hot application cut-back bitumen, which is used in the same manner as other hot asphalts. Bitumen emulsions can also be used with wet aggregate or on wet surfaces. Certain chemicals are also manufactured by the Imperial Chemical Industries which help the adhesion of bituminous mixtures to wet the aggregate. (See I.R.C. Journal vol. XVIII—1, page 83.)

**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 5 to 8 lbs. per 100 sq. ft. 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 about  $\frac{1}{4}$  lb. of lime to 1 c.ft. of

stone aggregate, and from 1 to 1½ lbs. of lime to 1 c. ft. of sand. Aggregate and sand should be mixed independently with lime. Mixing with lime should be done first and when thoroughly mixed, then only binder should be added. 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 premixing. Mixing should preferably be done in a mechanical mixer and the binder sprayed with a pressure sprayer. If the previous road surface dries out after cleaning, no lime need be sprinkled over it.

**Surface Dressing of Roads with Tar**  
Quantities of Materials Required per 100 sq. ft. of Road Surface

Traffic density	Quantity of tar	Tar No.	Quantity of chips	Size of chippings	Remarks
	lbs.		c.ft.	ins.	
1st Coat					Table based in general on Shalimar Tar specifications.
Light	35—40	3	4.0—4.5	$\frac{3}{8}$	
Heavy	35—40	3A	4.5—5.0	$\frac{1}{2}$	
Light	45—50	3A	4.5—5.0	$\frac{3}{4}$ to $\frac{3}{8}$	
Heavy	45—50	3A	5.0—5.5	1 to $\frac{1}{2}$	
2nd coat or Renewal coat					Add 2½ per cent wastage for tar and 5 per cent wastage for chips.
Light	20—25	3	3.0	$\frac{1}{4}$	
Heavy	20—25	3A	3.0—3.5	$\frac{3}{4}$	
Stabilizing coat with Sand					
	25—35	3	2.0—2.5	Use medium coarse sand.	

Tar is heated to the following temperatures:—

No. 3—2000 F. to 2200 F.      No. 3A—2200 F. to 2400 F.

*N.B.*—Traffic conditions: Up to 500 laden bullock carts per day is taken as light traffic, 500 to 800 laden bullock carts per day taken as heavy traffic.



**Surface Dressing of Roads with Bitumens**  
**Quantities of Materials Required per 100 sq. ft. of**  
**Road Surface**

Quantity of		Size of chippings	Remarks
Bitumen	Chips		
lbs.	c. ft.	in.	
<b>Two-coat work—Heavy type</b>			Table based on Burmah-Shell specifications (Revised)
25	10	1"	For cart traffic
45—50	5	$\frac{1}{2}$ " to $\frac{3}{8}$ "	1st coat } Simultaneous 2nd coat } work on open textured water-bound macadam.
<b>Two-coat work—Medium type</b>			For motor traffic
35—40	5—6	$\frac{3}{4}$ " to $\frac{1}{2}$ "	1st coat
25	4	$\frac{3}{8}$ " to $\frac{1}{4}$ "	2nd coat
<i>Alternative</i>			
25	5—6	$\frac{1}{2}$ "	1st coat } Simultaneous work
45	3 $\frac{1}{2}$	$\frac{1}{4}$ "* to fines	2nd coat }
<b>Single-coat work</b>			On dense and compact surfaces. Suitable for heavy traffic
25—35	5	$\frac{1}{2}$ " to $\frac{3}{8}$ "	
<b>Renewal coat</b>			On old surfaces
25	3 $\frac{1}{2}$ —4	$\frac{1}{4}$ "	

(i) Where an emulsion is used, the quantity will be 35 lbs. for each coat (or may be only 30 lbs. for the 2nd coat or renewal coat). Where hot cut-backs are used the quantities and process is the same as with hot bitumen.

(ii) Lesser quantities and smaller sizes are for dense and compact surfaces and for lighter traffic.

(iii) Add 2 $\frac{1}{2}$  per cent wastage for binder and 5 per cent wastage for chips.

**Estimating Quantities for Labour and Sundries**

**Heating of Bitumen and Tar (Requirement of fire-wood):—**

(i) 3 cwts. of fire wood required for heating 1 ton up to about 150 deg. F.

(ii) 5 cwts. of firewood required for heating 1 ton up to about 200 deg. to 250 deg. F.

(iii) 8 cwts. of firewood required for heating 1 ton up to about 350 deg. F.

#### Brushing Road Surfaces per 1000 sq. ft. :—

- |  |              |
|--|--------------|
| (i) Water-bound macadam or other kankar, laterite or open texture surfaces | 1½ labourers |
| (ii) Old bituminous or tarred surfaces                                     | ½ "          |
| (iii) Preparation of surface for 2nd coat painting                         | ½ "          |

#### Heating and Spraying Road Surfaces with Tar Boilers :—

- (i) 1 to 1½ labourers for spraying 1000 sq. ft. of surface.
- (ii) 1 to 1½ labourers (3 to 5 c.ft. of chips per 100 sq. ft. of surface sprayed) per 1000 sq. ft. of road surface for spreading stone chips.

#### Sundries required for road works :

Baskets, buckets, pick axes, forks, ropes, thermometers, wire brushes, soft brushes, gunny bags, empty tins for water, pouring cans, road template, Caution boards, barriers, watchman's lantern, soap, etc.

## 16. CEMENT CONCRETE ROADS

### Foundations under Concrete Pavements

It is very essential to have a good solid foundation of well consolidated and non-absorptive materials under a concrete road. This being a rigid type of construction, a concrete slab should not be placed over a yielding flexible base (foundation) as it will develop cracks. Under-surface must be properly levelled, hollow places filled up and consolidated with hard-core, cambered and cross-falls or longitudinal slopes given. The sub-grade must have equal bearing power all over. Lack of uniformity of support will result in much higher stresses in the concrete slab and it is therefore necessary to prepare the sub-grade in such a way that the slabs are supported as uniformly as possible. If a slab is uniformly supported by the sub-grade only a small variation in stress will occur for large differences in soil type and the majority of stresses will be well below the ultimate strength of the concrete. In confined sites, such as bridge abutments, behind re-



taining walls or in trenches, compaction of the soil is most efficiently carried out by heavy hand rammers.

A kankar sub-grade 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. If the foundation is of gravel or sand it should be well rolled and levelled and either sprayed with bituminous emulsion and blinded before the concrete is laid or covered with waterproof paper to prevent loss of liquid into it.

#### **Provision of a Base**

The load-carrying capacity of a concrete road structure lies mainly in the structural rigidity of the slab (and the uniformity of sub-grade support) and it is considered that undercourses or bases should be used only when they will correct or counteract an unsatisfactory or unstable soil condition. Inclusion of a base for the sole purpose of increasing sub-grade support or the structural strength of the road is generally uneconomical unless suitable base material can be obtained cheaply. It is considered that 1 in. of concrete is equivalent to between 3 and 6 ins. of base depending on the particular conditions. The provision of a base will result in only a small increase in the strength of the road structure. A base should only be used to form a working surface on clays, silts and sandy clay soils and other weak soils, 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. A base need only be thick enough to provide a smooth, hard working face over the formation so that the sub-grade is not deformed or otherwise damaged during the construction work. A sub-grade of hard-rock need also be covered with a layer of broken stone or gravel. When a base is considered to be necessary, it is seldom worth laying a thickness of less than 3 ins., and except on poor soils a base thickness of 3 ins. is usually adequate.

It has been reported that slabs laid on a rigid base have shown a somewhat unexpected tendency to crack; it is possible therefore that bases of stabilized soil and lean-mix concrete may, in certain circumstances, be unsuitable for concrete roads. Unlike granular materials they cannot accommodate themselves to the warped shape of the slab which may then crack as a result of non-uniform support.

The base should be carried at least 12 ins. beyond the edge of the concrete slab, especially if kerbs are not used or are not set on the slabs.

The most suitable base for clays is stabilized soil or a lean concrete mix of say, 1:4:8 or even 1:5:10, 4 to 6 ins. thick. Where the clay is very weak and plastic, a sub-base of 4 ins. of granular material is sometimes laid. Or alternatively, a base course may consist of 4 to 6 ins. of well burnt clinker, gravel, broken stone or brick ballast, well consolidated. A soling course (sub-base) of bricks or stones will be necessary in addition on weak sub-grades (such as black cotton soils), in which case the thickness of base course may be reduced to 3 to 4 ins. only. "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 sub-grades of gravel, sand, or gravel-sand-clay which can be thoroughly compacted.

It is essential to thoroughly wet the base before laying the cement concrete slab. The sub-grade or the base should be moist but not muddy at the time of placing concrete, it may be saturated with water the previous night or not less than 6 hours previous to the placing of concrete: 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.

Concrete contracts while setting, therefore, a newly laid slab should have free movement while drying. There will also be subsequent expansion and contraction due to temperature changes and the range of temperature and its rate of change will be greater in a concrete slab than in the foundation (variation of temperature between top and bottom of the slab is proportional



to the depth of the slab) and, as a consequence, the slab will tend to slide over the foundation. This horizontal movement of the slab is restrained by friction between the slab and the sub-grade or base. The friction may be reduced by laying the slabs on waterproof paper. The use of waterproof paper is particularly desirable for slabs laid in hot weather when considerable contraction may take place before the full strength of the concrete has developed. A layer of  $\frac{1}{2}$  to 1 in. 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. (See also next para.) Another advantage of a waterproofing layer between the slab and the sub-grade is, loss of cement paste to a porous base or sub-grade is prevented.

Experiments carried out in U.P. with 3 ins. and 4 ins. thick slabs laid on existing well consolidated water-bound macadam surfaces where the concrete was so laid as to bond thoroughly with the rough and uneven surface of the base, have proved very satisfactory. The existing surface is cleaned with wire brushes so as to remove all loose material and the protruding metal provides a good key.

#### 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 continuously in strips between longitudinal joints. Ends of slabs should be painted with bitumen before the intermediate bays are filled in. Concrete pavements are usually laid in series of bays with their dimensions varying in lengths from 30 ft. to 35 ft., with 50 ft. maximum, and widths from 8 ft. to 15 ft. On steep grades the slabs may be laid in bays of about 15 ft. lengths. Size of the slabs is fixed according to the joints—transverse and longitudinal. In alternate bay method fewer expansion joints can be provided.

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 the road is laid in two courses, the top course should be laid before the bottom course has set so that a good bond between the two layers is obtained. Where good hard aggregate 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 1 to 2 inches 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 ½" to ¾" higher than the profile to permit of its being rammed into position with the tamper.

Slabs with thickened edges have no advantage over slabs of one uniform thickness but on the other hand are rather unreliable since the sub-grades 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 6 ins. thick. Where this is not possible the corners should be strengthened by increasing either the thickness or providing corner reinforcements. The stress at a corner of 50-deg. is about double the stress at a right-angled corner and proportionately so with other angles. Widths of the slabs should not also be less than 4 ft. A plain concrete slab in which the dimension in any direction exceeds 15 ft. is liable to crack. Where the slabs are reinforced longitudinally and transversely with equal amount, widths can be increased up to 24 ft.

All the arrises of cement concrete slabs should be chamfered at 45 deg. for 1½ ins. width and rounded. All manholes and gullies should be surrounded by wooden boxes during concreting and filled round later. After completion of



the concrete edges of the slab should be protected by strips of water-bound macadam at least 12 ins. wide and 4 ins. thick. Stone, brick ballast, kankar may be used.

**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  $2\frac{1}{2}$ " max. size. But the size most commonly recommended is from  $1\frac{1}{2}$ " down to  $\frac{3}{8}$ " for the base course. It is considered that the coarse aggregate should not be less than  $1\frac{1}{2}$ " otherwise it would not take the load. For the top course the size should not generally be greater than half the thickness of the concrete (top course);  $\frac{3}{4}$ " is the common size with  $1\frac{1}{2}$ " max., graded down to  $\frac{3}{16}$ ". Requirement of surface texture may also sometimes effect the choice of size of coarse aggregate. Grading of coarse aggregate successfully used: (a)  $1\frac{1}{2}$ " to 1"—2 parts,  $\frac{3}{4}$ " to  $\frac{3}{8}$ "—1 part; (b) mixed 40%— $1\frac{1}{2}$ " and 60%— $\frac{3}{4}$ " nominal size; (c) 50%— $1\frac{1}{2}$ " and 50%— $\frac{3}{4}$ " nominal size, in the proportion 1 : 2 : 3  $\frac{1}{2}$ . The aggregate should be as much cubical as possible in shape.

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  $\frac{1}{4}$ " sieve and not more than 20 per cent shall pass through a 50 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.

#### **Base and Slab Thickness for Various Sub-grades**

See under—"18 Design of Pavements."

Cement concrete slabs of 3" thickness laid over and well bonded with old water-bound macadam have stood traffic intensity of 1400 to 1500 tons in U. P. Only some of the slabs developed cracks near joints and edges. Theoretically, 3" thickness is too small for this intensity of load but the endurance of these slabs may be due to the effective bonding of the slab to the existing road which virtually increases the thickness of the slab, and

also due to the higher modulus of sub-grade reaction of the thick and consolidated old road crust. As there is a tendency for the 3" thick slabs to crack on weak sub-grades, a 3" to 4" layer of well consolidated stone or over-burnt brick ballast should be able to take a fairly heavy traffic, and corners may be reinforced in addition. 4" thick slabs laid on a sub-grade of substantial macadam crust of 6" of stone or ballast have been found adequate to stand an axle load of 18000 lbs. (axle load recommended for pavement thickness design for Indian roads and which is also adopted by many American States) and heavy cart traffic. These slabs are without reinforcement.

There are many variables such as, allowable flexural stress in concrete, co-efficient of sub-grade reaction or the sub-grade support, wheel load, temperature variations, co-efficient of sub-grade resistance, i.e., the friction between the slab and the sub-grade, distance between joints, that must be considered in designing pavement thickness and many of these variables cannot be evaluated precisely, therefore, the design calculations give more or less approximate values, and reliance should be placed on previous experience.

Stresses due to wheel loads are higher at the edges and corners of the road slab 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 9 to 12-in. 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 6 ins. thick or where the slabs are on a poor foundation, it is advisable to strengthen their corners by means of "hairpin" reinforcement as detailed under "Reinforcement" in the following pages.

### Foot-paths

A finished thickness of 2" cement concrete 1 : 2 : 4 slab (cast-in-situ or pre-cast) laid over 2" of lean concrete say 1 : 6 : 12, over well consolidated sub-grade and sand filling, will meet most of the requirements of foot-paths. The top slab may be laid while the bottom slab is still green. On heavily trafficked paths and cycling tracks, the thickness of the top slab may be increased to 2½" to 3". Kerbs or rounded "edgings" may be provided at the ends (see under "Kerbs"). A thickness of 3" for the common foot-paths and 4" for heavily trafficked paths and cycling tracks is usually recommended in England and America; 2" is for the most lightly trafficked paths.

The pavements can be constructed in bays, or in panels, or continuous with dummy joints at 6 to 8 ft. intervals. For a 2" thickness the bay or panel should have no side longer than 6 ft., while 12 ft. should be the maximum length for the thickest slab. Sometimes pre-cast square slabs of sides about 14" (or 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 spaces left in between filled with half-square pieces. About 1/16" gaps are left as joints which may 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 when it is still green or broomed as explained under "Concrete Road Slabs".

### Reinforcement

Reinforcement is not always necessary where the slab is laid on firm and well consolidated foundations. Reinforcement helps to carry over any weak places in the sub-grade and is usually provided under heavy loads. Seldom, if ever, is reinforcement counted on to resist flexural stresses produced by loads or warping. Reinforce-

ment prevents the widening of cracks produced by flexure and holds the fractured faces in intimate contact. Hair cracks often occur at frequent intervals in reinforced concrete slabs, but such cracks show no tendency to open or deteriorate and some penetrate only just below the surface.

Experiments carried out at the Road Research Laboratory, Harmondsworth, have indicated that the maximum tensile stresses occur at the top of a slab at the corners and at 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. Reinforcements are placed  $1\frac{1}{2}$ " to 2" below the top of the slab. Another point in favour of the top reinforcement is that the sun shone 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. If the sub-soil is weak, the reinforcement must be laid in the bottom and about 2" above the bottom of the concrete; but if the sub-soil is unreliable as regards its settlement, and temperature stresses are expected, double layer of reinforcement should be provided. Reinforcement should be equal in both the layers. 6-ins. thick or less can 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 14 lbs./sq. yd. or more is required or for positions stated above. It is not economical to use sufficient steel to increase appreciably the structural strength of the slabs.

Amount of reinforcement is 5 to 14 lbs./sq. yd., per each layer. For moderate traffic 7 lbs./sq. yd. is sufficient and this is the commonest. About 10 lbs./sq. yd. of steel, with the greater proportion placed in the longitudinal direction, should be the minimum used in the



heavily trafficked roads.

Reinforcement is sometimes worked out from the following formula :

$$A_s = LWc/2f$$

where :

$A_s$  = area of steel in sq. ins. per ft. width of slab measured at right angles to the bars;  $L$  = length of slab between joints or edges in ft.;  $W$  = wt. of slab per sq. ft. in lbs.;  $c$  = co-efficient of sub-grade resistance. Assumption for it usually ranges from 1 to 2, with 1.5 as the average;  $f$  = working stress in the reinforcing steel in lbs. per sq. in. This is usually taken half of the yield point stress. May be taken 20,000 for steel bars and 28,000 where cold drawn steel wire fabric is used.

The formula can be applied to both longitudinal and transverse steel requirements, which will be in the same proportions as the longitudinal and transverse dimensions of the slab.

"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 at an angle of 30 deg. Each leg is 4 to 5 ft. long, and  $\frac{3}{8}$ " dia. and hooked at ends. Another method is to bend two 8 to 10 ft. long bars at 60 deg. Sometimes two straight bars are placed at the corner at right angle and a third bar at 45 deg. If the main reinforcement is not placed at the top then the corner reinforcement should be 2 ins. from the top of the slab otherwise at middle depth.

In slabs of normal width cracks usually occur transversely. As such, it is desirable to provide the greater portion of the steel in the form of longitudinal bars (in the direction of the greatest dimension of the slab) and a relatively small proportion as transverse bars. The longitudinal bars should therefore be heavier and more closely spaced than the transverse bars.  $\frac{3}{8}$ " dia. longitudinal bars at 12" to 18" centres and  $\frac{1}{4}$ " transverse bars at 20" centres may be provided. In the case of wide slabs (say 15 ft.) and where the transverse joints are about the width of the road, reinforcement should be given 50 per cent each side.

The reinforcement should always be delivered to the site in the form of flat mats and not in rolls so that it may not tend to take a curved shape when laid. Joints must be an overlap of one complete mesh of the fabric, or 40 times the diameter of the bars if a proprietary mesh is not used;  $\frac{1}{4}$ " bars, for instance, require an over-lap of 10 ins. Reinforcement should stop short 2" of all edges and joints in the slab. Transverse bars should not be lapped.

### Forms

The importance of careful and accurate form setting cannot be over-emphasized. The forms (steel or timber), should be supported firmly by stout stakes about 4 ft. distance throughout their entire bearing area on a uniformly firm foundation. Line pegs should be fixed at 100 ft. intervals on straights, at all tangent points, and at about 20 ft. intervals on curves. Steel forms shall be made of metal not less than  $\frac{3}{16}$ " thick and with base width of at least 8". Shall have square ends connected with a rigid lock joint. The forms shall be true to a straight line with tolerance of  $\frac{1}{8}$ " in 10 ft. for the top surface. Wood forms should be used for curves of less than 150 ft. radius; for smaller curves side forms of brick in clay plastered can be used. Wood forms must be securely supported and braced. The thickness of wooden forms should be at least one-third the depth with a minimum of 2". Level pegs should be fixed with a dumpy level at 100-ft. intervals and at all changes of gradient. All forms should be treated with oil on the inside face to prevent concrete from adhering to them. Forms shall not be removed until 24 hours after the concrete has been laid.

### 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) Depression 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.

Whenever work stops for a long period allowing the concrete to harden, a vertical butt joint should be made, but when concreting is stopped only for 10 to 15 minutes, the new and old concrete should be thoroughly sliced together to ensure that no cleavage is formed.

Joints are made in concrete pavements to keep the stresses caused by changes of temperature within safe limits and prevent the formation of irregular cracks. It has been explained in the Section on "Reinforced Concrete" under "Expansion Joints" that the total expansion that would occur in a 100 ft. length of concrete structure with a rise of temperature of 50 deg. F. will be about  $\frac{1}{8}$  in. and if this expansion is not allowed to occur, the concrete will develop a stress of only about 975 lbs. per sq. in. against a compressive stress of 2250 lbs. per sq. in. (for 1 : 2 : 4 concrete after 28 days, and which will be about 3500 to 4000 after one year). Transverse expansion joints are provided at intervals in a concrete road based on the above expansion limits. The joints are filled in with a compressible material and sealed with a sealing compound to prevent the entrance of grit or water. It is very important to keep the joints clean of any road grit or other foreign inert matter which can interfere with the free expansion of the concrete. Entrance of water through the joints will undermine the stability of the road structure. Joints are generally made at the time the pavement is constructed and they are either plain butt joints with full depth break in continuity of the slab or "Dummy" groove joints. Joints are formed by placing within the concrete strips of metal or wood or impregnated fibre of a thickness of the joint required embedded close under the pavement surface. A groove of the proper depth is formed in plastic concrete after surface finish. A grooving tool for dummy joints can be made of a rolled steel T-section with its stem facing down and handles provided at both

ends. Fewer contraction and dummy joints are required if the slabs are reinforced.

**Transverse Joints.** Are at right angles to the road length. Transverse joints fall in two general classes—*Expansion* joints and *Contraction* joints. No definite rules for the spacing of these joints can be given as the spacings depend upon many variable factors. Contraction joints are generally provided at intervals of 30 to 35 ft., with 50 ft. maximum and expansion joints at 100 to 120 ft. intervals or more, according to the temperature ranges and the thickness of the slab, thicker slabs and reinforced slabs having joints at greater intervals. A contraction joint consists of two neighbouring slabs butted together with a groove at the top similar to that of a dummy-groove joint. *Construction* joints made due to stopping of work at the end of a day's work are made into contraction joints. Contraction joints may be provided with smooth dowel bars if the spacing of such joints exceeds 20 ft. Dummy contraction joints are introduced at about 15 ft. intervals in between the contraction joints in unreinforced slabs.

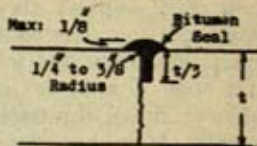
In expansion joints there is a complete separation between the abutting slabs and a space of about  $\frac{3}{8}$ " to  $\frac{1}{4}$ " is left in between the two slabs which is filled to within  $\frac{3}{4}$ " to 1" of surface with a joint filling compressible material (described hereafter) and the top filled in with a sealing compound. Expansion joints should be provided at intersections of pavements with structures or other pavements. Expansion joints at long distances is a definite advantage and roads have been laid in America with no expansion joints but only dummy-groove construction joints. If dummy joints are provided at short intervals, expansion joints may be made at longer intervals with joint widths of from  $\frac{3}{4}$ " to 1". Roads constructed in winter months should have expansion joints at closer intervals than those constructed in summer months. Joints should not be too wide as traffic damages slab edges with wide joints more quickly than those with narrow joints.

Transverse joints should preferably be made at right angles to the kerbs and continued straight across the carriageway. 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 9 to 12 inches. Some engineers are of opinion that staggered joints might form "sympathetic" cracks across the sound slab continuing the joint, but it is not essential.

**"Dummy" groove joints** are made similar to other joints except that the break in continuity of the slab is only about  $\frac{1}{4}$  to  $\frac{1}{3}$  of the pavement thickness and the lower solid portion, which forms plane of weakness, cracking during shrinkage of the concrete. The width of the groove is between  $\frac{1}{4}$ " to  $\frac{3}{8}$ " according to the expected temperature changes and the spacings of the joints. 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.



DUMMY JOINT

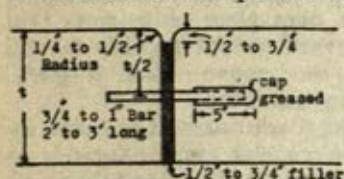
**Tongued and Grooved joints** are sometimes made instead of butt joints to prevent one slab rising relatively to the adjacent slab and to enable transfer of load to take place, but the efficiency of these joints is doubtful. Tie bars are also fixed in these joints as in butt joints.

**Longitudinal Joints.** Longitudinal joints are made when the width of the pavement is more than 8 to 12 ft. (with 15 ft. maximum). The width of a road slab should, preferably, be not more than 10 ft. This width makes tamping easier and prevents the formation of longitudinal cracks; it also reduces shrinkage and warping stresses. The joints may be of the dummy-groove type or butt joints (preferably latter). All longitudinal joints should be keyed with deformed tie bars, as explained under "Load Transfer Device", to hold the two slabs together.

It is advisable to use a mix slightly deficient in coarse aggregate near the joints to avoid risk of honeycombing at these positions. The arrises of the joints are carefully rounded to a radius of about  $\frac{3}{8}$ " with a suitable arrising tool about half an hour after the final tamping of the concrete when the concrete has become rather stiff but before it has hardened enough to be unworkable. Final edging

normally succeeds final belting or brooming. As soon as the joint is finished and before it is sealed, the joint should be tested with a 5 ft. straight-edge placed so that it extends on both sides of the joint. Arrises should be wire brushed before sealing to remove surface laitance and promote good adhesion of the sealing compound.

**Load Transfer Devices.** Are used in joints of concrete road slabs laid on sub-grades of doubtful bearing value such as peat, clay, made ground, or in roads of very heavy traffic. Dowels are also required across all contraction joints when the joint spacing exceeds 20 ft. Round mild steel dowels or tie bars are used which are placed half-way down the depth of the slab. They project horizontally from one panel to the next. With transverse joints smooth bars are used of which one-half is embedded in concrete of one panel and the other half greased



or pointed with bitumen to allow for free movement. The sliding or free end of the bar is fitted with a light metal cap about 5" long and slightly larger in diameter than the bar to which

it is secured. The closed end of the cap should be filled to a depth of about 1 in. with soft compressible material, such as cotton waste, to provide space for the bar to move when expansion takes place. In the case of longitudinal joints either deformed tie bars are used or they are hooked at the ends to have good bond in the concrete of both the slabs (and are not greased or painted). These bars are  $\frac{3}{4}$ " to 1" ( $\frac{5}{8}$ " average) in diameter and 2 to 3 ft. long for transverse joints where they are placed 1 to 2 ft. apart, and 3 to 5 ft. long for longitudinal joints, spaced 4 to 5 ft. apart. The dowels nearest the edges of the slabs should be placed closer together say, 9 to 12 inches from the ends.

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 may be used for the purpose. Opinions differ on the length of dowels, but it has been established that the portion of the dowel after the first 5 inches embedded from the joint



face plays a negligible part.

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.

### Joint Filling Materials for Concrete Roads

A joint filling material must be compressible which will not extrude from the joint as the concrete expands. Expansion joints are filled in (within  $\frac{3}{4}$ " to 1" of surface) with a strip of soft knot-free wood, impregnated fibre board, cork or cellular rubber. The most commonly used fillers are mixtures of asphalt (20/30 penetration) and saw dust or chopped hemp or coir. The joints are caulked up to a depth of about  $\frac{3}{4}$ " to 1" below the top of the pavement with hemp, chopped fibre board or saw dust and the sealing compound consisting of a plastic material, usually asphalt (heated) 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. A typical joint filler mixture used on some roads in India : Sand 60%, asphalt 30%, saw-dust 7%, cement 3%. 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  $\frac{1}{8}$ " on the surface.

In order to promote adhesion it is advisable to prime the joints with a bituminous paint or cut-back bitumen, prior to sealing. These particular primers are not suitable if a resinous sealing compound is used. Dry faces painted with kerosene oil also facilitate the adhesion of molten bitumen or any other bituminous emulsion used to prime the joints. Joints should be well cleaned before sealing work is done.

An extruding filler or excess material over a joint which cause jar or bump to passing vehicles can be trimmed off with a sharp edged tool which may be heated to facilitate its cutting action. 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. A specification for joint sealing materials is given in B.S. 2499 : 1954.

### **Screeding and Tamping**

The surface of the concrete slab is brought to the specified contour by means of a heavy screed or tamper fitted with handles, weighing 7 to 9 lbs. per running foot and not less than 3" wide and 9" high shaped to the cross section of the slab. The tamper or screed should rest about 9" 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 an inch. At transverse joints the tamper should be drawn not closer than 3 ft. towards the joints and should then be lifted and set down at the joint and drawn backwards away therefrom. Consolidation by 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 is laid to protrude about 1 in. above the side forms.

The tamper is generally made of wood. Plain wooden sections will serve widths up to 15 ft. and if of greater widths they will have to be braced with steel bars for rigidity. An iron plate  $3" \times \frac{1}{8}"$  is fixed to the bottom of the wooden tamper after the wood has been dressed to the required cross-fall. In two-course constructions a notched tamper will have to be made to consolidate the bottom course. Where the road is to be laid in a single bay, the tamper is made for the full width of the road plus 1'-6" for rest on the forms. Where the road width is to be laid in two bays, the tamper is made for the half road width and shaped accordingly.

**Hand Floating.** After the screeding or tamping has been completed the surface should then be floated with a wooden float board 2'-6" long and 3" wide. The float should be operated by a man sitting on a bridge made of a  $1\frac{1}{2}"$  to 2" 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 2 ft. 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 say 30 ft. has been tamped and finished it should be checked thoroughly with a straight edge and any differences of level over  $\frac{1}{4}$ " corrected immediately before the concrete has taken its initial set. The floated surface should then be finished by belting or brooming.

### Surface Finish

A smooth concrete surface is not suitable for animal drawn vehicles. To obtain an exposed aggregate effect (rough surface), immediately after the surface has been smoothed off it should be lightly brushed with a soft hair broom so as to remove all laitance, surface water, etc. After an interval of 12 to 24 hours, depending on the temperature, the surface should be well brushed with a stiff broom to expose the aggregate. If aggregate of uniform size of 1" or  $1\frac{1}{4}$ " gauge are used and surplus mortar is brushed off subsequently, it will leave the stones slightly "proud" on the surface.

**Smoothing Board.** It is generally made of hardwood about 15 ft. long and 6" to 8" wide, and 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.

**Belt finish.** The belt for finishing a concrete road surface is made of stout canvas, from 8" to 12" wide and at least 2 ft. longer than the width of the concrete strip, with handles fixed at the ends. 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.

**Brooming.** The broom should be made of stiff fibre brush with a handle long enough to reach half way across the slab. Brooming should be carried out immediately

after the hand floating in case of dry mixes but preferably after the surface has slightly set. The broom is drawn transversely across the slab, usually once only in most cases, from the centre outward, which leaves the marks of the individual fibres in the concrete. Scorings should not be over  $1/16''$  in depth. A hessian cloth drawn over the surface will also make the surface non-skid.

### **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 3" 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 (sold by The Imperial Chemical Industries) 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. (Also see 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 gallon of solution to about 225 sq. ft. of surface. More than one treatment may be given if necessary.

**Hardening Concrete Surfaces.** See Section 8.

### **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).



### **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" in the pages following.)

### **Kerbs**

Kerbs may be laid directly on the top of the road slab resting about 6 to 9 inches, 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 concrete and the latter can be screeded from them. Kerbs should have joints in continuation of the transverse joints of the road slab. Precast kerbs give excellent service. Minimum size is 6" x 6" and 9" below the road level, edge rounded 2", in 6 ft. lengths, 1 : 2 : 4 concrete. Instead of kerbs, rounded edgings made of precast concrete, 3" thick and of full height of the foot-paths 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 "Road-side Gullies or Inlets". Concrete kerb and channel are usually cast as one piece. The height of the kerb (side of the piece) may be 6" and the width of the channel 1'-6".

### **Repairs to Concrete Roads**

Joints, cracks and small pot-holes should be filled in with hot bitumen mixed with sand after they have been thoroughly cleaned. Cleaning can be done by compressed air. Hair-cracks should be opened out if sides are weak, otherwise no attempt need be made to fill them with bitumen which will not penetrate into hair cracks. Shallow patches not exceeding  $\frac{1}{4}$ " thick can be repaired by coating the surface with a cut back asphalt or emulsion after it has been cleaned and covering with coarse sand or finally crushed stone screenings. Depressions

from  $\frac{1}{2}$ " to 1" can be patched with premixed bitumen and stone chips of size  $\frac{1}{8}$ " to  $\frac{3}{8}$ " after the surface has been cleaned and primed. Deeper patches can also be filled in likewise and well rammed. Patches or pot-holes exceeding 1" in depth may be repaired with concrete. The faces of the old concrete should be thoroughly cleaned and wetted and given a coat of neat cement grout. (Some engineers do not favour a coat of cement grout.) The new concrete should be as nearly as possible, of the same mix as the old and shall be as dry as is consistent with workability. The place to be patched should be completely filled with concrete, slightly proud 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. Patches are very unsightly and unsatisfactory. If the surface shows signs of wear all over, it is best to put on a thin carpet of bitumen.

Where the slabs have settled due to defective sub-grades, the same can be remedied by grouting under pressure through a series of holes drilled not exceeding  $1\frac{1}{2}$ " diameter, with cement and sand grout of the consistency of thin cream. This will fill up voids between the sub-grade and the slab and will also raise the slab up.

**Wheel Tracks or Trackways.** These are known as **Creteways** when made of concrete.

A width of 2 ft. (for each track) is quite satisfactory (with 1'-6" min.) for both fast and slow moving traffic keeping to the path, but widths up to 2'-9" are adopted. Distance between centre to centre of the track varies between 4'-9" to 5'-6" depending on the prevailing axle length. A 6" thick 1 : 2 : 4 concrete slab will be required for trackways on stabilized soil or well consolidated hard natural surface, 5" on medium base, and a 4" 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  $\frac{1}{4}$ "



wide at 45 ft. intervals with butt joints at 15 ft. 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**

(See also under "Cementation and Grouting Method" in the Section on "Reinforced Concrete".)

A coarse aggregate consisting of broken stones, gravel or over-burnt brickbats of size  $2\frac{1}{4}$ " to  $1\frac{1}{2}$ " is laid on a prepared and compacted sub-grade to the thickness required (adding  $\frac{1}{2}$ " 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 7 gallons of water per cwt. 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. The sand used is coarse sand all passing No. 7 B. S. sieve and only 5% passing No. 100 sieve. If large size of aggregate has been used with much of voids, a layer of  $\frac{3}{4}$ " 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 8 ft. 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 % 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  $\frac{1}{4}$ " to  $\frac{3}{4}$ " 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  $\frac{1}{2}$ " to  $1\frac{1}{2}$ ". 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 chip-pings of size  $\frac{3}{16}$ " to  $\frac{1}{2}$ " 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 c. ft. of coarse aggregate and from  $10\frac{1}{2}$  to 15 cwt. of cement are used for 100 c. ft. 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 tons 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 2" for a 4" thick slab and this permits a slightly larger size of aggregate than with the com-



mon method. The pre-mixed concrete is laid rapidly on the sub-grade and rolled before initial setting of the cement starts; a 50 ft. 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 heavier weight than required for the mix and the thickness of the slab; a smaller thickness requires a smaller roller and a stiff mix a heavier roller. A heavy roller will produce corrugations on the surface. The level of the loose concrete is kept  $\frac{3}{4}$ " to  $1\frac{1}{2}$ " in excess 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 18" 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 laying 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 resistance 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 2 inches thick layer of single size stone of size between 2" to  $1\frac{1}{2}$ " (some engineers prefer to use graded stone between 2" to  $\frac{3}{4}$ " ) and give it one or two runs of 6 to 8-ton roller. Cement mortar of 1 : 2 or 1 : 3 and of the consistency of a bricklayer's mortar is spread over the surface (pre-

viously moistened)  $1\frac{1}{2}$ " to 1" thick and on this another 2" 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 1" to  $1\frac{1}{2}$ " thick of hard stone aggregate may be given. In this case the rolling on the base course should be stopped as soon as mortar comes on the top. This method gives a saving of about 20 to 25% of cement and 10% 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 2" layers of stone will consolidate to  $3\frac{1}{2}$ " and will need 110 c. ft. of stone per 100 c. ft. of consolidated thickness.

## 17. GRAVEL, KANKAR, BRICK AND SOIL STABILIZED ROADS

### Gravel Roads

Gravel roads are a layer of compacted gravel graded from fines to pebbles containing binding stuff (clay) in the fines. They are generally built in two courses, foundation course and surface course. A thickness of about 6" is required for light traffic and about 12" for heavy traffic, but such roads are suitable only for light traffic. The size and grading of the gravels should vary from 2" at the bottom to  $\frac{1}{4}$ " at the top. The proportion of fines passing a 200 mesh sieve should be about 10 to 15 per cent and should be 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. The consolidation of gravel should be done in layers not exceeding 4" thick.



### 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  $\frac{3}{4}$ " to  $2\frac{1}{2}$ " 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 3-in. sq. mesh screen.
- (ii) Not more than 20 p.c. is retained on a  $2\frac{1}{2}$ -in. sq. mesh screen.
- (iii) Not more than 10 p. c. passes a 1-in. sq. mesh screen.
- (iv) No quantity passes a  $\frac{1}{2}$ -in. mesh screen.

Soling may be of bricks or kankar. The kankar is generally spread in 6-inch layers. Two parallel mud walls, 8" wide and 6" 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 50 ft., 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 12-foot road, and by not less than 12 men and a leader on a 9-foot 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 3 ft. 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 furlong of road length. Not more than one furlong of the road should be under operation in any one mile 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 4" the whole of the old surface shall be picked up to a depth of 1½" and the roughened surface raked true to template. If it is 4" 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 1 in. in depth and shall be 9 inches 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**

In town streets for light traffic, can be made with bricks laid flat or on-edge over 2 to 3 inches of rammed ballast or lime concrete well consolidated. 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 1 inch 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 9-in. wide profile at right angles to the length of the street, at 8 to 9 feet 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 ½". Where the joints



exceed  $\frac{1}{4}$ ", 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 tons weight. After final rolling the pavement should be tested with a straight edge laid parallel with the kerb, and any depressions exceeding  $\frac{1}{4}$ " 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 STABILIZED 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%
Silt	..	.. 10 to 20%
Total sand	..	.. 70 to 85%
(Coarse sand)	..	.. 15 to 25%
		By weight

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.

From 45 to 60 per cent sand should be retained on a 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%
Sand	..	45%
Aggregate	..	37%

The aggregate is  $\frac{3}{4}$  in. down to  $\frac{3}{16}$  in. well graded. Sand is 85 per cent passing No. 7 B.S. sieve and none passing No. 200 sieve.

Another proportion recommended is :—

35 per cent of stone graded from  $\frac{3}{4}$  in. to  $1\frac{1}{2}$  in. The material passing  $\frac{3}{8}$  in. might show 90 per cent passing  $\frac{3}{16}$  in. B.S. sieve, 65 per cent passing No. 7 sieve, 50 per cent passing No. 52 sieve and 15 per cent passing No. 200 sieve.



Another simple gradation for stabilization :—

Max. particle size 3-ins. graded down. Not more than 5 per cent should be retained on 3-ins. sieve.

Passing No. 4 B. S. sieve .. 40 to 45%

" No. 40 " .. 15 to 100%

" No. 100 " .. Not more than 50%

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.

The percentage passing the No. 200 sieve shall be not more than one-half of that passing the No. 36 sieve for bases, and two-thirds for surfacings.

The following particle-size distribution for bases and surfacings of roads are suggested by Road Research Laboratory, Harmondsworth :—

B. S. sieve size	Percentage passing					
	Base			Surfacing	Base or Surfacing	
	Nominal max. size			Nom. max. size	Nominal max. size	
	3-in.	1½-in.	¾-in.	¾-in.	¾-in.	⅜-in.
3 in.	100	—	—	—	—	—
1½ in.	80—100	100	—	—	—	—
¾ in.	60—80	80—100	100	100	—	—
⅜ in.	45—65	55—80	80—100	80—100	100	—
⅜ in.	30—50	40—60	50—75	60—85	80—100	100
No. 7	—	30—50	35—60	45—70	50—80	80—100
No. 14	—	—	—	35—60	40—65	50—80
No. 25	10—30	15—30	15—35	—	—	30—60
No. 52	—	—	—	20—40	20—40	20—45
No. 200	5—15	5—15	5—15	10—25	10—25	10—25

Not less than 10 per cent should be retained between each pair of successive sieves specified for use, excepting the largest pair.

The two smaller sized material (¾-in and ⅜ in.) may have up to 35 per cent of stones not larger than 1½-in., provided that the material passing the ⅜ in. sieve is within the limits specified.

Where aggregate are used for stabilization, 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 the initial density of the soil is uniform throughout the area to be processed since uneven initial density will result in depressions and bumps occurring in the finished surface. Where a two-course construction is used, good compaction of the base course can be obtained if it is used by traffic for some months before the surface coat is applied. In such cases calcium chloride has been found to facilitate compaction and to preserve the stability of a base when used as a surfacing.

Liquid limits and plasticity indices have to be kept within certain limits according to the properties of the materials used and the climatic conditions. Such limits have been laid down or recommended by many agencies according to their own conditions which are not strictly applicable to all the cases. It is, however, considered that for cohesive soils the liquid limit should not exceed 40 per cent and plasticity index 18 per cent. The higher liquid limits and plasticity indices are desirable in the surfacing in order to provide greater cohesion and to help offset the moisture lost by evaporation but soils with high plasticity indices are not suitable for sub-grade or base courses for which the PI may not exceed 6. If plasticity index is higher than required by design, more sand is added to bring it down. If it is lower than more clay is added.

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  $\frac{1}{2}$ -in. to  $1\frac{1}{2}$ -ins. of coarse sand or cinders or slag, just before the rains also improve the riding qualities of a clay soil.

**Salts in Soils.** 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 soil 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 Cement**

Most coarse-grained soils and many fine-grained soils with clay contents not exceeding about 30 per cent can be stabilized successfully by the addition of 5 to 15 per cent of cement. The material produced by the addition of cement to the soil is termed "soil cement". It has the appearance of very dense soils, it is hard and does not readily dust, but has only limited resistance to abrasion, and forms a "flexible" rather than a "rigid" construction.

For gravelly or sandy soils 6 to 10 per cent of cement; for well graded sandy clays—9 to 10 per cent; for silty soils—8 to 12 per cent; for clayey soils—10 to 14 per cent of cement is usually required based on the dry soil weight. The higher the clay content the more cement is needed. The percentage of cement giving a compressive strength

(for road work) at 7 days of 250 lbs./sq. in. is usually accepted as satisfactory minimum. Test cubes or cylinders are made for measuring the crushing strength. For most soils, increase in cement content increases compressive strength of the soil which is more with sandy soils than with clayey soils. Soils with which this does not occur are generally unsuitable for use in soil-cement construction.

Type of soil is the most important factor in soil-cement stabilization as cement does not set in all the soils. Peaty soils, heavy clays, highly organic soils such as agricultural top soils and soils containing salts of sulphates cannot be successfully treated with cement. (Some experiments have indicated that addition of 5 per cent or more of cement makes the soils resistant to the action of sodium chloride and sodium sulphate salts.) The organic matter in the soil should not exceed 2 per cent. The organic matter content of the soil is determined by the dichromate oxidation method, or the soil is treated with hydrogen-peroxide and the loss in weight of the soil gives the organic matter contents.

The soil is crushed, sieved, mixed in the right proportions and collected in regular and uniform stacks. The quantity of water required per stack to bring the moisture content of the soil up to its optimum moisture, making due allowance for loss by evaporation and absorption, etc., is added uniformly over each stack and the moist soil let to stand for 24 hours (or at least overnight) before cement is added.

Mixing of the soil and cement is done either with the soil considerably wetter than the optimum moisture or with very dry soil. Ordinary concrete mixers may be satisfactory for granular soils but they are not suitable for the more cohesive types of soil because the material sticks in the drum. With such soils, the roller-pan type machines, double paddle mixers and pug mills may be used for mixing on the site or the cement may be ploughed into the soil with the aid of agricultural implements. Where cement is to be mixed by hand, the required quantity of cement is spread over the stack and hand mixing carried out by cutting the stack with spades and mixing till a mix



of uniform colour is obtained. The soil is watered during mixing as required; water should be well distributed throughout the soil so that there will be no dry places where cement cannot hydrate. Consolidation is carried out at somewhat above the optimum moisture content which usually gives the highest density and this also provides adequate water for the hydration of the cement. It is generally considered that with cohesive soils the moisture content selected should be one or two per cent below that at which the mix begins to "push" under the roller. An approximate estimate of this moisture content can be obtained by making a plastic limit test on a sample of the soil-cement mixture. The soil-cement should be placed in position for immediate compaction within  $\frac{1}{2}$  hour after the cement has been added. Not more soil should be handled at a time than can be mixed and compacted within three hours. Good compaction is essential for obtaining high strength with soil-cement which is best obtained by pneumatic or flat rollers.

After compaction the soil-cement is cured under humid conditions that prevent drying of the surface during initial period of the development of strength. This can be done by covering it with 2 ins. of soil, straw or gunny bags and keeping these moistened for at least 7 days after which water sprinkling is continued for 28 days. The mixture sets in 7 to 28 days to give a material that is hard and resistant to the disintegrating effects of water. The drying of soil-cement usually leads to fine shrinkage cracks on the surface which are not always injuries.

The following gradation with 5 per cent of cement by weight of the dry soil will produce a stabilized soil :—

Clay	.. 8 to 15%	Plasticity Index	8.5 to 12.0
Silt	.. 12 to 25%	(See also under	"Stabilized
Sand	.. 60 to 80%	Soil for Building Construction"	
		in Section 7.)	

Particle-size distribution with granular soils :—

Maximum size	..	..	3 ins.
Passing $\frac{1}{8}$ in. B. S. Sieve	..	..	above 50%
Passing No. 36 B. S. sieve	..	..	above 15%
Passing No. 200 B. S. sieve	..	..	below 50%
Finer than 0.002 mm.	..	..	below 30%

A soil-cement built at site gives only about 60 to 80 per cent strength of the same mixture made and tested in a laboratory.

Average Compressive (Cracking) Strengths of Soil-Cement Mixtures in lbs./sq. in. :-

Type of soil	Sand content per cent	L.L.	P.I.	Compressive strength		
				Cement content		
				5%	10%	20%
Sandy loam	64.5	21.6	6.3	1024	2393	3047
Loam	43.1	29.7	10.7	1322	1608	2504
Silty loam	13.9	26.9	9.4	830	1109	1806
Silty clay loam	11.5	36.7	16.2	1233	1650	2895

(Based on the experiments carried out at the P.W.D. Research Laboratory, Karnal, Punjab.)

### 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 1 gal./sq. yd. so that the binder penetrates about  $\frac{1}{2}$  in. to 1 in. 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 roll.

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.

Another particle-size distribution is :-

More than 50 per cent passing a  $\frac{3}{4}$ -in. B.S. sieve, 35 to 100 per cent passing a No. 36 sieve, and 10 to 50 per cent passing a No. 200 sieve.



Typical Particle-size Distributions of Soils for Bituminous Stabilization as Recommended by the Highway Research Board, America :—

B. S. sieve	Percentage passing		
	1½-in. max. size	1-in. max. size	¾-in. max. size
1½ in.	100	—	—
1 in.	80—100	100	—
¾ in.	65—85	80—100	100
⅝ in.	40—65	80—75	80—100
No. 7	25—50	40—60	60—80
No. 36	15—30	20—35	30—50
No. 100	10—20	13—23	20—35
No. 200	8—15	10—16	13—30

It is stated that the material passing the No. 36 B.S. sieve should not be less than 40 per cent of the fraction passing the No. 7 B.S. sieve. The fraction passing the No. 200 B. S. sieve should in no case be more than 60 per cent and preferably not more than 50 per cent of the fraction passing the No. 36 B.S. sieve.

### 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 300 ft. along the centre line of the road alignment or a double line offset 50 or 100 ft. on each side will usually be sufficient. The depth of boring should be 4 to 5 ft. 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 "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. The prac-

tical object in compacting soil is to obtain a high dry density. 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 disturbs 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 inadvisable to compact cohesive soil sub-grades below their optimum moisture content in cases where they are likely to be subject to the ingress or 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 compacted 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 with moisture content about 8 per cent less than the P.L., the sheepsfoot roller gives the highest densities. The rolling is finished off with a flat three-wheel roller 5 to 6 tons in weight. On an average about 60 trips of the roller (sheepsfoot) are required to consolidate a 12 ft. width of the road. Wearing course is rolled with a 5 to 6-ton flat roller. One roller is capable of doing one furlong of sub-grade, 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.

#### **Soil Stabilization 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 a repeated number of rollings with a light roller, say about  $1\frac{1}{2}$  tons. 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

The following measures can be adopted in order to provide a firm road surface :—

- (i) One ft. layer of good soil on top.
- (ii) Where much good earth soil is not available, a good road can be made by a 6-ins. thick layer of rolls of jungle brushwood with 6 ins. to 8 ins. 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 3 ins. to 6 ins. thick loaded with good earth at both ends.
- (iv) Wheel tracks of concrete or bricks : Concrete tracks have been detailed 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  $\frac{1}{4}$ " and sanded within 1" 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 Tracts

A tolerable kachha road can be made if 1 ft. of kallar soil is removed and 6 ins. of earth is put over 6 ins. of good sand.

## 18. DESIGN OF PAVEMENTS

The design of a pavement is governed by traffic density, i.e., the number of vehicles using the road during peak hours, and the maximum wheel load. The daily





	Min.	*Normal desirable
National Highways	150 ft.	200 ft.
Provincial Highways	100 ft.	160 ft.
District Roads	66 ft.	100 ft.
Village Roads	25 ft.	60 ft.

\*Includes land for borrow pits. Land width is the total width required to accommodate roadway, berms, drains, width reserved for future developments.

In a recent conference held in Paris on accident statistics it was considered that the width of the road must be adequate for the amount of traffic: outside built-up areas, a two-lane road is adequate for volumes up to 6,000 vehicles per day, a three-lane road is adequate for volumes up to 10,000 vehicles per day and a four-lane road is adequate for volumes up to 20,000 vehicles per day. 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.

According to the experiments carried out in America the basic (max.) capacity of a two-lane road is 2000 passenger vehicles per hour, all moving in the same direction and none in the other, or 1000 vehicles in each direction. The basic capacity of a three-lane, two-way road is 4000 vehicles per hour as the centre lane of the three-lane road enables vehicles passing in either direction. This is under ideal conditions of a straight road when all the vehicles move with the same speed of over 30 m.p.h. with no slow moving vehicles to cross. As traffic volume increases, top speed decreases, capacity also decreases with decrease in speed. The maximum capacity of a two-lane road, under above stipulated conditions, is cut to almost half when vehicles speed is decreased to 10 m.p.h. At 30 m.p.h. the minimum spacing of vehicles centre to centre is about 80 ft. whereas at 10 m.p.h. it is 45 ft.

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 30 m.p.h. It is, however, not economical to design a road so wide as to be congestion free every hour throughout the day.

On uphill stretches, vehicles spacings can be smaller, as the breaks are more effective, which permits increased capacity but restrictions of sighting distances decreases capacity.

For counting equivalents of trucks or slow moving vehicles to passenger vehicles, see under "Roundabouts or Traffic Rotaries". Heavy vehicles are lesser at peak hours than for average conditions. Width of a lane is 12 ft. *Traffic Capacity in Town Streets :*

Basic capacity considered is 1250 passengers vehicles per hour of "green light", per traffic lane of 10 ft. width, all moving in the same direction. In general it will be about 80 per cent of the possible capacity with 10 per cent of commercial (slow moving) vehicles or 7 per cent of bullock carts. Counting is done at street intersections and street width is taken from kerb to kerb.

*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.

The British Ministry of Transport use the following figures to determine the type of carriageway to be provided in "out of town" areas :

Peak vehicles per hour	Carriageway
Under 300 .. ..	Two-lane (22 ft.)
300 to 600 .. ..	Three-lane (30 ft.)
600 to 1500 .. ..	Dual two-lane
Over 1500 .. ..	Dual three-lane

The figures are taken of the average daily volumes over a 16-hour count, the peak figure being taken as one-tenth of the daily figure.

"In the absence of actual data based on observations it may be assumed that at the peak hours a traffic lane may accommodate about 200 slow moving and 250 motor vehicles all moving in one direction with no overtaking. With traffic in both directions this will be further reduced. There will be still further reductions at intersections and cross roads."—I. R. C.

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.



### Thickness of Pavements Design

The function of pavement design is to design the thickness and type of construction which will distribute traffic loads sufficiently to enable the sub-grade to support these loads. Two types of construction are considered in pavement design—rigid (concrete) and flexible (bituminous surfacing on a pitching, macadam, gravel or hard-core base). The only satisfactory methods of pavement design at the present time are wholly or partly empirical.

The thickness of a pavement depends upon the bearing capacity of the soil, i.e., the sub-grade on which the road is to be built, and the volume and character of the traffic expected to use the road. Thickness is increased for heavier loads and a larger total volume of traffic. 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 conditions in a given case and the basis of design should be the strength of the sub-grade in that condition. Testing the compacted soil for bearing strength at 100 per cent saturation is essential for areas subject to water-logging and floods. The sub-grade should be well compacted so as to reduce as much as possible the water it can absorb. The bearing value of a soil can be tested as explained in the Section on "Foundations". The test should be made after the sub-grade has been compacted. 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 subject to water-logging. The materials in successive layers downwards of a pavement crust may be progressively weaker.

A water-table less than 2 ft. 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 sand depends on its density therefore, it is important to

compact sandy sub-grades to the max. density possible.

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.

The sub-grades have the property of recovery if the stresses are not of a repeated nature and do not exceed the safe bearing capacity of the sub-grade. A certain amount of occasional and infrequent overstressing the pavement is not harmful. Therefore, it will be uneconomical to design pavements for unusual heavy loads passing once in a way.

Each time a load passes, some deflection of the surface and the underlying layers occurs and the useful life of a road is shortened. Repetition of loads has a destructive effect and repeated applications of an excessive load will lead to cracking and ultimate failure. An occasional overload of twice the design load will not be destructive.

#### **Design Load**

Bridge design is based on gross weight of vehicle while pavement design is based on axle load, i.e., wheel load and intensity of traffic.

For the present, for traffic loads in India, an axle load of 18000 lbs. or a single wheel load of 9000 lbs. is considered to be the limit for the design of pavement thickness. This wheel load of 9000 lbs. is further regulated by the rated capacity of the tyres. It is considered that two tyres will be used for the 9000 lbs. wheel load, each tyre carrying 4500 lbs. weight. Designing for a single-tyre load of 5000 lbs. with radius of contact circle of 4.5 inches, and a tyre pressure of 60 lbs./sq. in. seems adequate as it will cover two tyres with 4500 lbs. loading on each. The amount of load transferred from one tyre of a dual wheel to the point of maximum sub-grade pressure beneath the other tyre is not in excess of 10 per cent. Dual wheels are considered as one wheel load and one contact area if tyres are within 40 ins. centres. (An axle load is the total load transmitted to the road by all wheels whose centres may be included between two parallel vertical planes 40 inches apart.)

The main problem in India is of the iron-tyred bullock carts which may carry loads of 2 to 3 tons with very narrow



tyres of curved cross section and the area of the wheel in contact with the road is much less than it should be. Wheel axles are usually bent which seldom fit well and often have an oscillating motion throwing the whole weight on an edge of the tyre which tends to disintegrate the surface. The load intensity of an iron-tyred laden cart may be much more than due to a 10 or 12 ton road roller used for consolidation. Where cart traffic predominates, take load of the heaviest loaded cart and the width of its tyre; take square contact area. Usual width of iron tyres is 1 to 3 ins. In the absence of a definite data, take 1800 lbs. on one inch square contact area, which will meet most of the cases.

In addition to the wheel loads, impact effect is to be considered which may vary from 10 per cent to over 200 per cent of the wheel load. Impact is affected by the roughness of the surface, the nature and condition of the tyres, tyre pressure, the speed of the vehicle and the wheel load. A steel-tyred or hard-tyred wheel produces greater impact than a pneumatic tyred wheel. Also, faster a vehicle moves, the greater the stresses in the surface layer. A pneumatic tyred wheel at a speed of 30 m.p.h. produces the same impact as a solid tyred wheel at 12 m.p.h. The usual allowance given for impact effect is 20 to 50 per cent of the wheel load.

For concrete pavements the bearing power of a soil is usually taken as the *modulus of sub-grade reaction or co-efficient of sub-grade reaction*. This co-efficient expresses the stiffness of the sub-grade which is a measure of the resistance of the soil (prepared sub-grade) to penetration by a loaded steel plate 30 ins. in diameter and is stated in terms of load in lbs. per sq. in. of deflection. It is assumed that the sub-grade reaction at any point is proportional to the deflection. The values range from about 50 lbs./c.in. for very poor sub-grades up to about 300 for good materials and may be 700 for extremely good soils. A value of 100 lbs./c.in. is taken for general use. Reasonable variations in the values do not have serious effects on stresses. Moreover when a layer of selected base course is placed under the concrete pavement it will further minimize the effect of variation in the value of the co-efficient. This quantity

occurs in Westergaard's theory of the stresses and deflection in concrete slabs.

**Suitability of Road Materials as Regards Density of Traffic :**  
Per day per traffic lane—

- |  |   |
|--|---|
| (a) Gravel roads   | 50 tons of iron tyred vehicles,<br>or 80 to 100 tons of pneu-<br>matic vehicles.                      |
| (b) Water-bound maca-<br>dam                                 | 1000 tons of combined iron<br>tyred and pneumatic tyred<br>vehicles.                                  |
| (c) Bituminous surfaces<br>(surface treated<br>with bitumen) | 1200 tons of pneumatic tyred<br>and 500 tons of iron tyred,<br>or 750 tons all iron tyred<br>traffic. |
| (d) Pre-mixed bitumin-<br>ous or Grouted                     | 5000 tons combined traffic,<br>or 1200 tons iron tyred.   |
| (e) Semi-grouted   | 3000 tons combined traffic.   |

The above figures are only rough approximations and largely depend upon how the road has been built.

**Thickness of Pavements—Empirical Formulae**

**Flexible Pavements :**

- (i) \*Gray's formula: (Suitable  
for granular bases)

$$d = 0.564 \sqrt{\frac{W}{B}} - L$$

where :

$d$  = total thickness of pavement in inches,

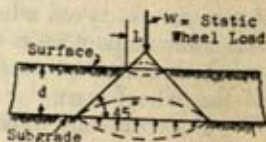
$W$  = static wheel load in lbs. (add for impact or repetition  
of loads = 20 %).

$B$  = bearing value of the ground or base in lbs./sq. in. (See  
note at bottom of table giving "Bearing Values of  
Soils" at page 18/151)

$L$  = radius of equivalent circular area of tyre contact in

$$\text{inches} = \sqrt{\frac{\text{contact area}}{\pi}}$$

Static wheel load is taken acting on its contact area and the load dispersed downwards conically at an angle of 45° in the form of truncated cone, distributing the load





over an area = ( contact width + 2t ). If the static wheel load is based on rubber tyres, the contact area is circular or elliptical. The unit pressure and required bearing value at any plane can be computed. Or, if the bearing value is known the required pavement and base thickness can be worked out.

(ii) *Boussenesq's formula*

$$d = r \times \sqrt{\frac{P}{p} - 1}$$

$$(iii) d = 0.564 \sqrt{\frac{W}{4P}}$$

where :

d = metal depth in inches; r = same as L in Gray's formula;  
p = tyre pressure in lbs./sq. in.; P = safe max. permissible pressure on the sub-grade in lbs./sq. in.

### **C. B. R. (California Bearing Ratio) Method :**

It is the most widely used and reliable of all methods. The chief difficulty in this method is to decide under what conditions of dry density and moisture content the C.B.R. values should be determined. American practice is to soak the specimens before test. British experience suggests that this gives thickness considerably greater and that a more satisfactory guide is the value determined in situ of the undisturbed samples of the sub-grade soil. However, values should preferably, be determined of the wettest conditions likely to be met with.

C.B.R. is a measure of the bearing power of a sub grade or soil expressed as the percentage of the bearing power of a water-bound macadam construction or an excellent base course of crushed stone of 100 per cent value. This is an arbitrary figure : a surface having a C.B.R. of 100 per cent is one in which a load of 3000 lbs. has to be exerted to drive in a cylindrical flat punch with a base area of 3 sq. in. to a distance of 0.1 in. at a rate of penetration of 0.05 in. per min. C.B.R. value increases sharply with increase in density. (See also under "Soil Mechanics.")

The following table gives the total base and pavement thickness over any sub-grade or sub-base of known C.B.R. value for the particular material, for flexible pavements :-

C.B.R. per cent	4	5	6	8	10	15	20	30	40	50	60	80
5000 lbs. wheel load	13"	12"	11"	9½"	8½"	6½"	5½"	4½"	3½"	3½"	3"	2½"
7000 lbs. wheel load	15½"	13½"	12½"	10½"	9½"	7½"	6½"	5"	4½"	4"	3½"	3"
9000 lbs. wheel load	17"	15"	13½"	11½"	10"	8"	7"	5½"	4½"	4½"	4"	3½"
12000 lbs. wheel load	19½"	17"	15½"	13"	11½"	9"	7½"	6"	5"	4½"	4"	3½"

The above figures contain safety factor and are for tyre pressure of 60 lbs./sq. in. The wheel loads are static wheel loads. Gross wheel load is static wheel load plus impact.

In the absence of laboratory tests approximate values for C.B.R. may be taken from the table on "Bearing Values of Soils" at page 18/151.

The C.B.R. method is considered to give extravagant thickness for roads which have to carry only light traffic.

#### Asphalt Institute (U.S.A.) Method :

This method gives total thickness of asphaltic concrete required over any base or sub-grade of known bearing value. Should not be used for low type pavements or for base thickness.

Bearing value of base or sub-grade lbs./sq. in.	Total thickness of asphaltic concrete pavement on a wheel load of	
	60 lbs./sq. in.	70 lbs./sq. in.
20	6"	7"
30	4"	5"
40	2½"	3½"
50	1"	2½"
60	..	1"

The bearing values for application to Gray's formula and the Asphaltic Institute table are taken on a circular plate of the same contact area as that of the design wheel load, causing a deflection of 0.5 inch.



Bearing Values of Soils (Sub-grades)

No.	Description of soil	Classification	Bearing * value lbs./sq. in.	Value of "c" for Sheets formula	Approx. C. B. R. per cent
1.	Marshy clay .. ..	Very soft	5	1.10	2
2.	Heavy clay, soft and plastic, poor drainage	Very poor, soft and plastic	10	1.00	3
3.	Black-cotton soil .. ..	..	13	0.98	4
4.	Heavy clay soils or clay soils where drainage is uncertain, silty clay	Poor	15 17	0.95 0.92	5 7
5.	Sand and clay mixtures poorly graded	Fairly hard	20 25	0.90 0.87	10 20
6.	Sand, sandy clay, gravel fairly graded	Hard-good	30	0.84	25
7.	Compact clay .. ..	..	35	0.82	30
8.	Gravel or kankar with fines, well graded	Very hard	40	0.80	40
9.	Good gravel or macadam ditto ..	Extremely hard	50	0.77	50

\* Bearing value is measured as the pressure which applied to the sub-grade uniformly over a circular area 3 ft. diameter produces a deformation of 0.10 inch.

**Stone Macadams :**

$$d = \sqrt{\frac{W}{3P} + \frac{T^2}{9}} - \frac{T}{3} \quad T = \text{width of tyre in inches.}$$

**Rigid Pavements (Cement Concrete Roads) :***Sheets' formulae:*

$$d = \sqrt{\frac{1.92 Wc}{S}} \quad \text{(For slabs where corners are protected by adequate load transfer device at the joints—dowel bars, etc.)}$$

With pneumatic tyres.

$$d = \sqrt{\frac{2.4 Wc}{S}} \quad \text{ditto.} \quad \text{with solid tyres}$$

$$d = \sqrt{\frac{2.4 Wc}{S}} \quad \text{Where corners are unprotected, no adequate load transfer at the joints. With pneumatic tyres.}$$

$$d = \sqrt{\frac{3 Wc}{S}} \quad \text{ditto.} \quad \text{with solid tyres.}$$

*Older's formula* (Cantilever formula—when sub-grade support is uncertain) :—

$$d = \sqrt{\frac{3 W}{S}} \quad \text{—edge thickness of a plain slab (where the concrete is thicker at the edge than at the centre). The depth at the centre of the pavement is generally made about 0.7 times the edge thickness.}$$

If continuous edge shear bars are provided then

$$d = \sqrt{\frac{1.5 W}{S}}$$

where :

d=thickness of plain slab in inches ;

W=moving wheel load in lbs. (without impact). Add for impact or an allowance for repetition of loads, 10 to 20 % ;

c=co-efficient of sub-grade support (from table at page 18/151).

S=allowable flexural unit stress in concrete in lbs./sq. in. Flexural strength (fatigue modulus of rupture) of concrete in tension is taken for the design of pavements. Beam test for determining flexural strength



(BM/Z) is given in the Indian Standard Code of Practice, IS : 456—1953. The value usually taken is 300 to 350 lbs/sq. in. which gives a factor of safety of 2 over the static modulus of rupture. This will allow practically unlimited stress repetitions caused by the design load.

The flexural test is particularly well suited for field use, since the testing device is relatively small and light.

*If a road slab is designed on the basis of shear :*

$W = d \text{ (contact width} + 2d) \times \text{permissible shear stress.}$

*Adam's formula for Reinforced Concrete Slabs :*

$$d = \frac{\sqrt{4.5 W}}{w} \quad \begin{array}{l} w \text{ is the bearing value of the soil} \\ \text{in tons per sq. ft.; } d \text{ is depth in} \\ \text{inches to centre of reinforcement.} \end{array}$$

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.

Westergaard's formulae for stresses in rigid pavements are generally used for the design of important works (concrete roads and airport runways) but as these formulae are rather complex and also give high stresses in thin slabs than have been found from experience, therefore, have been omitted.

#### Base and Slab Thickness for Various Sub-grades for Reinforced Concrete Pavements

The following table is based on the recommendations of the Road Research Laboratory, Harmondsworth and the Cement and Concrete Association, London :—

Condition of Sub-grade	Thickness of base	Type of traffic				
		Very heavy	Heavy	Medium heavy	Medium	Light
		Thickness of reinforced slab in inches				
(a) Very stable	nil	9"	8"	7"	6"	5"
(b) Stable	0"—3"	10"	9"	8"	7"	6"
(c) Poor	up to 6"	11"	10"	9"	8"	7"
Reinforcement per sq. yd.		14 lbs.	10 lbs.	10 lbs.	7 lbs.	5 to 7 lbs.

Unreinforced slabs are not recommended except under light loads and on very stable sub-grades. If reinforcement is not provided increase thickness by one inch.

*Description of sub-grades :*

(a) Very stable sub-grades are well compacted and undisturbed foundations of old roads; well graded highly stable gravel; solid rock. No base is needed on such sub-grades except where a levelling course is required.

(b) Stable sub-grades 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 a 3-in. base.

(c) Poor sub-grades are soils very susceptible to non-uniform movement; loams, peat and plastic clays.

(d) Sub-grades where the water-table may rise to within 2 ft. of the formation: No base is needed on soils which can be thoroughly compacted, otherwise a base thickness of 3 ins. to 6 ins. is desirable.

Where a hard base course has been given higher value for the sub-grade should be adopted.

*Traffic classification :*

Type of Traffic	Total daily av. flow of commercial vehicles (both directions)	Approx. total weight of all traffic (tons/day)
Very heavy	More than 3000	Above 24000
Heavy	1500—3000	12000—24000
Medium-heavy	450—1500	4000—12000
Medium	150—450	1500—4000
Light	45—150	Less than 1500

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. An 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.

**Slab thickness for Concrete Pavements** based on the commendations of the Portland Cement Association, America :—



Allowable flexural stress in lbs./sq. in.	Wheel load on dual tyres lbs.	Co-efficient of sub-grade reaction				
		50	100	200	300	500
		Thickness of slab in inches				
250	12000	9.50	9.25	8.75	8.50	8.00
	10000	8.75	8.50	8.00	7.75	7.40
	9000	8.25	7.90	7.60	7.30	7.00
	8000	7.75	7.60	7.10	6.90	6.60
	6000	6.75	6.50	6.20	6.00	5.75
	5000	6.40	6.10	5.80	5.60	5.40
300	12000	8.60	8.20	7.80	7.50	7.20
	10000	7.80	7.50	7.20	6.90	6.60
	9000	7.40	7.10	6.80	6.50	6.20
	8000	7.00	6.75	6.40	6.10	5.80
	6000	6.10	5.75	5.50	5.40	5.10
	5000	5.75	5.40	5.10	5.00	4.75
350	12000	7.80	7.50	7.10	6.80	6.50
	10000	7.10	6.80	6.50	6.20	6.00
	9000	6.80	6.50	6.10	5.80	5.60
	8000	6.40	6.10	5.80	5.50	5.25
	6000	5.50	5.30	5.10	4.90	4.60
	5000	5.10	4.90	4.75	4.50	4.10

Slab corners are considered to be protected by load-transfer devices. The slabs are not reinforced and where reinforcement is provided thickness may be reduced by one inch. This is for heavy-duty pavements.

### 19. GRADIENTS

The rate of rise or fall of a road surface is called road gradient or grade and is expressed as a ratio of 1 vertical in—horizontal which is the distance measured along the length of the road. Ruling gradient is the desirable gradient, and limiting gradient is the maximum allowable gradient.

#### Motor Roads :

Ruling gradient on plain roads	..	..	1 in 30
Ruling gradient on hill roads	..	..	1 in 20
Limiting gradient in stretches not exceeding 300 ft.	..	..	1 in 15
Absolute max. gradient for short lengths not exceeding 300 ft. per mile	..	..	1 in 12

**Jeep Roads :**

Ruling gradient	..	..	1 in 15
Limiting gradient	..	..	1 in 12
Absolute max. gradient for short lengths	..	..	1 in 10
Gradient at curves should not exceed	..	..	1 in 30
Max. sustained grade with safety in operation			1 in 20

Short and frequent changes in grades result in bumpy surfaces.

The absolute maximum gradient which can be used by animal traction is 1 in 12 (or  $8\frac{1}{2}$  per cent). This is to be used only in hill roads where unavoidable, for not more than  $\frac{3}{4}$  mile at a stretch. A gradient of 1 in 10 can be given where absolutely essential for distances not exceeding 300 ft. 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.

Gradient at hair-pin bends or other sharp curves with inside curve of 25 to 50 ft. should never exceed 1 in 20 (prefer 1 in 25) and should be as flat as possible for 100 ft. above hair-pin bends, even if it is necessary to increase the gradient for a short distance beyond this length of 100 ft. Curves with radius of less than 10 ft. should not have any rise.

When a curve with a radius of less than 500 ft. occurs on a grade of more than 5 per cent (1 in 20), the grade on the curve should be reduced below that of the tangent by not less than  $\frac{1}{2}$  per cent for each 50 ft. by which the radius is below 500 ft. For example, if a curve whose radius is 400 ft. adjoins a tangent on a grade of 7 per cent, the grade on the curve should be not greater than :

$$7 - \frac{1}{2} \times \left( \frac{500 - 400}{50} \right) = 6 \text{ per cent.}$$

A simple rule for grade reduction used in America is :  
Reduction per cent of grade =  $250/R$ . The use of grade reduction is commonly restricted to curves sharper than 6 degs. and to grades 5 per cent or greater. The breaks in grade line are placed on the tangents, about 50 ft. from the ends of the circular curve.



This treatment is required in mountainous country where the combination of sharp curves with steep grades occur.

Long stretches of steep gradients should be separated by comparatively easier grades or level lengths over 100 ft. long which should be given every half mile or so. Total rise per mile should be kept below 250 ft. for important roads and 300 ft. for other hill roads. Efforts should be made to provide flatter gradients at high altitudes as the efficiency of an automobile engine is considered to fall with increase in altitude.

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.

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 3 ft.-9 ins. 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.

#### **Pack Mule roads in hills :**

Animals can draw only the following percentages of weight on an incline :

	Per cent		Per cent
1 in 10	25	1 in 40	70
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 300 ft. ; rise per mile must not exceed 750 ft. Curves with radius of less than 10 ft. should not have any rise.

Maximum gradient for pedestrian paths is 1 in 5 (prefer 1 in 7.5).

## Road Grades

Inclination 1 in—	Angle with horizon Deg.-Min.	Rise per mile ft.	Inclination 1 in—	Angle with horizon Deg.-Min.	Rise per mile ft.
5	11—18½	1056	30	1—54½	176
6	9—28	880	33	1—44½	160
7	8—7½	754	35	1—38½	151
8	7—7½	660	37	1—33	143
9	6—21	586	40	1—26	132
10	5—43	528	43	1—16½	123
11	5—12	480	45	1—10½	117
12	4—46	440	50	1—8	106
13	4—24	406	55	1—2½	96
14	4—5½	377	60	0—57½	88
15	4—49	352	70	0—49	76
16	3—34½	330	80	0—43	66
17	3—22	311	90	0—38½	59
18	3—11	293	100	0—34½	53
19	3—1	278	110	0—31½	48
20	2—51½	264	115	0—30	46
22	2—36	240	120	0—28½	44
24	2—23	220	125	0—28	42
25	2—17½	211	150	0—23	40
27	2—7	196	200	0—17½	27
28	2—2	189	400	0—8½	13

Stadia Reductions Chaining over Sloping Grounds  
Difference in Elevation for 100 ft. Inclined Distance

Min.	0 deg.	1 deg.	2 deg.	3 deg.	4 deg.	5 deg.	6 deg.
0	0.00	1.74	3.49	5.23	6.98	8.72	10.45
10	0.29	2.04	3.78	5.52	7.27	9.00	10.74
20	0.58	2.33	4.07	5.81	7.55	9.29	11.03
30	0.87	2.62	4.36	6.10	7.85	9.58	11.32
40	1.16	2.91	4.65	6.39	8.15	9.87	11.61
50	1.45	3.20	4.94	6.68	8.43	10.16	11.90
Min.	7 deg.	8 deg.	9 deg.	10 deg.	11 deg.	12 deg.	13 deg.
0	12.19	13.92	15.64	17.36	19.080	20.79	22.49
10	12.48	14.20	15.93	17.65	19.37	21.08	22.78
20	12.76	14.49	16.12	17.94	19.65	21.36	23.06
30	13.05	14.78	16.50	18.22	19.94	21.64	23.34
40	13.34	15.07	16.79	18.51	20.22	21.93	23.63
50	13.63	15.36	17.08	18.79	20.51	22.21	23.91



Min.	14 deg.	15 deg.	16 deg.	17 deg.	18 deg.	19 deg.	20 deg.
0	24.19	25.88	27.56	29.24	30.90	32.56	34.20
10	24.47	26.16	27.84	29.51	31.18	32.83	34.47
20	24.76	26.44	28.12	29.79	31.45	33.11	34.75
30	25.04	26.72	28.40	30.07	31.73	33.38	35.02
40	25.32	27.00	28.63	30.35	32.01	33.65	35.29
50	25.60	27.28	28.96	30.62	32.28	33.93	35.56

## Horizontal Distance for 100 ft. Inclined

Min.	0 deg.	1 deg.	2 deg.	3 deg.	4 deg.	5 deg.	6 deg.
0	100.0	99.98	99.94	99.96	99.76	99.62	99.45
10	100.0	99.98	99.93	99.85	99.74	99.59	99.42
20	99.99	99.97	99.92	99.83	99.71	99.57	99.39
30	99.99	99.97	99.90	99.81	99.69	99.54	99.36
40	99.99	99.96	99.789	99.67	99.67	99.51	99.32
50	99.99	99.95	99.88	99.78	99.64	99.48	99.29

Min.	7 deg.	8 deg.	9 deg.	10 deg.	11 deg.	12 deg.	13 deg.
0	99.25	99.03	98.77	98.48	98.16	97.81	97.44
10	99.22	98.99	98.72	98.43	98.11	97.75	97.37
20	99.18	98.94	98.68	98.38	98.05	97.69	97.30
30	99.14	98.90	98.63	98.32	97.99	97.63	97.24
40	99.11	98.86	98.58	98.27	97.93	97.57	97.17
50	99.07	98.81	98.53	98.22	97.87	97.50	97.10

Min.	14 deg.	15 deg.	16 deg.	17 deg.	18 deg.	19 deg.	20 deg.
0	97.03	96.59	96.13	95.63	95.11	94.55	93.97
10	96.96	96.52	96.05	95.54	95.01	94.46	93.87
20	96.89	96.44	95.96	95.46	94.92	94.36	93.77
30	96.81	96.36	95.88	95.37	94.83	94.26	93.67
40	96.74	96.28	95.80	95.28	94.74	94.17	93.56
50	96.67	96.21	95.71	95.19	94.65	94.07	93.46

## 20. CAMBER OR CROSS FALL

Camber has been defined earlier.

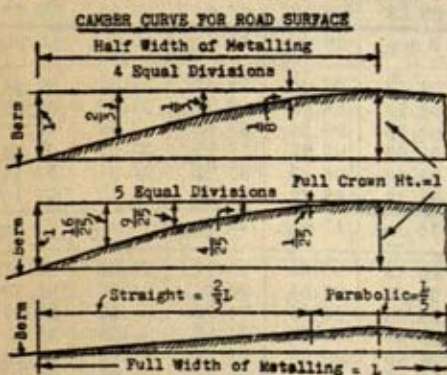
Excessive camber induces drivers to keep near the crown of the road and tends to the uneconomic use of the highway. A cross-fall which is excessive increases the tendency of vehicles to side-slipping. Therefore, excessive camber should be avoided and surface made as flat as can be permitted.

Since the object of cross-fall is to facilitate drainage of the surface water from the road the rougher 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. Dry areas are made flatter than areas subject to heavy rainfalls. 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 templates are made according to the proposed camber of the road, which are fixed at 50 to 100 ft. apart across the centre line of the road.

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 four 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.



Bituminous roads in towns with speed limit	1 in 36 to 40
Bituminous high speed roads outside towns	1 in 48 to 60
Cement concrete roads	1 in 60 to 72
Waterbound macadam	1 in 30 to 48
Moorum and Kankar roads	1 in 24 to 30
Earth roads and footpaths	1 in 20 to 24

(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.



## 21. CYCLE TRACKS & FOOT-PATHS

### Cycle Tracks

The width of cycle tracks should normally be 9 ft. and should in no case be reduced to less than 6 ft. for up to 2000 cycles per hour, increasing in multiples of 3 ft. per 1,500 cycles more. Cycle tracks may be made for over 400 cycles at peak hour. Max. gradient is 1 in 20.

Cycle tracks should be separated from the carriageway and footpaths by grass verges of not less than 6 ft. and 3 ft. respectively. Should it not be practicable to obtain sufficient space for the verges, the cycle track may be separated from the carriageway and footpath by kerbs or fencing. These tracks should be provided on both sides of important carriageways.

### Foot-Paths

The width of a foot-path is taken 24 inches per 20 pedestrians or less per minute. Min. width should be 5 ft. In principal shopping centres, width should be 15 ft. min. for the foot-ways. Height should be 6 inches (min.) from road edge level.

## 22. 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 arcs) of different radii which lie on the same side and have a common tangent and a common tangent point. Compound curves are not recommended 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.

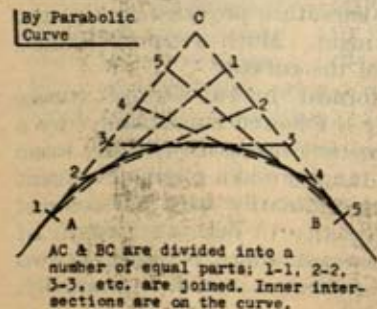
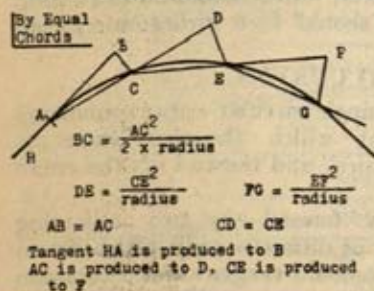
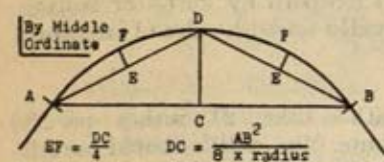
A **Reversed curve** is formed by two simple curves in which a right-hand curve is followed immediately by a left-hand curve, or *vice versa*, not necessarily of the same radii, which have a common tangent and a common tangent point. Reversed curves are ordinarily used to connect parallel lines. It is good practice to insert a length of straight (say 200 ft.) or an easement curve between the two curves, which introduces the change in radius gradually.

Super-elevation is changed along this transition section. Compounding circular curves of greatly different radii is considered poor practice.

**Transition curves**—See under "Super-elevation."

**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 the hill and which is a concave curve is known as *re-entrant curve* or *open curve*. Serpentine curves, hairpin bends and corner ends have also to be provided in hill alignments.

#### SETTING OUT CURVES



Curves of small radii should be set out in chords not exceeding 1/20th of the radius. 100 ft. chords to be used up to 5°; 50 ft. chords for 5° to 15°; 25 ft. chords from 15° to 30°. Curves of 1/2 mile radius and upwards can be set out with 200 ft. chords. Some engineers prefer to use 100 ft. chords for radii 2000 ft. or more; the difference in length between a chord of 100 ft. and its arc is practically negligible. Refer to the first figure of the illustration. Curves can also be set out by marking offsets from the long chord AB, by calculating a number of offsets as per equation given on the next page for calculating "O".





A curve is designated by its radius in feet, or by the number of degrees of the central angle covered (or subtended) by 100 ft. length of the chord of the curve. For practical purposes the length of the chord and the arc is taken as the same. Thus a  $1^\circ$  curve is a curve in which a chord of 100 ft. is subtended by a central angle of  $1^\circ$ . It is called *Degree of Curvature*.

A circle with 5730 ft. radius has a circumference of 36,000 ft. subtending  $360^\circ$ . A 100 ft. chord (or curve) has  $1^\circ$ . Or in other words, a  $1^\circ$  curve is considered to have a radius of 5730 ft.; a  $2^\circ$  curve 2865 ft., and so on (with 100 ft. chord). This is not absolutely correct but a close approximation for a  $5^\circ$  curve or any flatter curve.

$$R = \frac{5729.58}{\phi} \quad \text{or precisely} = \frac{50}{\sin \frac{1}{2} \phi}$$

### Vertical Curves

A curve with convexity upwards is called a Summit Curve or a Vertical Curve. Vertical curves are necessary at all points where a change of gradient occurs. The design of a curve (length) is principally governed by the sight distance required to provide adequate visibility, unless the summit is so small (less than 3 ft. 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 tangentially.

**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 known as the *non-passing sight distance*.

**Sighting or Sight Distance.** Is the length of the road ahead of the vehicle which is visible to the driver. On a vertical curve the sighting distance represents the distance apart of two vehicles approaching from opposite directions, when they just become visible to each other.

To provide visibility, hill sides have to be cut down at curves and bends.

**Overtaking Sight Distance.** Is the distance required for a moving vehicle to overtake and safely pass another



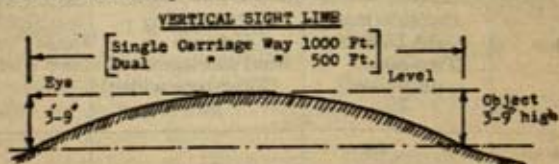
vehicle moving in the same direction but at a slower speed. The minimum distance ahead that must be clear to permit safe passing is called the *passing sight distance*.

From actual tests made, a car travelling at 30 m.p.h. and passing a car at rest, requires 117 feet to turn on the traffic line and return to it. A car travelling at 60 m.p.h. and passing one travelling at 45 m.p.h. would require a distance of 468 feet. Hence for two cars coming from opposite directions at 60 m.p.h. a sight distance of 936 feet would be required. As such, a sight distance of 1000 feet on national highways outside towns should be aimed at.

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. Majority of roads in India, in fact, are single carriage-ways but in practice are used for two-way traffic by the temporary use of road side berms. The safe sight distance is dependent on the braking power of the vehicle and is generally taken as being equal to twice the distance required for a vehicle to stop.

American practice usually requires from 350 to 500 ft. minimum sight distance between any two points 4'—8' above the road surface for horizontal and vertical curves.

British Ministry of Transport recommendations :—



Desirable minimum sighting distance for single carriageway roads is 1000 ft. and dual carriageway 500 ft. at 3'-9" height above road level. Where the speed is limited to 30 m.p.h. this may be reduced to 500 ft. When the height point of the vertical curve is elevated not more than 3 ft. (some engineers recommend 4 ft.) above the lowest point on the curve, no sighting distance need be considered.

Some British engineers recommend 500 ft. and 150 ft. for dual carriage-ways, and 900 ft. and 250 ft. for single carriageways for 60 m.p.h. and 30 m.p.h. speeds respectively. But these stopping distances are too small for Indian conditions.

Where for steep gradients to the approaches a vertical curve cannot be avoided, the summit of the curve (or hump) should be made horizontal for some distance, say 100 ft.

Same visibility is recommended on horizontal curves as on vertical curves. The sight distance is measured along the centre line of the road.

### Speed Standards for Design (I.R.C.):

			Flat country	Hills or urban area
National and Provincial Highways	..	..	50 m.p.h.	30 m.p.h.
Major District Roads	..	..	40 "	25 "
Other District Roads	..	..	30 "	20 "
Village Roads	..	..	20 "	15 "

Turning speed should be taken at 0.7 of the speed of through road.

The Specifications and Standards Committee of the I.R.C. have recommended the following Standards:—

Design Speed in m.p.h.	Safe Stopping Distance	Min. Sight Distance	Min. Overtaking Sight Distance (Two-lane undivided)  ft.
	Absolute min. Sight Distance (Two-lane undivided) ft.	Overtaking distance on dual carriageways (Four-lane width) ft.	
15	90	..	..
20	120	200	300
25	160	250	450
30	200	350	600
35	250	450	800
40	300	550	1000
50	400	700	1450

**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 3 times the distance given above should be provided at frequent intervals. Roads of 12 ft. width and less where there are no berms such as canal and village roads, overtaking places of not less than 200 ft. length should be provided at frequent intervals.

### Horizontal Curves Standards for Minimum Radii in Curves (I.R.C.)

Flat or rolling country			Hilly country		
Design speed m.p.h.	Min. Radius in ft.		Design speed m.p.h.	Min. Radius in ft.	
	Ruling	Absolute		Ruling	Absolute
50	1000	800	30	400	300
40	800	500	25	300	200
30	500	300	20	200	150
20	300	150	15	150	100

A minimum radius of 60 ft. with 40 ft. in exceptional cases, is being used on some of the hill roads in India. California practice is to have a minimum radius of 200 ft. for "blind curves" and 100 ft. 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 500 ft. long for  $\phi = 5$  deg. which should be increased 100 ft. in length for each decrease of 1 deg. in the  $\phi$ . Where topography permits, use simple 20 minutes to 1 deg. curves without super-elevation or widening.

The straight portion in between two curves in a Scurve should not be less than 500 ft.

#### British Ministry of Transport Recommendations:—

When speed is restricted to 30 m.p.h. curves should have radius of not less than 1000 ft. for through roads and 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 one ft. per traffic lane for curves between 1000/1500 ft. and  $1\frac{1}{2}$  ft. for radius of less than 1000 ft. Transition lengths on each side of the curve will be 100 to 150 ft. minimum.

At **Railway Crossings** the minimum curve should be of 200 feet radius in the case of main roads and 150 feet in the case of other metalled roads, measured to the centre of the road. The angle of crossing should be not less than 45 deg.

The following alignment principles 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.

#### **Extra Widths of Pavement for Curves of all Speeds (I.R.C.)**

Radius of curve	Up to 200	201 to 500	501 to 1000	1001 to 1500
Extra width of pavement in ft.	4	3	2	1

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 superelevation starts reducing.



Figures in the above table apply to pavements up to 22 ft. width. For pavements wider than 22 ft. reduce figures in table by width minus 22 ft. Negative values to be ignored. (A 24 ft. wide pavement with radius of 450 ft. will have 3—(24-22) or 1 ft.) For pavements narrower than standard widths (of 12 ft. or 22 ft.) increase figures in table by the width of pavement short of the standard width. (A 20 ft. wide pavement with radius of 450 ft. will have 3+(22-20) or 5 ft.) Widening may be equally distributed on the inner and outer sides or it may be on the inner side only, especially in the case of sharp curves with radius of curves less than 200 ft. (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. for roads designed for a speed of 50 miles and for radius flatter than 700 ft. for speeds of less than 30 miles.

### 23. SUPER-ELEVATION OR BANKING

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.

Theoretical formulae:—

$$(a) E = \frac{9}{16} \times \frac{V^2}{15R} = \frac{3V^2}{80R} \quad (b) E = 0.067 \frac{V^2}{R} = \frac{V^2}{15R}$$

$$(c) E = \frac{V^2}{64R} \quad \text{Subject to a maximum of } 0.067 \text{ or } 1/15.$$

Formula (b) is based on the assumption that the road is covered with ice or greasy mud and there is no tyre grip.

$E$  = super elevation in ft. per ft. width of road surface;  
 $V$  = design speed in m.p.h. (may be taken  $\frac{3}{4}$ th of the road speed);

$V$  = design speed in ft. per sec.;  $R$  = radius of curve in ft.

## British Ministry of Transport Recommendations:—

Roads subject to 30 m.p.h. speed	Roads not subject to speed limit
1 in 40 on curves of 800 ft. and over	1 in 15 on curves of less than 1200 ft.
1 in 30 " " 600 ft.	1 in 17 " " " " " 1400 ft.
1 in 25 " " 500 ft.	1 in 19 " " " " " 1600 ft.
1 in 16 " " 300 ft.	1 in 22 " " " " " 1800 ft.
	1 in 24 " " " " " 2000 ft.
	1 in 30 " " " " " 2500 ft.
	1 in 36 " " " " " 3000 ft.
	1 in 38 " " " " " 4000 ft.
	1 in 40 " " " " " 5000 ft.

## I. R. C. Recommendations:—(per foot width)

Radius of Curve in ft.	speed in m. p. h.			
	20	30	40	50
150	0.067			
200	0.067			
250	0.060			
300	0.050	0.067		
350	0.043			
400	0.038			
450	0.033			
500	0.030	0.067	0.067	
600	0.025	0.056		
700	0.021	0.048		
800	0.019	0.042		
900	0.017	0.037	0.067	
1000	0.015	0.034	0.060	
1100		0.031	0.056	
1200		0.028	0.050	
1300		0.026	0.044	
1400		0.024	0.043	0.067
1600			0.038	0.059
1800			0.033	0.052
2000			0.030	0.047
2500			0.024	0.037
3000			0.020	0.031

Super-elevation where provided shall not be less than the camber of the road (appropriate to the surface) to facilitate drainage.

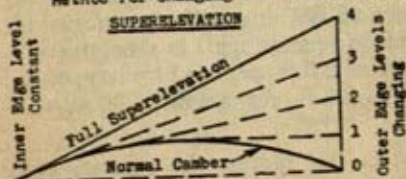


A super-elevation of 1 in 15 should not generally be exceeded ; a max. up to 1 in 10 can be given on very sharp-curves. The greater the super-elevation, more the inconvenience to slow-moving traffic with danger of side slip. For regard to the requirement of the standing vehicles, super-elevation of the curve should be not more than 1 in 16. If ice and snow conditions prevail, the maximum should be reduced to 0.08 ft. per ft.

**Radius in feet beyond which no Super-elevation is necessary (I. R. C.)**

Design speed m. p. h.	Water-bound macadam		Black top	Cement concrete
	Camber 1 in—			
	36	48	60	72
15	300	400	500	600
20	550	700	900	1000
25	850	1100	1400	1700
30	1200	1600	2000	2450
40	2150	2900	3600	4300
50	3400	4500	5600	6750
60	4850	6500	7500	7500

Method For Changing From Normal CAMBER to



There are in general two methods of applying superlevation : (a) Outer edge super-elevated and inner edge depressed, (b) Grade at inner

edge retained and outer edge super-elevated.

### Transition (Easement or Spiral) Curves

A curve of progressively decreasing (or increasing) radius used in joining a tangent with a simple circular curve or in joining two circular curves of different radii. It is a curve of varying radius.

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 connect-

ing 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.

Forms of transition curves are : (i) cubic parabola, (ii) spiral, (iii) lemniscate.

*Length* required of a transition depends on the "comfortable rate of turning" and is governed by the principle that a vehicle travelling at a given speed requires definite time rate of application of centrifugal force up to the maximum, and a similar time rate when the centrifugal time is eased off from a maximum to nothing. The comfortable rate of change of acceleration (centrifugal) is recommended by the I.R.C. to be taken as 2.5 ft. per sec.<sup>3</sup> up to speeds of 20 m.p.h. and it would be 2.0 ft. for speeds of 30 m.p.h. and above.

Length also depends on the radius, the sharper the radius the longer the transition length. The length of the curve should be the same as the "run" of the super-elevation.

$$L = \frac{1.6V^3}{R}$$

L is length of transition,

V is speed in m.p.h.,

R is radius of circular curve in ft.

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.

*British Ministry of Transport Recommends*—"Curves above 3,000 ft. radius require no transition spiral for roads



subject to 30 miles speed and curves above 5,000 ft. for roads not subject to speed limit. If the total length of the curve does not permit a transition approach, the curve should be made transitional throughout."

In U.S.A. a minimum transition length of 150 ft. for roads of 30 miles speed and above is usually recommended. Easement curves are usually omitted from 2-deg. and flatter circular curves.

### Minimum Transition Lengths for Various Speeds and Curve Radii (I.R.C.)

Plain & Rolling Country					Hill Tracts				
Radius in ft.	Min. Transition lengths in ft. for speeds in m.p.h.				Radius in ft.	Min. Transition lengths in ft. for speeds in m.p.h.			
	20	30	40	50		15	20	25	30
above					above				
3000	50	50	75	100	500	50	50	50	100
2000	"	"	"	150	450	"	"	"	"
1300	"	"	100	200					
1100	"	"	"	250	400	"	"	"	
					350	"	"	"	100
1000	"	75	150	"	300	"	"	100	150
900	"	"	"	300	250	"	"	"	
800	"	100	"	"	200	"	50	"	
700	"	"	200						
600	"	150	"		150	100	100		
					100	100			
500	75	"	250						
400	"	200							
300	100	250							
250	150								
200	200								
150	250								

Adopt greatest possible radius opposite firm stepped line. In extreme cases figures between firm and thin lines may be adopted.

## 24. 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. 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 35 ft. which is sufficient to enable even the largest vehicles to keep close to the kerb when turning. The additional space for manoeuvring 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 8 ft. in a length of 200 ft.

Fig. (a)

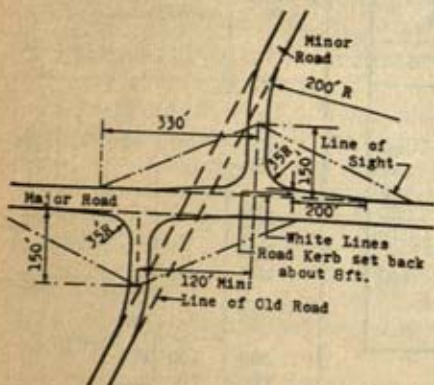
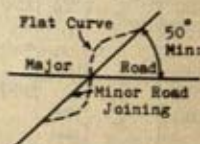


Fig. (b)



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 Fig. (a). When the side road is important but not sufficiently so as to justify a "roundabout", the "baffle" junction may be used by making separate turning lanes (with an island in between the two) for the side road.

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.

### 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  $50^{\circ}$  min : with  $30^{\circ}$  absolute min : ; prefer not less than  $60^{\circ}$ . It has been established that all things considered, the intersection of traffic streams at about right angles ( $75^{\circ}$  to  $106^{\circ}$ ) is most favourable.

Fig. (b) shows 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  $90^{\circ}$ . Min : turning radii at junctions for different intersecting angles are given in the following table.

**Minimum Turning Radii at Junctions**  
Turning Speed 20 m.p.h.

Angle of Junction	Radius for Cars up to 20 ft. length	Radius for Bus or Truck
90-deg.	30 ft.—35 ft.	50 ft.—55 ft.
105-deg.	35 ft.	60 ft.
120-deg.	45 ft.	70 ft.
135-deg.	60 ft.	90 ft.
150-deg.	120 ft.	150 ft.

### Camber or Cross-fall at Junctions

The camber of the side road should not be carried into the major road, the crown of the side road 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 bone, and the necessary changes of cross-section for marrying in with the crown of the intersecting road should be as gradual and sweet as possible.

## T-JUNCTIONS

### Visibility at Corners, Bends and Junctions

It is of first importance in the interest of road safety that drivers should have full view of approaching vehicles at a T-junction. The following illustrations 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 15 to 20 m.p.h. This is formed by the buildings at the corners splayed to a line joining points 15 ft. back each way from the junction of the building lines. The foot-paths on both the roads are considered to have 15 ft. of width. Where footways are narrower the splay should be increased proportionately. If modification is desirable due to aesthetic reasons the adjustment should be made by lengthening one of the sight lines.

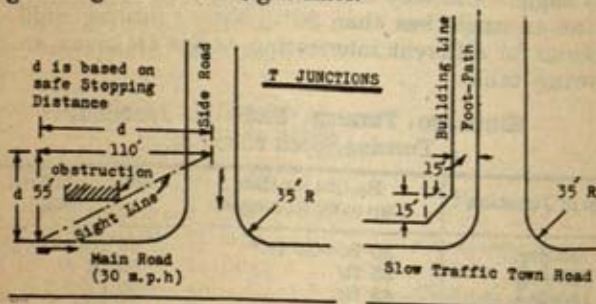


Fig. (c)

Fig. (d)

Fig. (c) shows the sight distance required when the driver on the through road of the T maintains a speed of 30 m.p.h. 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 330 ft. and on the minor road 150 ft. to enable the drivers approaching from a minor road to have view of the major road (Fig. (a)). No obstruction should exist between the sight line and the road kerb. (Based on the British Ministry of Transport recommendations.)



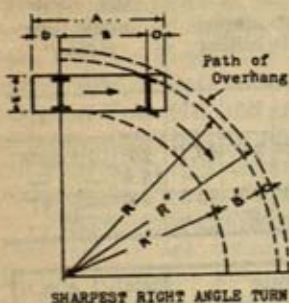
The following sight distances at road junctions are recommended for Indian conditions, which are taken on the centre lines of the roads :—

Speed in m.p.h.	15	20	25	30	35	40	50
Sight distance	60	90	130	160	210	250	360

(This is different from the safe stopping distance given earlier.)

T-junctions with carriageway of more than 55 ft. width with another of similar size should preferably be controlled by a rotary.

### Turning Circles:



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 30 ft. radius, preferably 35 ft., most of the vehicles can manipulate easily; and which should be 50 to 55 ft. 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 15 ft. should be regarded as absolute minimum and it must have road width of at least 15 ft. The vehicles can turn only with very slow speeds. Where the angle is more than 90-deg. greater radius will be required, as given in a previous table.

When the kerb radius is less than 15 ft. 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.

Some American engineers recommend for important 90-deg. inter-sections which have to accommodate large

trucks, a three-centered compound curve of minimum radii successively of 120, 38 and 120 ft. With this, minimum lane width near the centre of the curve will approach 20 ft. for turning large trucks.

Certain design features of complicated intersections can be tested by actual use. Temporary channelizing islands can be made of sand bags which can be easily shifted around. Sand sprinkled over the carriage-ways will indicate vehicle paths.

#### Minimum Turning Radii for Different Classes of Vehicles

Type of vehicle	Length	Width	Back side of wheel	Front side of wheel	Path width	Overhang	Min. desirable radius			Min. right angle turn radius		
	A	E	D	O	B'	C	R	R'	R''	R	R'	R''
Car	19'	6'	4'	3'	8.7'	1.3'	38.7'	30'	40.7'	28'	19.3'	29'5"
Bus or Truck	9.8'	4.2'	—	—	—	—	—	—	—	15.8'	—	—
	30'	8'	6'	4'	12.7'	2'	61.3'	50'	63.3'	45'	32.3'	47'
	35'	8'	—	—	—	—	—	55'	—	—	37.5'	—

#### Sizes of Motor Vehicles : (Cars)

The dimensions are overall inclusive of extreme projecting points.

Size	Max.	Min.
Length ...	20'	9'-10"
Width ..	6'-11"	4'-2"
Height ..	7'-0"	5'-0"

#### ROUNDBABOUTS 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. The British Ministry of Transport recommend the provision of a rotary at the crossing of a light trafficked and a heavily trafficked road at 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. I.R.C. recommend a rotary where the intersecting motor traffic is about 50 per cent of the total motor traffic on all the intersecting roads or where the fast traffic turning sight



forms at least 30 per cent of the total. 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 for a traffic density of over 400 vehicles.

For conditions in India I.R.C. assume a bullock cart equivalent to 3 passenger cars on level terrain and to 6 in rolling country; trucks being counted equal to 2 and 4 respectively and a cycle is counted equal to one car. (Trucks with dual back wheels are commercial vehicles.)

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.

**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 50 ft. 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. The recommended minimum design speeds around rotaries for Indian conditions are 20 and 25 m.p.h.

Vehicles can travel round at a speed of 15 m.p.h. of a 100 ft. diameter island and at 25 m.p.h. round a 150 ft. diameter island where super-elevation has been provided. Size of the central island should not be less than 100 ft. diameter although islands have been built with 60 ft. diameter. A 60 ft. diameter island has been observed to take 2,500 vehicles (including cycles) per hour, a 75 ft. diameter island 3,000 vehicles per hour and a 100 ft. diameter island 3,500 vehicles per hour. (Not in India).

Width of the carriageway round the island (rotary roadway) 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, whichever is more. A min : width of 24 ft. is recommended in U.S.A. and 30 ft. in U.K. 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.

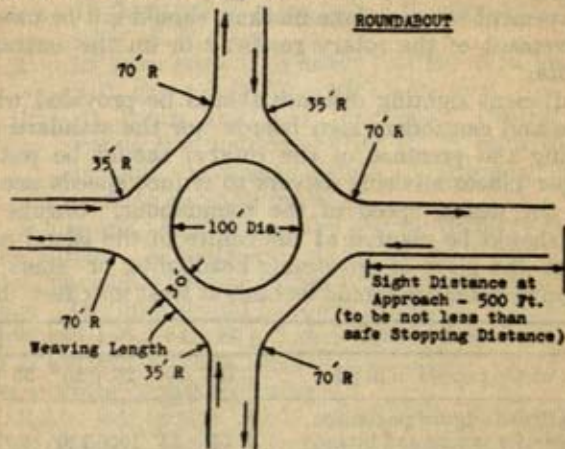
The width of carriageway of the radial road at the entrance and the exit of the rotary, i.e., the width between the channelizing island and the kerb should be increased. Under Indian conditions of mixed traffic where slow moving traffic will cluster at the approaches to a busy intersection, it is desirable to have a minimum width of carriageway equivalent to two lanes at the entry and exit of a rotary. Where the rotary diameter and the width of carriageway according to the table given at page 18/182 does not provide for adequate weaving length, the diameter of the rotary should be increased.

Adequate and proper super-elevation can rarely be given on rotary roads, 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.

**Radii of Curves at Entrance and Exit.** The British Ministry of Transport recommends a max: radius of kerb at entry of 35 ft. and at exit 70 ft. in built-up areas, and 60 ft. and 150 ft. in areas outside towns. I.R.C. recommend the radius of the entry curve equal to the min: radius of the central island recommended for the design speed and the radius of the exit curve  $1\frac{1}{2}$  times the radius of entry with a min: of 100 ft. for design speed of 20 m.p.h. and 150 ft. for 25 m.p.h.



### Channellizing 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. Channellizing islands are also used as pedestrian refuges and for erecting signs and lighting the roadway. All islands should be provided with highly visible rounded kerbs, 4 to 6 inches 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 7 to 9

inches high. By providing channelizing islands, vehicles are confined to definite 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** 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 curb 2 to 4 ft. 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 exits.

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 should be built at least four feet high.

Straight road speed in m.p.h.	20	25	30	40	50	60
Rotary turning speed in m.p.h.	15	20	25	30	35	40
Min. radius of edge of pavement or kerb (for trucks and buses)	55'	55'	100'	130'	180'	250'
Min. diameter of central island: without super-elevation with 1/16 super-elevation	100' 100'	140' 120'	210' 180'	280' 250'	480' 380'	640' 540'
Width of carriageway around island	22 to 30 ft.					
Weaving length	Min. Max.	90' 200'	120' 300'	150' 400'	180' 500'	210' 600'

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 the 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 of roads approaching or within a rotary should in no case exceed 1 in 20.

## TRAFFIC MARKINGS AND ROAD SIGNS

### White Lines—Traffic-Lane Marking

The lines mark the divisions of carriageway lanes and are made 4 to 5 in. wide (3 ins. min.; 4 ins. common) longitudinal strips and 6 in. wide transverse strips set out in accordance with the following schedule :

- (a) On straight lengths of highways: 3 ft. lines with 15 ft. gap; 9 ft. lines with 9 ft. gap ; or 5 ft. lines with 10 ft. gaps.
- (b) In town areas with heavy traffic : 3 ft. lines with 3 ft. gap ; 3 ft. lines with 9 ft. gap.
- (c) At bends and near junctions : Central continuous line extending 100 ft. 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 18 ins. 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 I.R.C. Certain general principles are

recognized internationally. Signs giving warning of danger incorporate a red triangle, this surmounts a rectangular plate on which symbols and words indicating the danger are given. The red triangle is  $18'' \times 18'' \times 18''$  outside edges with  $3''$  red border and the centre white or hollow. The rectangular symbol plate is  $18'' \times 15''$  to  $12'' \times 10''$  size, preferably with a red border. The space between the red triangle and the symbol plate is about  $6''$ . Prohibitory signs include a red disk of 2 ft. diameter, surmounting a rectangular plate on which the subject of the prohibition is indicated, or a red ring surrounding a disk on which similar indication is given. Mandatory signs include a red ring surmounting a rectangular plate. Informatory signs are rectangular plates with no red colour.

All road signs should be erected at a distance not greater than 10 ft. and not less than 6 ft. 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 edge of the pavement (face of kerb) shall be not less than 12 inches. The height of the lower edge of the signs shall be 7 ft. 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 3 ft.-6 ins. and in towns 6 ft.-9 ins. above road level.  $3'' \times 3'' \times \frac{3}{8}''$  T-iron may be used for the post fixed about 2 ft. inside ground. Vertical posts should be painted in 9-in. black and white bands, commencing black from bottom.

All warning or information signs should be fixed at 400 ft. on nearly level roads, 500 ft. on steep down grades, and 250 ft. 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 approaching traffic. Warning signs should be placed, as far as possible, on a straight section of the road (at least 75 ft. visibility is necessary) and should be clear of bushes, trees or other obstructions to visibility so that the full length of the supporting post is seen. (Route marker-signs for National Highways are given in I.R.C. Journal vol. XVIII—2, Dec. 1953).



**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 30 ft. centres on straight roads midway between the dashes of the interrupted white lines, and at 12 ft. centres on bends which have the continuous white line. On vertical planes the studs are fixed 3 ft.-6 ins. above road level.

All reflectorized signs shall 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 approximately perpendicular to and in line with the headlines of approaching vehicles.

### **Mile and Furlong Stones**

Type designs for highway milestones are given in I.R.C. Journal vol. XVIII-1, Dec. 1953 and for furlong and boundary stones in vol. XVII-1, Dec. 1952.

Mile and furlong stones may be made of dressed hard stone or of lightly reinforced concrete. Mile and furlong stones shall be located on the left-hand side of the road as one proceeds from the terminal station from which the mileage count starts. The faces of the milestone on the main roads shall make an angle of 60 deg. with the centre line of the road. Furlong stones may be fixed with their faces either parallel or at right angles to the centre line of the road, in which cases both the faces will bear the (same) inscription. The stones shall be fixed outside the edge of the formation (or shoulder) on specially erected platforms, protected by revetment, if necessary. In cutting they shall be fixed clear of the shoulder, set 5 ft. from the metal edge in specially cut niches 6 ft. long. The stones should be fixed 2 ft. below ground level in compacted hard soil. In loose and damp soils the stones should be encased with 6" of lean concrete on all sides below ground level.

The size of the milestone slabs prescribed by I.R.C. is 2'-9" wide by 2'-11" high, and of furlong stones 1'-3" total height with top rounded, by 9" wide and 4" thick,

above ground level. On furlong stones, the number indicating the mile should be inscribed above the number indicating the furlong, and the inscription should denote its distance from the starting point of the road, viz.,  $\frac{2}{3}$  indicates 5th furlong stone of the first mile;  $\frac{4}{3}$  indicates the end of 6th furlong in 5th mile, or, in other words, the furlong stone is situated 4 miles and 6 furlongs from the starting point of the road. Prefer to have all letters and figures cut into the stone with a V shaped incision which greatly facilitates neat repainting and prevent errors; depth of cutting is  $\frac{3}{8}$ ". The stones shall be painted with a background of light yellow and the letters in black.

### STREET LIGHTING

Lighting from one side of the road only will be unsatisfactory except on bends. Maximum distance between the two rows of light sources should not exceed 30 ft. if a dark centre to the road is to be avoided. Therefore, roads up to 30 ft. wide can be effectively lighted by lights mounted vertically above the kerb-line at each side. For road widths up to 40 ft., brackets should be used to overhang over the roadway to reduce the distance between rows of light to 30 ft. Roads more than 40 ft. wide 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 6 ft. in order to avoid unduly dark patches on the kerbs and footways. Centre line of the lamp posts should preferably be 2 ft. inside from the kerb face. Side-mounted lamp posts may be arranged either opposite to each other or staggered. A staggered arrangement is generally preferred. Usually suitable mounting height of lamp posts is 20 to 25 ft. in streets (with 15 ft. min.) and 25 to 30 ft. in highways, with 120 to 150 ft. 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.



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. Single-side mounting should be employed at long curves having a radius of curvature of less than 2,000 ft.

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, 60 ft., a single light of the cut-off type having symmetrical distribution, and mounted centrally at a height of 30 to 35 ft. 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 100 ft. 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-coming traffic lane). An intersection of two roads will require four lamps, one not more than 40 ft. along each road on the left-hand side, and the other not more than 120 ft. from the crossing.

#### **Parking Places and Lay-bys**

For parking at open places a gross area of about 250 sq. ft. for each car may be taken. (This makes due allowance for access roads, irregularities in manoeuvring and opening of doors). For parking parallel to the kerb, 8 ft. of additional pavement on each side should be provided,

and for parking either diagonally or at right angles to the kerb an extra width of at least 18 ft. is needed.

On important roads lay-bys should be provided at intervals to enable vehicles to draw off the road for temporary parking or for repairs. Bays should also be provided for bus stops. A size of about  $100' \times 8'$  is adequate for lay-bys.

## 25. ROAD EMBANKMENTS

(Embankments have been described in detail in the Section on "Irrigation".)

### **Borrow Pits for Highway Embankments**

Borrow pits should be rectangular in shape with their long sides parallel to the road, and no borrow pit should be dug within 15 ft. of the toe of the final section of the road (of a bank). Borrow pits should not be continuous unless intended to be used as side or catch-drains, and should be of regular shape, preferably of equal sizes in multiples of 10 ft. to facilitate measurements, and not more than 2 ft. deep.

Where the road embankment acts as a flood bank, the finished road level should be at least 18" above the highest flood level. Allow a monsoon to pass over a road bank and compact it again before metalling. The downstream slope of the bank should have sufficient cover over the saturation line as explained under "Irrigation". All earth for the embankments should be borrowed, as far as possible, on the river side. The inner edge of any borrow pit should be 50 to 100 ft. away from the toe of the bank, the distance depending upon the magnitude and height of the flood to be withstood. Ordinarily no borrow pits should be dug on the land side, where this is not possible a berm at least 80 ft. wide should be left between the borrow pit and the toe of the bank.

All *tatties* or *mutams* (dead men) must be removed as soon as the work has been measured up and checked for payment, and the earth used up in the embankments.

### **Outer Slopes of Embankments**

Side slopes should be as gentle as possible. Slopes steeper than 2 : 1 (hor. : ver.) should never be used even though structurally stable. Provide 4 : 1 if the embank-



ment is up to 2 ft. height to enable motors getting down in case of emergency, and natural slope of the soil if over 2 ft. height. Some engineers recommend 4 : 1 if under 6 ft. high and 2 : 1 if over 6 ft. high. Any slope steeper than 3 : 1 is not desirable. Provide guard rails where embankment is 6 ft. high or higher. Desirable shoulder slope has been given under "camber or cross fall". Top and bottom of the slopes of banks and cuttings should be rounded off.

## 26. 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".

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 perhaps, the cheapest. A bottom width less than 4 ft. is difficult to construct and clean out. The depth should vary from a minimum of about 1 ft. below the edge of shoulder in regions of low rainfall intensities to a minimum of  $1\frac{1}{2}$  ft. in districts of high rainfall. The bottom should be rounded since a rounded section is less conducive 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 150 to 300 ft. and each gully is required to drain an area of about 2500 sq. ft. This spacing will be all right for a road about 15 ft. 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 and 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 2 ft. 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 8" wide and  $1\frac{1}{2}$ " deep towards the kerb to give a cross slope. If the road slab is of concrete, a longitudinal joint should be made between the pavement slab and the channel slab.

**Underground Ducts.** All ducts should be laid with a slope of 3" per 100 ft. to drain out any water.

**Telegraph, Telephone or Electric Poles etc.** should be fixed normally at least 5 ft. outside the existing road edge and if possible, at the road boundaries.

## 27. 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 sec-



tion 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 work should be followed by trial survey and finally be the permanent detailed survey. 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. Ten to fifteen miles can be covered in a day in the plains and two to four miles 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, locations 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-sections 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 500 to 1000 ft. 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 150 to 300 ft., 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-sections should be taken at about 100 ft. 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 100 ft. on straights and every 50 ft. or less on curves. All tangent points are pegged and reference pegs placed apposite all tangent points and not more than 200 ft. to 300 ft. 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, 100 ft. 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 4 miles per day in the plains (adding extra time for laying out curves) and only about 10 miles per 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; rainfalls 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, mile and furlong 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, mile and furlong stones, sign posts, siding and parking places.

(b) Bridges, culverts, causeways, rest houses.

(c) Arboriculture, nurseries, wells.

(d) Acquisition of land—estimates of costs 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 3 miles and the existing means of communications (rail, road, waterways) in the neighbourhood.

Scales : 1 inch to 1 mile

$\frac{1}{2}$  " " 1 mile

$\frac{1}{4}$  " " 1 mile

$\frac{1}{16}$  " " 1 mile

*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 50 ft. or 100 ft. intervals. Geological survey maps may be obtained. Try to get maps with 4 inches to a mile. 1 inch to a mile maps generally available are too small for above details and contours. (For small works only one of the above maps need be made.)

*Preliminary survey and location or alignment plans*: Are drawn to a scale of 8 inches to 1 mile or 16 inches to 1 mile, 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

Vertical for L or cross-sections

220 ft.=1 in.

10 ft.=1 in.



Close or hilly country :

Horizontal for plan and L—section	100 ft. = 1 in.
Vertical for L or cross section	20 ft. = 1 in.

(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.

(The above are based generally on the instructions issued by the Consulting Engineer (Roads) to Government of India.)

Initial costs of different types of roads taking 100 for water-bound macadam per unit of surface area :

	Water-bound macadam	1½" asphalt macadam	4½" cement concrete
Cost	100	230	420
Life	3 to 4 years	10 to 12 years	30 years

### Repairs and Maintenance

The following figures may be taken for approximate estimating of different road surfaces under medium conditions of traffic :

#### *Water-bound macadam :*

	per unit of surface area		
Taking initial cost ..	..	..	100.0
Routine annual maintenance ..	..	..	4.9
Renewal cost every 3 or 4 years (or earlier) ..	..	..	52.0

**1½" Asphalt macadam :**

Taking initial cost	..	..	100.0
Routine annual maintenance	..	..	2.3
Seal coat every 4th year		..	24.0
Renewal coat every 12th year	..	..	85.0

**4½" Cement concrete :**

Taking initial cost	..	..	100.0
Annual maintenance	..	..	00.4

It is considered that about 60 to 70 per cent of the repair grant should be available for re-surfacing.

It has been estimated that the total saving on the cost of transportation is about 12 nP. per vehicle mile from bad to good roads or from *Katcha* to average roads.

For bullock-cart traffic the saving has been estimated at 50 nP. per ton mile.

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## SECTION 19

### HIGHWAY BRIDGES & CULVERTS

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*(The standards given are generally in accordance with the recommendations of the Indian Roads Congress or other recognized practices in England and America.)*

## 1. APPROACHES TO BRIDGES

The approach on either side of a bridge should have a minimum straight length of 50 ft. and for a culvert 20 ft., 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 30 ft. 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 25 ft. (prefer 30 ft.) for arterial and district roads, which may be reduced to 15 ft. 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 30 ft. 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 over-topped by floods.

## 2. PARAPETS AND HAND-RAILS

Parapets or hand-rails must be protected by wheel-guards or kerbs. Hand-rails may be of L-irons 5 ft. to 8 ft. apart with G.I. piping of diameter  $1\frac{1}{2}$ " to 2". A  $4'' \times 3'' \times 5/16''$  L-iron will suit for 3 ft. high post 8 ft. apart. A masonry parapet can be  $13\frac{1}{2}$ " thick.

Railings may be made of the following section :

Verticals : L— $2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{3}{8}''$  @ 5.9 lbs., 5 ft. apart. Curved to 4.5 ft. radius for the top 2.25 ft. where desired.

Horizontal top : L— $3'' \times 3'' \times \frac{1}{4}''$  @ 4.89 lbs., with two flat irons  $1\frac{1}{2}'' \times 1\frac{1}{2}''$  @ 2.55 lbs.

Roadway and footway railings or parapets of high level bridges shall have a minimum height of 3 ft.-3 ins.

less one half the horizontal width of the top rail, or the top of the parapet.

The clear distance between the lower rail and the top of the kerb shall not exceed 6 ins. unless the space is filled by vertical or inclined members. (This is for safety of the users.)

For culverts the parapets should have a minimum height above the road surface of one foot, and be 9 ins. thick.

Parapets or other fencings should be provided on each approach of a bridge and on both edges of all banks over 8 ft. high. *Guard-rails* should be provided where embankment is 6 ft. high or more. *Fenders* consisting of stout posts or rails (or of stone or cement concrete) well-embedded in the ground should be provided at both ends of parapets; size—2½ ft. long, about 15 inches above ground.

On hill roads parapets may be made of dry stone with tops in mortar, about 2 ft. high in small lengths, with gaps in between.

Hand-rails, parapets or posts shall be designed to resist a lateral horizontal force and a vertical force each of 100 lbs. per lineal foot applied simultaneously at the top of the railing or parapet.

### 3. KERBS OR WHEEL-GUARDS

Roadway kerbs should have a solid section not less than 9 ins. wide at the bottom, and not less than 9 ins. high above the adjacent road surface of which the lower 6 ins. of height should be vertical and the upper face edge rounded (sloped back). Safety kerbs to be not less than 2 ft. wide, and are designed as foot-paths.

### 4. FOOTWAYS ON BRIDGES, & PEDESTRIAN BRIDGES

Minimum width for footways or foot-paths on bridges is 4 ft. but prefer 6 ft. (I.R.C. recommend 5ft.). Live load for design may be 85 lbs. per sq. ft. up to 25 ft. span and 60 lbs. per sq. ft. for a span of 100 ft., intensity of load reducing uniformly, for all footways on bridges, pedestrian bridges and bridges used by animals. Some engineers recommend a live load of 100 lbs./sq. ft. for all pedestrian and animal bridges.



## 5. WIDTH OF ROADWAYS ON BRIDGES

Provide a width of 12 ft. (prefer 14 ft., and 10 ft. absolute minimum) for a single-lane roadway bridge from edge of kerb to edge of kerb (clear carriageway). For each additional traffic lane, increase road width by 10 ft. Width for footways and parapets will be extra.

Formation width for village road bridges is 20 ft. and for arterial and district roads 32 ft.

Minimum vertical clearance should be 16'-6" above crown of road surface, 14'-6" on edges for single-lane bridges and 12'-6" for double-lane bridges, allowing space for any end bracings on top.

## 6. 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
One sq. mile or less .. ..	500 ft.
From 1 to 5 sq. miles .. ..	1000 ft.
Over 5 sq. miles ... ..	$\frac{1}{2}$ mile to 1 mile

For non-meandering natural streams not wider than 100 ft. 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. (I.R.C. Code)

For large natural streams in alluvial beds and having undefined banks, the linear waterway should be determined from the designed discharge, using any rational formula or some other method as detailed in Sections 16 and 17.

**Waterway Areas Required for Catchment Areas**  
Based on the Dun Drainage Table

Catchment area in sq. miles	Area of waterway in sq. ft.				Catchment area in sq. miles	Area of waterway in sq. ft.			
	In hills		In plains			In hills		In plains	
	120%	100%	80%	50%		120%	100%	80%	50%
1.0	120	100	80	50	40	1620	1350	1080	675
1.5	180	150	120	75	50	1812	1510	1208	755
2.0	240	200	160	100	60	1980	1650	1320	825
2.5	300	250	200	125	70	2136	1780	1424	890
3.0	360	300	240	150	80	2280	1900	1520	950
3.5	419	349	279	175	90	2418	2015	1612	1008
4.0	466	388	310	194	100	2544	2120	1696	1060
4.5	509	424	339	212	120	2778	2315	1852	1158
5	546	455	364	228	140	3000	2500	2000	1250
6	611	509	407	255	160	3198	2665	2132	1333
7	667	556	445	278	180	3384	2820	2256	1410
8	721	601	481	301	200	3564	2970	2376	1485
9	769	641	513	321	250	3970	3308	2646	1654
10	815	679	543	340	300	4338	3615	2892	1808
12	888	740	592	370	400	4998	4165	3332	2083
14	966	805	644	403	500	5532	4610	3688	2305
16	1038	865	692	433	600	6036	5030	4024	2515
18	1104	920	736	460	700	6504	5420	4336	2710
20	1164	970	776	484	800	6969	5800	4640	2900
25	1296	1080	864	540	900	7296	6080	4864	3030
30	1416	1180	944	590	1000	7656	6380	5104	3190
35	1528	1273	1018	637	...				

Use the 120 per cent column in bare stone covered hills liable to heavy rain storms; 100 per cent column in hills covered with vegetation; 80 per cent column for plains close to the hills; 50 per cent. column for plains distant from hills. These are approximate percentage



values and may require modifications for special conditions, and from local experience with existing culverts and bridges. Choice of correct factor is very essential.

*Determination of Effective Width of Linear Waterway :*

Lacey's formula :

$$P_w = 2.67 \sqrt{Q}$$

(See under "Irrigation").

$P_w$  is wetted perimeter in feet which is nearly effective width of the waterway in straight reach of the river in regime conditions. This is generally taken as the width of channel at mid-depth. The co-efficient 2.67 is not constant but may vary from 2.5 to 3.5 according to the local conditions.  $Q$  is discharge in cusecs.

Lacey's formula gives the minimum width of waterway for undisturbed cross-section prior to the construction of piers. The nett water-way may be determined by deducting twice the width of the piers for the total waterway and for end piers half pier width plus a small margin.

It has been discussed under "Storage of Rainwater for Irrigation" and "Drainage" that there are a number of formulae and methods suggested by Ryves, Inglis, Dicken, Talbot, Kuichling, McMath, Lloyd-Davis, Elliot, Barkli-Zeiglar, etc., (both Indian and foreign) for the determination of flood drainage, which cannot be relied upon for all conditions and places which are so variable, therefore, have been omitted.

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 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 2 ft. in alluvial and deltaic regions, 3 to 4 ft. in trough regions and higher in steep reaches of rivers with boulders and rocky beds.

Two formulae, of Molesworth and Merriman, are sometimes used for determining the height of afflux, which give only approximate figures and are not very popular.

(See under "Rise of water caused by weirs" and "Discharge over broad-crested weirs" in "Hydraulics.")

#### **Free-board**

Is the difference between the designed high flood level, allowing for afflux if any, and the lowest part of the bridge structure. Freeboard for high level bridges should not be less than 2 ft.

For arched openings of high level bridges, the clearance below the crown of the intrados of the arch should not be less than one-tenth of the max. depth of water plus one-third of the rise of the arch intrados. Springing line should be above the high flood level in small bridges :

The following min. vertical clearance is according to the I.R.C. Bridge Code for flat openings :

Dis. cusecs.—Below 10	10 to 100	100 to 1000	1000 to 10,000
Ver. clearance—6 in.	1½ ft.	2 ft.	3 ft.

### **7. BRIDGE FOUNDATIONS**

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 from 0.1 to 5 ft./sec. and in floods from 15 to 30 ft./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 site of the bridge during or immediately after a flood. 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** below H.F.L. in alluvial streams in regime condition, on a straight and unobstructed part (where a bridge structure does not obstruct the flow), based on Lacey's theory :—(See also under "Irrigation.")

$$D = 0.473 \left( \frac{Q}{f} \right)^{\frac{1}{3}}$$

Depth of scour under abutments is taken  $= 1.5 \times D$  and under piers  $= 2 \times D$ . 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.

When the water-way of a regime channel is obstructed by the construction of a bridge (the linear waterway of which is kept less than the regime width of the stream), the normal scour depth under the bridge will be greater than the regime depth of the stream and can be found from the equation :

$$D' = D \left( \frac{W}{L} \right)^{0.61} \text{ or } D' = D \left( \frac{V_2}{V_0} \right)^{0.61} \text{ (Kennedy's Silt Theory)}$$

where :

$D$  = the hydraulic mean depth in ft. which is also approximately the depth to which the scour will take place in a normal channel (unobstructed). It is the depth of non-scouring flow below H.F.L. 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's formula. ( $D$  is normal scour depth).

$Q$  = total discharge in cusecs,

$f$  = Lacey's silt factor for the bed material,

$D'$  = normal scour depth under the bridge, below water level,

$W$  = the regime width of the stream,

$L$  = obstructed width of waterway under the bridge,

$V_s$  = av. velocity under the bridge (or obstruction),

$V_o$  = critical velocity.

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 banks .. ..	2.75 D

For bridges causing contractions, and where the section of the stream or river is not uniform and where there are main currents which scour channels in the bed :

$$D'' = D \left( \frac{W}{L} \right)^{1.56} \quad \text{or} \quad D'' = D \left( \frac{V_s}{V_o} \right)^{1.65}$$

$D''$  is the max. scour depth.

Depth of scour according to Sir Robert Gales :-

$$S = 1.67d + r$$

$d$  is the deepest observed scour below low water level and  $r$  is the flood rise.

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 Foundations

The depth of foundations is determined by consideration 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 little variation in the moisture content of the sub-soil.

In erodible strata if 'D' is the anticipated max. depth of scour below the designed highest flood level including that on account of possible concentration of flow, the minimum depth of foundation below H.F.L. should be taken =  $1.33 D$  (at the site). The minimum depth of foundations below the scour line should not be less than 6 ft. for piers and abutments with arches and 4 ft. for piers and abutments supporting other types of structure unless resting directly on rock where rock is met with at a lesser depth. For small bridges of moderate height on dry land sufficiently solid, a depth of 5 to 6 ft. below the bed level in non-erodible streams will be found sufficient. (See also under "Size of Abutments".)

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 in conjunction with curtain walls (as explained under Culvert Foundations) or sheet piling, etc. Foundations 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". 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 a soil generally increases with depth so that the safe load on a deep foundation is generally greater than that on a shallow foundation. If the depth of a stratum suitable for foundations 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.

As regards the depth of well foundations, according to Spring, a grip of 50 ft. is sufficient for a maximum scour of 70 to 140 ft. below the highest flood level. According to I.R.C., a grip equal to one-third of the maximum scour is required, while according to Gales a grip of 65 ft. should be provided.

The following safe loads on foundations are recommended by I.R.C. :—

Material	Lbs. per sq. ft.	
	Deep foundation	Shallow foundation
Confined quicksand	7,000	—
Other fine sand	9,000	6,000
Coarse sand and gravel	11,000	8,000
Mixed sand and clay	7,000	5,000
Alluvium and silt	5,000	3,000
Hard Clay	12,000	9,000
Hard pan	18,000	14,000
Rock	40,000	40,000

In hard beds, not erodible by the max. velocity, the foundations should be securely anchored about a foot or so into the rock and about two feet into other hard material, if met with.

Foundations for irrigation channels need not be carried as low as for natural streams or rivers since flow in such channels is controlled. But as the scour depth is likely to exceed the normal scour depth when there is a breach



in the channel, foundations should be carried beyond this scour.

### 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 4 to 5 ft.) 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 Foundations".) Bore holes are sunk to determine the nature of the sub-soil strata before well sinking.

*Well Curbs and Steening* have been described under "Water Supply".

The usual shape adopted for small wells up to 20 ft. diameter is a single circular or octagonal and for large wells a combination of two or more cylinders can be made. Twin octagon seems to have given the most satisfaction. They are connected at the top by a cap to give the required base area.

Load taken by well foundations depends upon the skin friction which can be roughly calculated from :

$C \times \text{height of well sunk} \times \text{outside circumference of the well masonry.}$

C is a co-efficient varying from 5 to 15 for sand and clay and 2 to 8 for mud and silt.

Much reliance cannot always be placed on skin friction for the reasons that : It is not possible to measure accurately depth which can contribute towards skin friction. The co-efficient of friction may be very substantially reduced by the vibrations of the super-structure in floods.

The thickness of the masonry shell or steening is about  $1\frac{1}{2}$  ft. to 3 ft. according to the diameter of the well, anti-

icipated skin friction and the span and load. Where high skin friction is anticipated and also 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 6 to 8 ft. diameter (prefer 8 ft. 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 3 to 4 ft. 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 3 ft. from top. The top portion is again filled with cement concrete and R. C. C. capping built to form a platform for the pier.

Where wells are less than 15 ft. 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 4 ft. 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 (2 ozs. or less), 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 10 ft. more than the outside diameter of the well to be sunk. It may be built of sand, clay and boulders mixed together. A mixture of clay and gravels in equal proportions known as "puddle" is quite suitable. The finished bund should be 2 ft. above water level with 3 to 5 ft. width at the top for small enclosures and shallow waters say up to 8 ft. deep, and 8 to 10 ft. above water level and 25 to 30 ft. 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 2 ft. per sec. and depth not exceeding 8 to 10 ft. A sheet coffer-dam is generally adopted if the depth of the water is more than 10 ft. 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 9"×3". 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 10 ft. 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. (See also 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 5 to 8 ft. 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.

In the case of timber piles the allowable unit stress (bending) on wet planks can be taken at 800 lbs. per sq. in. Height of water should be taken for the pressure on one side. Section modulus per ft. width is  $2d^2$ , where  $d$  is the thickness in inches; the max. B.M. can readily be found. 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 by 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 2" dia. to a depth of about 10 to 12 ft. The bottom 3 to 4 ft. 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 foundation 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 R.C.C. The caisson has sharp edges 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 R.C.C. 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 3 to 10 ft. without vertical joints and of 4 to 10 ft. diameter, with plates  $\frac{3}{8}$ " to 1" thickness with  $1\frac{1}{2}$ " to 2" 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 inches, to withstand the pressure of the water, of small thin cylinders may be found approximately from the formula :  $t = dh/1000$ , where  $d$  = dia. in ft.,  $h$  = depth of water in ft. This will stand a stress of 12000 lbs./sq. in. The thickness should not be less than  $\frac{1}{4}$ ". Steel caissons or cylinders should be braced internally to prevent collapsing while sinking. Cylinder piers are used for river beds requiring well foundations.

## 8. DESIGN OF ABUTMENTS AND WING WALLS

(Design of Abutments and Wing Walls has been described in detail in Section 7—"Masonry Structures", and Abutments for Arches under "Bridge Arches" in this Section.)

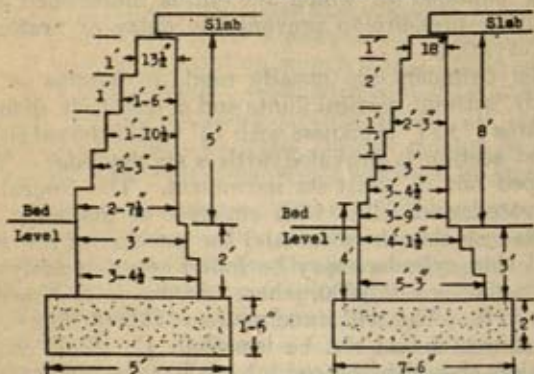
### Abutments

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.

Where rock is met with at a higher level than the normal recommended depth, the abutment should be heeled in the rock for a depth of 6 to 8 inches and if the rock is sloping the masonry should be anchored to the bed rock by means of dowel bars for about one foot.

ABUTMENTS FOR LIGHT LOADINGS



The length of abutments at the top shall 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 10 ft. lengths on either side of a bridge no live load surcharge need be considered (see page 19/20). 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. (See also 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	By "height of earth cushion" is meant the height between the top of abutment and the road level.  (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.")
5 ft.	4 ft.	
10 ft.	4 ft.	
16 ft.	3½ ft.	
20 ft.	3 ft.	

Culverts under high overfills should be either of the 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 1 ft. 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 should be provided in all wing walls at 8 ft. centres (see Section 7). 6-in. dia. drain pipes placed 6 ins. above normal water level are recomm-

ended to be placed in all abutments with porous back-fills.

### Size of Abutments

The top width of abutments may be taken 2 ft.-6 ins. for spans up to 10 ft. and 2 ft.-9 ins. for spans above 10 ft. for road slabs. This provides 1 ft.-6 ins. width for the backfill wall and 1 ft. or 1 ft.-3 ins. for the slab seating according to the span. 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

5'	6'	7'	8'	9'	10'	12'
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Width of abutment for 2 tons/sq. ft. max. pressure on soil

5'-3"	5'-9"	6'-4"	6'-11"	7'-9"	8'-10"	11'-0"
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Width of abutment for 4 tons/sq. ft. max. pressure on soil

4'-3"	4'-9"	5'-1"	5'-4"	5'-8"	6'-2"	7'-3"
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The above dimensions are for spans up to 10 ft., for spans above 10 ft., increase width by 3 ins. The front of the abutments may be made vertical or with a batter of 1:12. The bottom width will be 4 ft.-6 ins. more which provides for a projection of 9 ins. for concrete all round. Minimum depth below ground level in ordinary soils should be 6 ft. (4 ft.-6 ins. for masonry and 1 ft.-6 ins. for the concrete). This is for I.R.C. heavy loadings where no "approach slab" is to be provided; surcharge due to live load is based on the Bridge Code.

It is generally safe to increase the permissible bearing pressure at the rate of 1 cwt. per sq. ft. for each additional foot of depth below four feet. (See also under "Depth of foundations".)

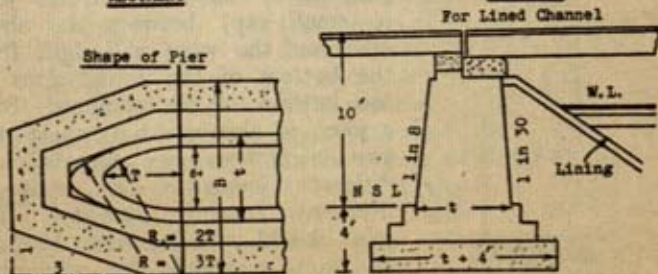
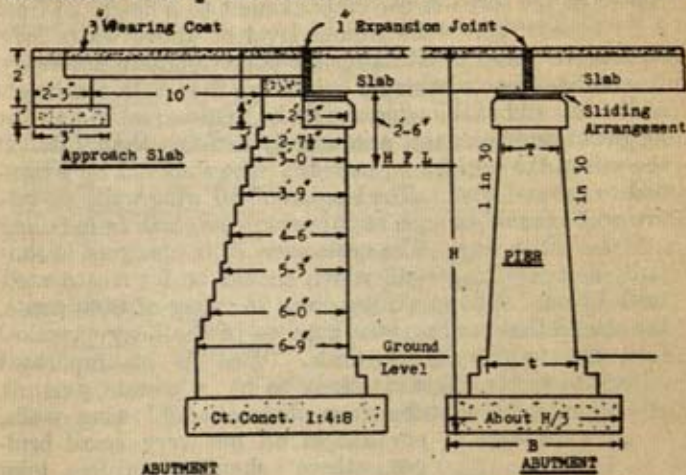
### Roadway Approach Slabs

If a concrete slab is provided as an approach on either side, no live load surcharge for 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 overall span of the bridge is equal to or less than 3 ft. provide 9" thick metalling in a length of 5 ft. normal to the abutment instead of R.C.C. approach slab. If the embankment is over 10 ft. high, roadway approach slabs should invariably be used, or width of abutment increased as tabulated above.

The approach slabs may be 10 ft. wide, 12 ins. thick, reinforced with main bars  $\frac{3}{4}$ " dia. at  $4\frac{1}{2}$ " c/c, distribution bars  $\frac{5}{8}$ " dia. at 11" c/c, for arterial roads. For light load village roads, the slabs may be 5 ft. wide, 8 ins. thick, reinforced with  $\frac{1}{2}$ " dia. main bars at 8" c/c. Every fourth



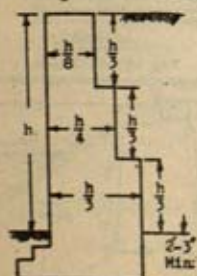
Sections Suitable For I.R.C. Class "A" Loadings

main bar to be bent up. Main bars are parallel to the road length.

The illustration shows size of abutment where an approach slab has been provided.

### Wing 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 opening 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 2 ft. to 4 ft. above the ground level or above water level to prevent water outflanking the culvert, where necessary. For embankments having a height less than 8 ft. the height of the low end of the wing should be made equal to half the height of embankment, and usually not less than 2 ft. 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,



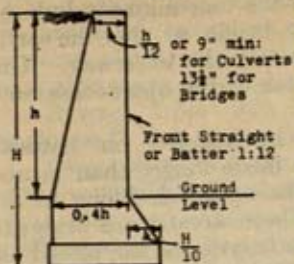
RETAINING WALL  
(Economical Section  
up to 10 height —  
used on U.K. Rlys.)

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.



Wing walls for culverts, say up to 10 ft. span,

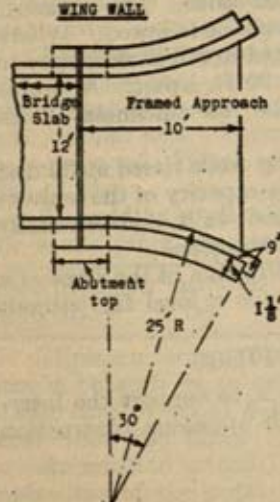
### WING WALL



generally have vertical faces with batter at the back of 1:4 and 9" min: broad at the top on the square. In the case of bridges, say 15 to 30 ft. span, the walls have a face batter of 1:12, a back batter of 1:6 built in steps and a top breadth of 18" (13 1/2" min:). Where the height of the wing wall is reduced towards its outer edge, the thickness is also reduced accordingly.

Wing walls may

### WING WALL



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 larger 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 located so 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 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 30 ft. 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.

## 9. 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 Pier

*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 smooth 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 down in the figure at page 19/21 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 Jan. of "American Soc. 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.

#### Size of Pier

Top widths of piers for I.R.C. standard heavy loadings, for spans other than arches:—

Span in ft.	10' and under	11' to 15'	16' to 20'	30' to 32.5'
Top width-T	1.9 ft.	2.3 ft.	2.7 ft.	3.1 ft.

Minimum top widths for simply supported spans should be such as to accommodate the bridge seat with a clearance of about 6 ins. 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  $\frac{1}{3}$ 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

(Extracts from B.S. Code of Practice C.P. III: 1948)

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. R.C. caps should be reinforced in two perpendicular directions, with shear reinforcements. Caps should be designed to spread the load from the bearings at an angle of 45 deg. until the compressive stress on horizontal planes is within that allowed in the pier shaft.

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. In the latter case, the load bearing part of the pier may be designed as thin as is practicable on structural grounds, the protective shell providing such an appearance of mass in the pier as may be desired for aesthetic reasons.

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 62.5 lbs. per cubic foot (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 axis of the bridge. The maximum unit pressure at the downstream toe and at the side should not exceed the allowable masonry and foundation pressure: (See also 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. For other conditions a fraction of the full buoyancy may be taken. To allow for full buoyancy a reduction 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 I.R.C. Class A and Class B loadings for spans 10 to 150 ft. :—

$$\frac{15}{20+L} \quad \text{for concrete bridges}$$

$$\frac{30}{45+L} \quad \text{for steel bridges}$$

Where  $L$  is the length of the span in ft. 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 10 ft.

For I.R.C. Class AA loading, impact may be taken 25 per cent for spans up to 40 ft. in the case of R.C. bridges and up to 75 ft. 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 is reduced on spandrel filled arches and solid floors of considerable depth. If there is an earth cushion over the culvert slab or arch crown, the impact effect is considerably reduced and will be about half of the above amount with 1 ft. of earth and is practically nil with about 2 ft. of earth filling. Therefore, for the design of culverts under heavy fills, impact factor need not be considered, but weight of the earth fill will be added to the dead load. Impact effect on a railway bridge is much more than on a road bridge

(f) **Traction Forces** producing longitudinal thrust due to braking 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 :—

For I.R.C. Class AA loading = 14 tons

For I.R.C. Class A loading =  $5 + 0.12 L$  in tons, where  $L$  is the span in ft. under consideration.

For I.R.C. Class B loading = 60% 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 bearings 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 assumed to act 4 ft. 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 :—

$$C = \frac{WV^2}{15R}$$

where:

$C$  = horizontal load due to centrifugal force in tons per r. ft. of the span under consideration ;  $W$  = total live



load in tons per r. ft. on that span with all traffic lanes loaded;  $V$ =max: design speed of the vehicles using the bridge, in miles per hour;  $R$ =radius of curvature in feet.

The centrifugal force is considered to act at a height of 4 ft. 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) **Wind Load** acting on the structure as seen in elevation, and on the moving load; uplift due to wind. See under "Wind Pressure" in Section 11. Wind has an overturning effect and is considered an horizontal force to act in any horizontal direction.

(i) **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  $\times$  its cross-sectional area. The following range of temperature are generally assumed in design:—

*Steel Structures:*

Moderate climates—from 0-deg. F. to 120 deg. F.

Extreme climates—from minus 30-deg. F. to 120-deg. F.

*Concrete Structures:*

Moderate climates—30-deg. F. rise and 30-deg. F. fall.

Extreme climates—45-deg. F. rise and 45-deg. F. fall.

Provision should be made for expansion of 1 inch per 100 ft. length of exposed structures.

(j) Horizontal forces due to water currents on any part of the road bridge which may be submerged in running water (generally taken for depths of water 20 ft. or more). This can be determined from the following formula:—

$$P = KAV^2 \quad (\text{on piers parallel to the direction of water current})$$

where:

$P$ =total pressure in lbs. due to water current,

$K$ =co-efficient according to the shape of nose of the pier:—

$A$  = area in sq. ft. of the vertical projections of the exposed part (pier face),

$V$  = velocity of current in ft. per sec.

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  $60^\circ$ ,

0.90 ditto. with  $90^\circ$ ,

0.50 for piers 5 to 6 times longer than their breadth with triangular cut and ease waters subtending an angle of  $30^\circ$  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  $90^\circ$ ,

Trestle columns are treated as solid with value of  $K$  as 1.25,

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. 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. 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  $\frac{1}{3}$  of the distance measured from the top between the upper and the lower wetted limits of the surface under consideration.

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 per c. ft. 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).

$H$  = 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 = 32.2 ft./sec<sup>2</sup>.

(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.

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 unnecessary obstruction to the flow of the stream resulting in increased backwater upstream in times of flood. A skew span should 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 R.C. 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 R.C., 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 4 ft. apart. Pile bents are strong and stable and are suitable for deep streams with muddy bottoms. R.C.C. trestle bridges are suitable for spans of 12 to 20 ft.

**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 beams 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 transversely 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.

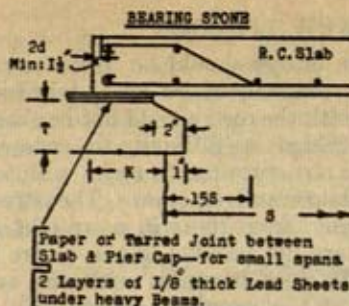
### Bearings

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 of 1 ft. for spans up to 10 ft. and 1 ft. 3 ins. for spans 12 to 20 ft. over abutments and piers for R. C. road slabs, is considered enough by some engineers, instead of the "K" shown in the illustration below. Tarred paper may be used under R. C. slabs of small spans. The forward edges of the abutments tops should be chamfered off to avoid the possibility of spalling.

**Sliding Plate Bearing :** Permits slight sliding of a girder end under expansion or contraction over the bearing plate. *Bearing plates* are satisfactory where the pressure





$K =$  Length of Bearing =  $0.5 + 0.04S$  Ft.  
or 14 inches  
whichever is less.  
 $\tau =$  Depth of Bearing Slab or Pier Cap  
=  $0.25 + 0.02S$  Ft.

is not more than about 400 lbs./sq. in. in bearing surface, and for spans not exceeding 45 ft. The bottom bearing plate should rest on a sheet lead plate of  $\frac{1}{8}$ " thickness. The lead plates also serve to equalize bearing pressures on the base when the span deflects under live loads and thus prevent high edge pressure. (This is generally for R.C. slabs.)

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 so that a line bearing is maintained under unbalanced loading.

For spans larger than about 45 ft., 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 60 ft., 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 3 to 10 ins diameter are used.

## 10. SKEW BRIDGES

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 not 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. (See also 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 reactions 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 obtuse angle corners at 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 obtuse 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 obtuse 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 obtuse angle should make an angle of 60 deg. with the axis of the channel and towards the acute angle should make an angle =  $\frac{1}{3}$ rd 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.

## 11. I.R.C. STANDARD LOADINGS

*Class "AA"* : 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-ton road roller passing one at a time over the bridge.

For details (for design) see the I.R.C. Bridge Code.

## REINFORCED CEMENT CONCRETE ROAD SLABS

*Dead weights of Concrete Bridges :*

- |                                  |  |
|----------------------------------|--|
| (ii) Solid slabs: $W = 100 + 9L$ | } in lbs./sq. ft. of floor<br>area of slab |
| (ii) T-beams $W = 100 + 5L$      |  |

$L$  is clear span in feet.

(Includes weight of wearing surface at 40 lbs./sq. ft.)

*Joint in R.C.C. 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 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 illustrations 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 bridges. 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 1 to 2 inches, 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) 2 to 3 ins. dia should be provided on both sides of the road through the parapets at intervals of say, 8 to 10 ft., 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  $1\frac{1}{2}$ " of earth as cushion over which  $4\frac{1}{2}$ " to 6" of metalling can be given duly surfaced.

(b) Asphaltic concrete about 2" 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 3" cement concrete can be given as a second coat on the bridge slab as explained under Cement Concrete Roads.

#### **Strengthening Existing R.C. Slab Bridges**

The strength of a 3" slab can be increased by adding more concrete on its top and bonding it with the old slab so that the two slabs act in conjunction.



## I. DATA FOR R.C.C. SLABS FOR ROAD BRIDGES

Freely Supported with 3" Wearing Coat  
for I.R.C. Heavy Loadings :

Clear span	B. M. per ft. width	Shear per ft. width	Total thickness of slab (excluding wearing coat)	Concrete cover below centre of steel	Reinforcement			
					Main		Distribution	
ft.	in. lbs.	lbs.	ins.	ins.	Dia. of bar	Spacing c. to c.	Dia. of bar	Spacing c. to c.
3	34,000	3,970	7	1½	½	3½	½	7
4	46,000	4,440	7½	"	½	4½	½	9½
5	56,000	4,520	8	"	½	4	½	9½
6	93,000	5,670	9½	"	½	3	½	8½
7	113,000	6,050	10	"	½	4½	½	7½
8	138,000	6,250	11	"	½	4	½	7½
9	159,000	6,590	11½	"	½	5½	½	7½
10	190,000	6,660	12½	"	½	4½	½	11
12	250,000	7,210	14½	"	½	4½	½	10½
14	320,000	7,630	16	1½	½	3½	½	10
15	357,000	7,860	17	"	1	6½	½	10
16	394,000	8,160	17½	"	1	6	½	9½
18	474,000	8,630	19½	"	1	5½	½	9½
20	567,000	9,260	21	"	1	5	½	9
22	662,000	9,760	22½	"	1	4½	½	8½
24	755,000	10,380	24	"	1	4½	½	12
25	830,000	10,750	25	"	1	4	½	12
27	961,000	11,520	26½	"	1	3½	½	11
30	1190,000	12,880	29½	"	1	3½	½	10

The above table may be used for small bridges on national and provincial highways where I.R.C. Class AA or Class A loadings are prescribed. The heaviest loading may come on the bridge only once in a while and which can be taken by the factor of safety.

*Notes on Road Slab Tables :—*

(i) The bending moment for design includes for impact, wind load stress, dead load due to weight of slab and wearing coat.

(ii) Design is worked out with stresses—18000, 750 and 15. Shear stress is within permissible limits therefore, no shear reinforcement is provided.

(iii) Longitudinal bars to be bent up 45° at a dis-

tance of 0.15S from the edge of abutment as shown in the illustration in Section 8.

No. of main bars cranked at each end :—

One longitudinal reinforcement bar should be bent up in every four up to 10 ft. span and in every two for spans above 10 ft.

(iv) For spans over 10 ft. a longitudinal camber of  $S/240$  may be given.

(v) It is generally economical to use single slabs instead of T beams for spans up to 30 ft.

(vi) If full length bars are not available and joints have to be provided, lap joints for 45 dia. length provided no joint comes at mid-span and all joints are well-staggered.

(vii) Distribution reinforcement given in the table applies to middle portion of slab. For slabs having spans 12 ft. and over, nominal surface reinforcements should be provided at top to counteract temperature effects.

(Based on I.R.C. Paper No. 167).

### 11. DATA FOR R.C.C. SLABS FOR ROAD BRIDGES

Freely Supported with 3" Wearing Coat  
for I.R.C. Class "B" Loading :

Clear span	Total thick- ness of slab	Concrete cover below centre of steel	Reinforcement			
			Main		Distribution	
			Dia. of bar	Spacing c to c	Dia. of bar	spacing c to c
ft.	in.	in.	in.	in.	in.	in.
2	5½	1½	½	3½	½	8
3	6	1½	½	3	½	7
4	6½	1½	½	5	½	7
5	7	1½	½	4½	½	10
6	7½	1½	½	4	½	9½
7	8	1½	½	5½	½	9
8	8½	1½	½	5	½	9
9	9½	1½	½	4½	½	8½
10	10	1½	½	6½	½	12
12	11	1½	½	5½	½	12
14	12½	1½	½	4½	½	11
15	13½	1½	½	4½	½	11
16	13½	1½	1	7½	½	10½
18	15½	1½	½	6½	½	10
20	16½	1½	½	9	½	10



### Rolled Steel Beams for Bridges with R.C.C. Decking for I.R.C. Class "A" Loading

Note :

Clear span	Size of R. S. Beam	
ft.	ins.	lbs.
15	15×6×45	
16—17	16×6×50	
18—19	18×6×55	
20—23	20×6½×55	
24—26	22×7×75	
27—30	24×7½×95	
35	22½×10×118	

- (a) R.S. Beams are 5'-6" apart c/c.  
 (b) *Details of R.C.C. Slab Decking:-*  
 Slab thickness 7½" with 3" wearing coat. Slab will be reinforced with ½" dia. bars at 4" c/c at right angles to the direction of the beams. Adequate reinforcement shall be provided at the top of slab over beams for pos. B.M. Alternate bars may be bent up. Longitudinal

bars (temp.) at bottom of slab ½" dia. at 8" c/c and at top ½" dia. at 12" c/c shall be provided.

(c) The decking slab will project 6" plus width of parapet on both sides of the end beams to make up for a 12 ft. traffic lane. The slab may project up to 2'-6" on the end beams to make up for greater road widths.

(d) Mild steel (rustless) bed plates ¾" thick, or two layers of ½" thick lead sheets, may be provided under beams.

(e) Provision must be allowed for expansion of girders at the rate of 1" per 100 ft. of span.

(f) ¾" dia. anchor bolts in slotted holes shall be provided on each girder end; 2 bolts on each end.

(g) Bottom of the R.C.C. slab either rests on the top of the beams or is made level with the bottom side of the top flange and the neg. B.M. bars rest on the beams.

(h) 2½"×2½"×¼" L spacer is fixed at the centre at right angles to the girders, in spans above 20 ft.

(i) Drainage holes should be provided on both sides of the road at 8 ft. intervals.

## 12. MASONRY ARCH BRIDGES

### Suitability of Arches for Bridges

Recent tests have shown that masonry and brick arches, with a good rise and ring thickness, proper maintenance, and good foundations are capable of carrying very heavy loads. A good deal of strength is derived

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 30 ft., 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.

### Rise/Span Ratio

Every endeavour should be made to keep the rise/span ratio as large as possible; temperature and shrinkage effect increase rapidly as the ratio falls below about  $\frac{1}{4}$ , and there is also an increase in the bending moment and thrust due to live and dead loads.  $\frac{1}{10}$  should be regarded as the lowest limit permissible; the most economical designs lie in the range  $\frac{1}{4}$  to  $\frac{1}{3}$ . For economy in material in the arch ring and the abutments, the rise of the arch should be between  $\frac{1}{4}$  and  $\frac{1}{3}$  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. Parabolic or segmental shapes usually give more economical designs than elliptic or 3-centred varieties; the segmental assumes generally the best appearance.

(See also under "Masonry Structures".)

### Thickness of Arch Rings

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 :

$$E = \frac{R}{5} + \frac{r}{10} + 2 \text{ ft. for } h \text{ less than } 1\frac{1}{2}L$$

For greater  $h$ , add  $h/5$  to the  $E$  obtained from the equation.

$$b = E + \frac{E.Z}{24r}$$

$$R = \frac{a^2 + r^2}{2r}$$



The 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  $\frac{1}{3} h$ .  $F$  may be equal to  $\frac{1}{2} G$ .  
(See illustration on the following page.)  
Take  $G = 1$  ft.-6 ins. and  $F = 9$  ins.

$R$  = radius of the arch intrados in feet,

$r$  = rise of the arch in feet,

$E$  = thickness of abutment at springing,

$b$  = thickness of abutment at depth " $Z$ " below springing,

$t$  = thickness of arch ring at crown in ft.,

$S$  = span in ft.

The abutments should be back-filled up to the springing before striking the centering.

### 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 top must be adequate to accommodate the skew backs on both sides.

Short piers may have vertical sides or batter of  $1$  in  $24$  to  $1$  in  $30$  and long piers, a batter of  $1$  in  $12$ .  $K$  should not be less than  $h/3 + 9"$  and at least  $1$  ft. more than  $P$ .  $J$  may be  $1'$  to  $2'-6"$ .

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.

*Spandrel* is the space extending from the top of a masonry arch to the top of the roadway.

$$D = \frac{r+t}{2} \quad C \text{ is earth cushion.}$$

Working stresses for Masonry Structures have been given in Section 7.

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  $\frac{1}{4}"$ .

## DATA FOR MASONRY ARCHES

Span in ft.	2	3	4	5	6	8
$t_1$	8"	9"	1'-0"	1'-0"	1'-0"	1'-0"
$t_2$	9"	9"	1'-1 $\frac{1}{2}$ "	1'-1 $\frac{1}{2}$ "	1'-1 $\frac{1}{2}$ "	1'-1 $\frac{1}{2}$ "
$t_3$	9"	9"	1'-1 $\frac{1}{2}$ "	1'-1 $\frac{1}{2}$ "	1'-1 $\frac{1}{2}$ "	1'-1 $\frac{1}{2}$ "
C	1'-6"	1'-6"	1'-6"	1'-6"	1'-6"	2'-0"
E <sub>1</sub>	2'-0"	2'-0"	2'-6"	2'-9"	3'-0"	3'-3"
E <sub>2</sub>	2'-3"	2'-3"	2'-7 $\frac{1}{2}$ "	3'-0"	3'-0"	3'-4 $\frac{1}{2}$ "
P <sub>1</sub>	1'-4"	1'-4"	1'-8"	1'-10"	2'-0"	2'-3"
P <sub>2</sub>	1'-6"	1'-6"	1'-10 $\frac{1}{2}$ "	2'-3"	2'-3"	2'-3"
P <sub>3</sub>	1'-8"	1'-8"	1'-10 $\frac{1}{2}$ "	2'-3"	2'-3"	2'-7 $\frac{1}{2}$ "
G <sub>1</sub>	1'-0"	1'-0"	1'-0"	1'-3"	1'-3"	1'-6"
G <sub>2</sub>	1'-6"	1'-6"	1'-6"	1'-9"	1'-9"	2'-0"

$t_1$  1st class dressed stone or cement concrete (1:2 $\frac{1}{2}$ :5) block masonry in cement mortar 1:3, arch ring.

$t_2$  1st class brick masonry in cement mortar 1 : 3, arch ring.

$t_3$  Ditto. Ditto. in lime mortar 1 : 2, each ring

E<sub>1</sub> 1st class stone masonry in cement mortar 1:3, abutment top.

E<sub>2</sub> 1st class brick masonry in cement mortar 1:3, abutment top.

P<sub>1</sub> 1st class stone masonry in cement mortar 1 : 3, pier top.

P<sub>2</sub> 1st class brick masonry in cement mortar 1 : 3, pier top.

P<sub>3</sub> 1st class brick masonry in lime mortar 1 : 2, pier top.

G<sub>1</sub> Cement concrete 1 : 3 : 6, depth of foundation.

G<sub>2</sub> Cement concrete 1 : 4 : 8 or good lime concrete, depth of foundation.

(Based on I.R.C. Paper No. 167).

Arches for I.R.C. Class "B" loading are given under "Masonry Arch Culverts".

The tops of all abutments and piers should be in cement mortar 1:4 for a depth of at least 2 ft.

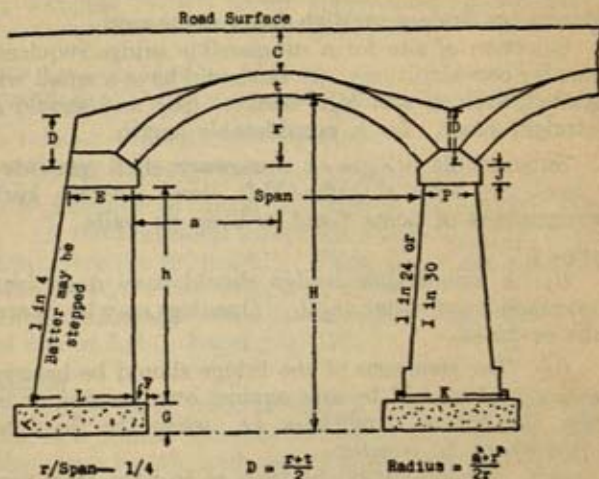
### Fillings over Arches and behind Abutments

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 one foot over the crown. Similar porous backfilling should be done at the back of the abutments. To safeguard against water penetrating and leaching, the entire extrados of the arch ring and the lower 6 inches 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 6 ins. above normal water level.



## I.R.C. STANDARD CLASS "A" LOADING

10	15	20	25	30	35	40
1'-0"	1'-3"	1'-6"	1'-8"	1'-9"	1'-10½"	2'-0"
1'-1½"	1'-6"	1'-10½"	1'-10½"	2'-3"	2'-3"	2'-7½"
1'-6"	1'-10½"	1'-10½"	2'-3"	2'-7½"	3'-0"	3'-4½"
2'-0"	2'-0"	2'-0"	2'-3"	2'-6"	2'-6"	2'-6"
3'-6"	4'-3"	5'-0"	5'-9"	6'-6"	7'-3"	8'-0"
3'-9"	4'-6"	5'-3"	6'-0"	6'-9"	7'-6"	8'-3"
2'-6"	2'-9"	3'-3"	3'-9"	4'-3"	5'-0"	5'-9"
2'-7½"	3'-0"	3'-4½"	4'-1½"	4'-6"	5'-3"	6'-0"
3'-0"	3'-4½"	3'-9"	4'-1½"	4'-6"	5'-3"	6'-0"
1'-6"	1'-9"	2'-0"	2'-6"	2'-6"	2'-6"	2'-6"
2'-0"	2'-6"	2'-9"	3'-0"	3'-3"	3'-6"	3'-9"



**Striking centres of arches.** The centres should not be struck before one week after the completion of the arch.

**Skew arches** shall be so constructed that the courses are everywhere at right angles to the lines of thrust.

### Conditions of Stability

**Abutments.** An arch abutment should be investigated for the following three conditions of loading—(i) Dead load plus live load on  $\frac{1}{2}$  span adjacent to the abutment. (ii) Dead load plus live load on the other  $\frac{1}{2}$  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.

**Strengthening Existing Masonry Arches :** Existing arches of small spans can be strengthened by additional rings at the bottom. Additional rings at the top of the existing rings do not add to the strength of the existing ring.

### 13. 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 (22 ft. 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 bridge 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 fillings 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.



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 lbs./sq. ft.), where  $W$  is weight of water per c.ft., 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 = 32.2.

(ii) Pressure due to eddies:

=  $W(V_v - V)^2/2g$  (horizontal force in lbs./sq. ft.)

$V_v$  = 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  $\frac{1}{3}$ rd height). These forces are generally small compared to the bridge structure.

A detailed design for a submersible bridge has been worked out in I.R.C. 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  $1\frac{1}{2}$  ft. above H.F.L. 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.

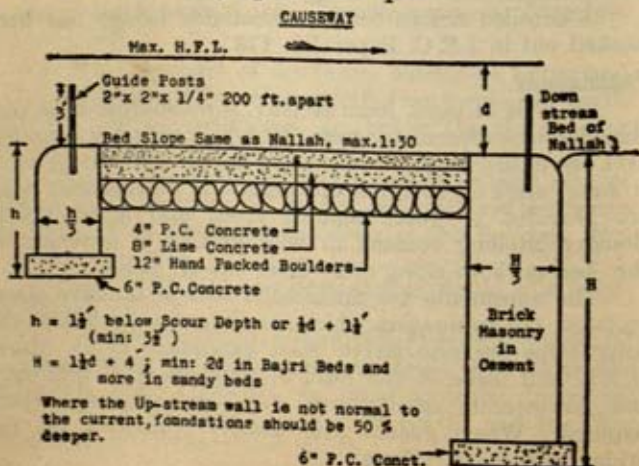
### Causeways or Irish Bridges

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 current of 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 3 ft. below the stream bed level and  $1\frac{1}{2}$  ft. above H.F.L.

### 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 on 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 2 feet span.





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.

#### 14. 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 5 ft. 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 available, 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  $1/10$ th to  $1/12$ th of the span and the towers are 3 to 5 ft. 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 100 tons/sq. in. 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 of the main bridge. Stiffened type of suspension bridges should be preferred. Braced type of stiffening is better than truss type stiffening.

$$H = \frac{wL^2}{8d}; \quad T_1 = \frac{H}{\cos \phi}$$

$$T = \sqrt{H^2 + \frac{w^2 L^4}{4}}$$

Pressure on pier, per

$$\text{cable} = \frac{wL}{2} + T_1 \sin \phi,$$

Length of cable

$$= L + \frac{8d^2}{3L}$$

H=horizontal component of tension in cable,

w=load on cable per horizontal ft. on span,

d=dip of cable at centre of span,

T=max. tension in cable,

$T_1$ =tension in anchor cable,

$\phi$ =angle of inclination of anchor cable to horizontal.

## 15. GIRDER BRIDGES

### Plate Girders

Plate girders are suitable for spans from 40 ft. to about 60 to 70 ft. 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. No plate less than  $\frac{1}{4}$  in. in thickness should be used. Girders should be connected by the web and 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. Panel length should not exceed  $1\frac{1}{2}$  times its width.

Girders of over 50 ft. span should have cast iron shoes upon the ends for expansion. Where cast iron is used for such purposes as bearing plates and other parts of the structure liable to straining action, it shall be of the strength as prescribed in Section 5. Large girders should have one end supported on rollers working in a roller box. Working stresses for steel structures are given in Section 5. Shear in webs of plate girders is taken at 5 tons/sq. in.

Approximate weight of a plate girder =  $WL/(900-L)$  where :

W=total load on the girder in tons; L=span in ft.



## 16. 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 2" thick for ordinary traffic and 3" to 4" thick for heavy and much wheeled traffic. 2" thick planks supported on road bearers or stringers at 2 ft. intervals can carry axle load not exceeding  $1\frac{1}{2}$  tons.

## 17. CULVERTS

In the B.S.S. a culvert is defined as :

- (a) "A drain, sewer or watercourse totally enclosed and usually of a size through which a man can pass. (b) An opening through an embankment for the conveyance of water by means of a pipe or an enclosed channel."

A culvert is a bridge structure of less than 20 ft. span between faces of abutments and does not generally has two spans. (Sometimes less than 12 ft. single spans are taken as culverts.)

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 only flowing half full when the approach channel is wide and shallow; if the banks are steep and the channel narrow  $\frac{3}{4}$ th 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. (An opening running full gives less discharge than when running partially full.)

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 50 ft. 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 depth about 1 to 3 ft.

In the case of culverts or causeways on black soils or soft strata, a complete raft of concrete 3 to 6 ft. 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 may be placed at an angle with the head wall, usually 30 deg. or 45 deg. 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.

It is not generally convenient to make a culvert of a size smaller than 3'  $\times$  3' or 3' dia. and it is then called a *Ventway*. No opening should be less than 4 ft. wide  $\times$  2 ft. deep.

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 3 ft. 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  $1\frac{1}{2}$ :1 slope of soil.

### Foundations and Pavements

General principles laid down for the design of bridges apply equally to culverts. Where the foundations of 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 the bed material, there should be no necessity of providing a pavement. Where the waterway has been restricted but the velocity is not expected to exceed 7 ft./sec., loose stone or block pitching may be constructed extending up to the end of the wing walls. Where the waterway is considerably restricted so that the resulting velocity exceeds 7 ft./sec., and also for culverts founded on erodible soils, floors should be paved and a paved apron given downstream extending up to the end of the wing walls. 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 one foot 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 soil) the abutment foundations should be taken to at least 5 ft. below the natural bed level; upstream drop walls 3 to 5 ft. deep, and downstream drop walls 5 to 8 ft. 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 exit is a free overfall a suitable cistern with the drop wall must be added for the dissipation of energy. (See under "Water Cushion".)

In the case of small culverts in non-erodible beds, foundations should be taken down to at least  $1\frac{1}{2}$  ft. to 2 ft. from bed level.

### Box Culverts

For R.C. box culverts, the thickness  $t$  of the top slab may be derived from the formula :

$$t = 0.085 \sqrt{B.M.}$$

where :  $t$  is the effective thickness in inches and B. M. is the bending moment in ft. lbs. Tensile reinforcement



will be 0.89 per cent of the cross-sectional area.

Vertical walls are designed for a lateral pressure equal to  $\frac{1}{3}$  of the vertical load, or bending moment is calculated as for a simple beam with average horizontal pressure as weight and height as span. (These are only approximate equations erring on the safe side.) The junctions of the walls are made rigid by providing fillets at the corners.

As box culverts are not generally built for more than 12 feet span or height, all the four walls are usually made of the same cross-section and the same reinforcement provided as for the top slab. Tensile rods are placed on the inner faces of the walls and the floor. Temperature reinforcement equal to about  $\frac{1}{4}$ th of the main reinforcement is provided in the longitudinal direction. All the four inner corners are splayed about 3 to 6 ins. and extra reinforcement provided in the fillets (same size of rods as for the tensile reinforcement—put about 12 inches apart). Additional rods of the size as for the tensile reinforcement should be provided about 12 inches apart on the outside faces of both the vertical walls and extended to  $\frac{1}{3}$ rd the distance of the top and bottom slabs so as to adequately reinforce the corners and also to act as compression reinforcements for the top and bottom slabs. Bottom slab may be extended (about 3 inches) on either side and 3 inches of lean concrete consolidated (as foundations) under the bottom where the soil is weak.

*Expansion Joints* in box culverts can be provided by leaving out a gap of  $\frac{3}{4}$ ". The two bits of the conduit are jointed by a copper strip  $\frac{3}{16}$ " thick and  $9\frac{1}{4}$ " wide made with a loop of  $\frac{3}{4}$ " dia. in the centre. The gap is filled with pre-moulded bitumen, saw dust and sand.

*Earth Cushion* : All slabs which are not designed for live loads directly on to their tops should have an earth cushion of at least 3 ft. under Class "A" loading and 2 ft. under Class "B" loading. For arches and pipes see under "Bridge Arches" in the table appended thereto.

### Pipe Culverts

For small drainage crossings pipe culverts are often found in practice to be the most economical and easily con-

structed. These culverts can be easily enlarged subsequently to take more discharge by the addition of one or more pipes. The pipes may be of C.G.I. 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 18" 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.

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.

The over-all lengths of pipes required will be 32 ft. for national and provincial highways, 26 ft. for district roads and 20 ft. for village roads. The standard length of precast R. C. pipes is 8 ft. Additional short lengths should be placed at the middle section of the barrel and not at its extreme ends.

### **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  $1:1\frac{1}{2}:3$  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 difficult to consolidate concrete if the height to be poured at a time exceeds  $2\frac{1}{2}$  to 3 ft., the forms 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 I.R.C. Journal vol. XVI-I, page 131). The pipes should be made in lengths of 4 to 6 ft. Pipe culverts are not generally made more than 6 ft. in diameter. Tubes normally have ogee joints and pipes have spigot and socket joints.

### 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.

$$(i) \quad M = \frac{1}{16} pd^2$$

$M$  = bending moment at crown and at centre of base for a unit length of culvert;  $p$  = load per unit area;  $d$  = diameter of culvert.

$$(ii) \quad M = \frac{1}{2} r^2 (w - p)$$

$M$  = bending moment 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  $\frac{1}{4}$ th to  $\frac{1}{2}$ th of the helical reinforcement.  $\frac{1}{4}$ " dia. bars at 9-ins. c/c 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;  $\frac{1}{3}$ rd 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 24" dia., or (ii) in elliptical form for pipes 24" to 72" dia., as shown in the illustration, or (iii) in two concentric layers for large diameter pipes under heavy external loads or internal 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  $\frac{1}{2}$ " 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.

### Pipe Culverts

Dia. ins.	Thickness of shell			Reinforcement for concrete pipes	
	Bwk. in cement	Plain ct. conct.	R.C. conct.	Hoop bars c/c	Longitudinal bars c/c
12"	4 $\frac{1}{2}$ "	3"	...	...	...
24"	9"	4"	2 $\frac{1}{2}$ "	1— $\frac{1}{4}$ " dia. @ 3"	1— $\frac{1}{4}$ " dia. @ 4"
36"	9"	5"	3"	2— $\frac{1}{4}$ " dia. @ 4"	2— $\frac{1}{4}$ " dia. @ 6"
48"	13 $\frac{1}{2}$ "	6"	3 $\frac{1}{2}$ "	2— $\frac{1}{4}$ " dia. @ 3"	2— $\frac{1}{4}$ " dia. @ 6"
60"	13 $\frac{1}{2}$ "	7"	4 $\frac{1}{2}$ "	2— $\frac{1}{4}$ " dia. @ 5"	2— $\frac{1}{4}$ " dia. @ 9"
72"	13 $\frac{1}{2}$ "	8"	5"	2— $\frac{1}{4}$ " dia. @ 4"	2— $\frac{1}{4}$ " dia. @ 8"

### Masonry Arch Culverts

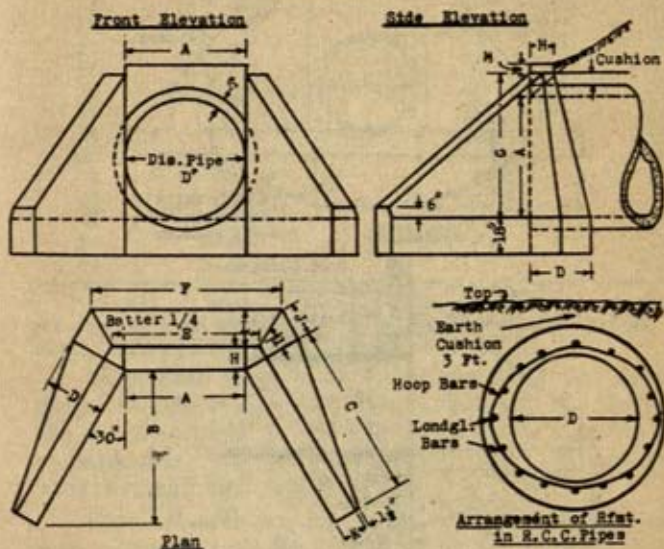
Size of arches for I.R.C. Class "A" loading has been outlined earlier. For Class 'B' loading, the following thickness of arch rings for brickwork in cement mortar may be taken :—

For Spans up to 5 ft.	9"
For Spans 6 ft. to 10 ft.	1'-1 $\frac{1}{2}$ "
For Spans 11 ft. to 18 ft.	1'-6"



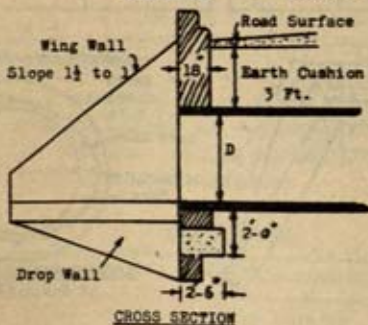
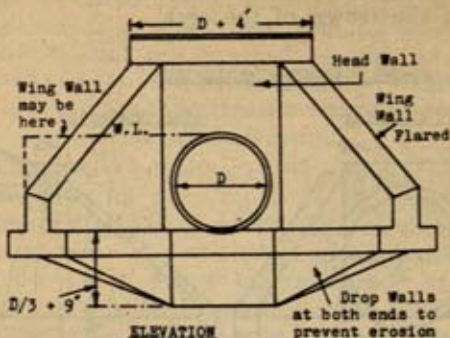
There should be an earth cushion of at least 1 ft. 6 ins. above the crown of the arch.

CEMENT CONCRETE PIPE CULVERT

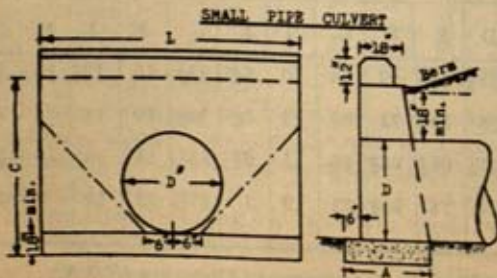


All Dimensions in Inches

Dia.	A	B	C	D	E	F	G	H	I	J	K	L	M	T
42	42	60	69½	23½	53½	69	54	10	5½	13½	10	13½	3½	4½
48	48	66	76½	25	59½	77	60	10	5½	14½	10	15	4	5
54	54	72	83½	27½	66½	85½	66	11	6½	15½	11	16½	4½	5½
60	60	78	90	30	74	94½	72	12	7	17½	12	18	5	6

PIPE CULVERTS IN BRICKWORK

*Head-Wall* is a retaining wall built at the inlet and outlet ends of a culvert, a pipe, or a sluice.



Min: Dimensions  
in inches

D	L	C	A
12	72	48	24
15	81	51	24
18	90	54	24
24	108	60	27
36	144	72	30



## Requirements for C.G.I. Sheet Pipe Culverts

Dia. ins.	Length of sheet before forming ins.	Min. width of lap ins.	Sheet gauge No.
8	28½	1½	16
10	35	1½	16
12	41	1½	16
15	50½	1½	16
18	60	1½	16
21	69½	1½	16
24	80	2	14
30	98	2	14
36	117	2	12
42	137	3	12
48	156	3	12
54	2-80	3	12
60	2-98	3	10

Pipes 42" or larger in diameter should be elongated vertically 5 per cent by field strutting or by an approved shop method. The struts shall be placed before the backfill is started.

High head walls need not be provided for retaining deep overfills as they are costly; instead, the length of the culvert should be increased suitably so that the road embankment, with its natural side slopes, is accommodated without high retaining walls.

Pipe culverts can also be made of plain sheets up to 6 ft. diameter and No. 8 gauge. Sheet culverts must be surrounded by concrete and should have adequate earth cushion. Use heavier gauge under heavy fills.

Rivets may be of the following diameters for the gauge Nos. specified :

Gauge No.	16	14	12	10	8
Dia. in.	$\frac{5}{16}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{3}{8}$	$\frac{3}{8}$

The longitudinal laps in all pipes 42 inches or more in diameter should be double riveted. Circumferential shop-riveted laps should have a maximum rivet spacing of 6 inches; 6 rivets will be sufficient in a 12-inch pipe. The outside laps of circumferential joints should be pointing upstream and the longitudinal laps on the sides.

## Theoretical Weights of Galvanised Sheets

Gauge No.	8	10	12	14	16
Wt. oz. per sq. ft.	112.5	92.5	72.5	52.5	42.5

**Rules for Construction of Pipe Culverts under Deep Fills**

(Based on British Ministry of Transport instructions)

(a) All pipes and tubes with 20 ft. of cover to be surrounded with at least 6 inches of concrete.

(b) Subject to (a) all pipes and tubes with over 14 ft. of cover to be bedded on and haunched with at least 6 inches 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 18 inches diameter and over to be bedded on and haunched with at least 6 inches 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 18 inches diameter and with less than 14 ft. 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 3 ft. of cover in fields or 4 ft. of cover in roads to be surrounded with at least 6 inches 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 one foot.

**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. See also "Approach Slab" under "Abutments".

## 18. TRAINING WORKS

(See also 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  $\frac{1}{10}$ th to  $\frac{1}{5}$ th of this length. The exposed ends of training bunds should be curved off at an angle of 120 deg. to 140 deg. 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 300 ft. 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. (I.R.C.)

### Wire Crates and Mattresses for River Training Works

Wire crates for shallow or accessible situations can be about  $10' \times 5' \times 4'$  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  $25' \times 10' \times 2'$  or smaller than  $6' \times 3' \times 1'$ . Sides of large mattresses should be securely stayed at intervals of not more than 5 ft. to prevent bulging.

The size of the netting will depend upon the size of the boulders available. For size of boulders 8" min. 6" mesh will be required and can be made from No. 6 gauge galv. iron wire. This will require approximately 65 lbs. of wire per 100 sq. ft. For 6" boulders (min. side), a 4" mesh or 6"  $\times$  4" mesh can be woven with No. 8 wire. This will also need about 65 lbs. of wire for 100 sq. ft. Boulders of one foot average diameter and weighing not less than 80 lbs. are best.

**Stone Riprap for Foundation 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 tons.

## 19. 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:  $S=1\frac{1}{2}H$ .
- (iii) R.C.C. beam and slab on masonry piers:  $S=1\frac{1}{2}H$ .
- (iv) R.C.C. slab on pile bents:  $S=\frac{3}{4}H$  to  $1H$ .
- (v) Steel truss spans on masonry piers:  $S=3H$ .

where :

$S$ =clear span length in ft.,

$H$ =total height of one pier in ft. from the underside 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.



## 20. PREPARATION OF PROJECT ESTIMATES FOR BRIDGES

(Based on the instructions issued by the Consulting Engineer (Roads) to the Government of India. See also under "Roads and Highways".)

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  $1\frac{1}{2}$  to 2 times the design depth. The estimated depth of scour with details of observations or any other special causes responsible 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 inch to 1 mile.

(b) *A Site Plan* drawn to a suitable scale showing details of the site selected and extending not less than 500 ft. upstream and downstream and covering the approaches to a sufficient distance.

(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.

This may be for 300 ft. for catchment areas less than one square mile, 1000 ft. for catchment areas of 5 sq. miles; in case of bigger catchment areas, the survey may be extended up to 1 mile. (Scale 1 inch to 100 ft. for small areas and 1 inch to 330 ft. for large areas).

Complete details about the catchment area on which the waterway of a bridge and its full design depends, are very essential.

(d) *A Cross-Section* of the river at the site of the proposed bridge. It should show: The highest flood levels, ordinary flood level, low water level, bed level, the nature of the soil and various strata down below the depth of foundations. It should be drawn to a sufficient distance beyond the edges of the stream. Suitable scale is not less than 1 inch to 100 ft. horizontal and not less than 1 inch to 10 ft. vertical.

(e) *A Longitudinal Section* of the stream extending to about 300 to 500 ft. on either side of bridge site and showing all the levels described for the cross-section.

(f) *Boring Charts.*

## 21. GLOSSARY OF TERMS

*Abutment Pier*—Is a heavier pier, 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).

*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.

*Baffle 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.

*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 specified section or (b) a reservoir. (I.R.C.)

*Catchment Drain*—A drain excavated on the upper slope of a hill road to 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. (I.R.C.)

*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 R.C.C. which are assembled into a wall for retaining stones and rubble, as a retaining wall. (Not generally used in India).

*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. (I.R.C.)

*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 R.C.C. columns connected by thin R.C.C. web for their full heights.

**Fender**—A replaceable device for protecting structures 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.

**Gorge**—A narrow passage between hills.

**Guard Rail**—A side barrier or protection consisting of a rail supported on posts to constrain traffic.

**Kerb Inlet**—Apertures formed in a kerb to convey storm water to a gully.

**Over Bridge**—A bridge that enables one form of land communication over the other.

**Piping**—The flow of water under or round 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.

**Running Ground**—Either water-bearing sand or very dry sand that will not stand up without support. (See under "Quicksand" in Section 6).

**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 overtopped 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



the 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.





**ESTIMATING & QUANTITIES**

(For Water Supply and Road Works see under the respective Sections)

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In an ordinary type of building the cost of materials comes to about 60 per cent and labour about 40 per cent of the total cost. Labour is divided as follows :—

Excavation—5% ; Masons—50 % ; Carpenters—40% ; Smiths—5%. The trades include labourers working with them.

Further, the costs may be roughly, for single storey :—

Plinth 18% ; Superstructure 27% ; Flooring 8% ; Roofing (flat) 20% ; Woodwork 17% ; Miscellaneous 10%.

*Note regarding Labour :*

The task work for artisans and labourers is for Northern India and done under “contract” conditions. It is generally considered that labourers in the other parts of the country usually put in less work (which may be as low as only half). If women or boys are employed, the figures should be increased accordingly. Estimates for labour are always approximate.

## 1. EARTH-WORK & EXCAVATION

One labourer does in one day about 100 to 150 c.ft. of excavation in ordinary soil (from moderately hard to soft) with small lead, say about 50 ft., and dressing. This includes a lift of 5 ft. from the borrow pit to the bank. (If the bank is above 5 ft., lift will be depth of pit plus height of bank and this will involve extra labour.) Labour involved in lifting a load through one foot height is taken equal to travelling a horizontal distance of 10 ft., and in some provinces 12.5 ft.

*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.

Excavation in the following soils under above conditions will be :—

In soft moorum or hard earth	...	75 c.ft.
In hard moorum	...	35 c.ft.
In soft rock	...	20 c.ft.
In hard rock	...	10 c.ft.



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.

The following allowances shall be made in profiles of embankments for settlement of fresh earth :

Compact earth fill ... 1" to 1½" per ft. height.

Loose earth fill ... 1½" to 2" per ft. height.

Black cotton soil ... 2" to 3" per ft. height.

(Also see under "Embankments" in Section 17).

## 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 total volumes of all the ingredients together. The volume is further reduced when water is added.

In the case of lime mortars, the volume of wet mortar is about two-thirds the total volume of dry ingredients. 1 c.ft. sand + 1 c.ft. lime + 1 c.ft. surkhi will make about 2½ to 2¾ c.ft. dry and 1½ to 2 c.ft. of wet mortar. 3 c.ft. lime + 6 c.ft. sand will produce 8½ c.ft. of dry mix and 6 c.ft. of wet mortar. Usually 100 c.ft. of dry mixed mortar is taken to make only 70 c.ft. of wet mortar.

### Cement Mortars

Quantity of cement and dry sand per 100 c. ft. of mortar :—

Mix.	Cement in cwts.	Sand c. ft.	The quantity of cement varies according to the fineness of the sand used. Fine sands need more cement. Well graded sands need much less cement.  1 c.ft. of neat cement will produce 0.87 c. ft. of cement paste.
1 : 1	48.5 to 52.5	65	
1 : 2	34.5 to 38.5	95	
1 : 3	24.5 to 27.5	100	
1 : 4	18.5 to 20.0	100	
1 : 5	16.0 to 17.0	100	
1 : 6	13.5 to 14.5	100	
1 : 7	12.0 to 12.75	100	
1 : 8	10.5 to 11.0	100	
1 : 10	8.0 to 8.5	100	

Water also occupies space.

## 3. CONCRETES

(See also Section 12 under "Mixtures for Mortars & Concretes").

**Lime Concrete** : per 100 c. ft. of finished concrete :-

*Materials*  
Ballast  $1\frac{1}{2}$ "—110 c. ft.  
Wet mortar—37 c. ft.

Dry mortar :  
kankar lime 40 c. ft.  
or  
white lime 17 c. ft. }  
sand or surkhi 34 c. ft. }  
or  
white lime 18 c. ft. }  
sand 18 c. ft. }  
surkhi 18 c. ft. }

Ramming reduces the volume of aggregate.

*Labour*  
Mason  $\frac{1}{4}$  } Materials  
Bhistie 1 } available at  
Labourers 4 } site of work.

*Breaking brick metal* :  
30 to 40 c. ft. per day by one man,  $1\frac{1}{2}$ " to  $1\frac{1}{4}$ " size, and 20 to 25 c. ft.  $\frac{3}{4}$ " to  $1\frac{1}{2}$ " size. 800 to 1000 bricks give about 100 c. ft. of brick bats. 100 c. ft. of brick bats make about 80 to 90 c. ft. of brick ballast. About 1100 to 1200 bricks make about 100 c. ft. ballast. (9" bricks are considered.)

For concrete over jack-arches, 10 per cent. extra mortar should be used with aggregate of  $\frac{1}{2}$ " to  $\frac{3}{4}$ " size.

Brick ballast must be soaked well in water before mixing.

**Cement Concrete** : 1 c. ft. of loose cement will make :—

4.3 c.ft.	1 : 2 : 4 concrete	} Approx.
5.0 c.ft.	1 : $2\frac{1}{2}$ : 5 concrete	
5.8 c.ft.	1 : 3 : 6 concrete	
7.5 c.ft.	1 : 4 : 7 concrete	

"All-in" Aggregate	Cement, cwt. or bag	C. ft. of combined aggregate	Mass concrete
* 1 : 6 Mix.	1	7 $\frac{1}{2}$	All combined aggregate to pass a 2-in. ring and to contain sufficient sand.
1 : 8 "	1	10	
1 : 10 "	1	12 $\frac{1}{2}$	
1 : 12 "	1	15	

\* 1 : 6 "all-in" aggregate is not the same as 1 : 2 : 4 mix. which will be about 1 : 5.



Quantities of materials required for various mixes for 100 c. ft. of finished concrete :—(Machine mixing)

Mix.	Cement in cwt. for		Dry sand	Aggregate	Labour Mass work
	Broken Stone	Shingle, Gravel	c. ft. av.	c. ft. av.	Mason $\frac{1}{2}$ Labourers $\frac{4}{4}$ Bhistie 1
1 : 1 : 2	32.0	30.5	40	80	One bag of cement is 110 lbs. net (112 lbs. less 2 lbs. for weight of the sack)
1 : 1½ : 3	22.1	20.6	42	84	
1 : 2 : 3	21.0	19.8	52	78	
1 : 2 : 4	17.0	15.9	45	90	
1 : 2½ : 5	14.8	14.0	46	92	
1 : 3 : 6	11.6	10.8	47	94	
1 : 4 : 8	8.7	8.0	48	97	
1 : 5 : 10	7.1	6.6	49	98	
1 : 6 : 12	6.0	5.5	50	100	
1 : 6 : 18	4.2	3.8	3	108	
1 : 9 : 16	5.2	4.8	59	106	

Cement should be measured by weight. If mixing is done by hand, add 10 per cent. extra cement.

Volume of aggregate is reduced by ramming. Cement concrete is not rammed heavily (see under Section 8.) Crushed rock or broken stone need more cement and more sand. Fine sands need more cement. Well graded sands need less cement.

Mechanical mixers are denoted by their drum capacities for dry ingredients and mixed concrete. By 10/7 capacity mixer means 10 c. ft. of dry ingredients will produce 7 c. ft. of wet mix.

Very approximate figures of the quantities of ingredients of concrete may be obtained by considering the volume of all ingredients at 1½ times that of the resultant concrete.

Comparative costs of cement concrete in foundations, per 100 c. ft. :—

1 : 4 : 8 mix. Rs. 83.00 ; 1 : 5 : 10 mix. Rs. 73.40 ;  
1 : 6 : 12 mix Rs. 67.45.

*Breaking Stone Metal :*

One man will break about 20 to 30 c. ft. of stone

metal 1" to 1½" gauge ; 4 to 6 c. ft. ½" to ¾" gauge ; 2 to 3 c. ft. ¾" to 1" gauge according to the hardness of the stone. (Brick metal has been given under Lime Concrete.)

Broken brick or stone metal when re-stacked and measured may show a difference in volume up to 7 per cent, while gravel and shingle may differ by 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.

### Reinforced Concrete :

Take about 4 lbs. of reinforcement rods per c. ft. of concrete where only simple tensile bars are used. This quantity will be about 7 lbs. per c. ft. where stirrups, transverse and shear reinforcements are also to be included.

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	½"	¾"	1"	1½"	2"	2½"	3"	3½"	4"
Allowance ... ..	5"	7"	10"	12"	14"	16"	18"	22"	25"

Add for extra labour say, about ½ smith for the reinforcement work per 100 c. ft. of concrete. And cost of soft iron binding wire, 14 or 16 gauge, at the rate of 15 to 12 lbs. per ton of bars.

### Labour for Cutting and Bending rods :

½" to 1" dia.	2½ cwts.	one man per day
1" dia.	3 cwts.	one man per day
1½" and above	4½ cwts.	one man per day

Add for forms where required and extra labour for the same. Total labour for laying a 100 sq. ft. roof slab will be about :

Masons	4	} Hire charges for forms and column bricks for supporting the forms should be added.
Carpenter	½	
Labourers	10	



*Pre-cast Concrete works* (in steel moulds) :

Hollow blocks in hand machine :

1 mason and 2 labourers will make about 120 blocks per day

Paving flags— 35 nos. ;      Roofing tiles— 100 nos.

#### 4. BRICKWORK

**Brickwork in Lime** : per 100 c. ft.

<i>Materials</i>	<i>Labour</i>
Bricks—9" standard size—1350 to 1400 nos. (inclusive of wastage)	Masons 2½ Bhistie 1
Mortar, dry (lime, surkhi, sand, etc.) 33 to 40 c. ft. or	Labourers 3 (excluding scaffolding)
Kankar lime 25 c. ft.	

(a) Honey-comb brickwork requires about 2 c. ft. of mortar and half the labour, per 100 c. ft.

(c) Archwork requires double the labour of simple work.

(d) For well steening work, if cut bricks are used, mortar will be 33 c. ft. but when special moulded bricks are used mortar required will be only 25 c. ft.

**Brickwork in Cement** : Quantity of cement required in cwts. per 100 c. ft. of brickwork in cement/sand mortar with 25 c. ft. of mortar.

1 : 2	1 : 3	1 : 4	1 : 5	1 : 6	Add 10 per cent for wastage
8.0	6.3	5.0	3.8	3.2	

Labour—3 masons and 4 labourers (excluding scaffolding).

Comparative costs of Brickwork in Superstructure :—

- (i) Masonry in mud mortar—Rs. 56.70
- (ii) Ditto. with jubiface in 1 : 3 cement—Rs. 63.80
- (iii) Ditto. in cement mortar 1 : 7—Rs. 80.0
- (iv) Ditto. in cement mortar 1 : 6—Rs. 81.70
- (v) Ditto. in cement mortar 1 : 3—Rs. 90.0

**Reinforced Brickwork** : per 100 c. ft.

with cement mortar 1 : 3 in joints (Roof Slabs)

<i>Materials</i>		<i>Labour</i>	
Bricks 9"	900 nos.	Masons	3
Cement	18 c. ft.	Bhistie	1
Sand	54 c. ft.	Labourers	5
Steel (approx.)	5 cwts.		

If the work is for slabs or beams, add extras for form-work as detailed under Reinforced Concrete.

With cement concrete 1 : 2 : 4 in joints and 1½" on top of slab, per 100 c.ft. of work (based on 6" thick slab) :

Bricks	...	500 nos.
Cement concrete	...	70 c. ft.

Add for forms and steel etc.

**5. STONE MASONRY**

*Materials* : per 100 c. ft. of finished work : (Approx.)

(i) Uncoursed rubble masonry

120 c. ft. stone and 30 to 35 c. ft. wet mortar

(ii) Coursed rubble

130 c. ft. stone and 20 to 28 c. ft. wet mortar

(iii) Ashlar

110 c. ft. stone and 6 to 15 c. ft. wet mortar.

Quantity of mortar is according to height of courses.  
Stone measured in loose stacks.

*Labour* :—1 mason, ½ Bhistie and 1 labourer :

Dry stone masonry	50 c. ft. per day	Complete
Uncoursed rubble	30	job includ-
Coursed rubble 1st class	12	ing dress-
Coursed rubble 2nd class	18	ing.
Coursed rubble 3rd class	25	

Add to all items extra for scaffolding and heights.

**6. CEMENT CONCRETE FLOORS**

(a) 1 : 2 : 4—1½" thick —per 100 sq. ft.

<i>Materials</i>		<i>Labour</i>	
Cement	2.8 c. ft./2.25 cwts.	Mason	1
Sand	5.5 c. ft.	Labourer	1
Gravel	11.0 c. ft.	Bhistie	1



(b) 1 : 3 : 6— $1\frac{1}{2}$ " thick floor with  $\frac{1}{2}$ " thick wearing surface on top of fine grit 1 : 2.

(i) Gravel for $1\frac{1}{2}$ " bottom layer, size $\frac{1}{8}$ " to $\frac{3}{4}$ "	12 c.ft.
Sand	6 c.ft.
Cement	2 c.ft.
(ii) Grit fine $\frac{1}{4}$ " and less for $\frac{1}{2}$ " top layer	4 c.ft.
Cement	2 c.ft.

(c) For marble chips flooring  $\frac{1}{4}$ " thick (top) :

Chips $\frac{1}{8}$ " grit	..	..	2 c.ft.
Coloured cement	..	..	1 c.ft.

(d) For red oxide colour floor top  $\frac{1}{4}$ " layer we need about 10 lbs. red oxide per c.ft. of loose cement.

Weight of colouring pigment used in excess of 12 per cent. of the weight of cement reduces the strength of the mortar. Where the strength will not matter, a max. proportion of 1 colouring pigment to 3 of cement can be used, and 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 : coarse, medium and fine grades. 1 part beeswax and 3 parts turpentine are well rubbed over the floor for high class polish.

The following materials are required to treat about 300 sq. ft. of floor surface.—

6 oz. beeswax ; 1 pint turpentine ; 4 oz. pigment.

See also Section 8.

**Brick Floors :** per 100 sq. ft. —9" bricks.

(i) Flat—3" thick with pointing :

Bricks— 356 nos. (without wastage)		Labour	
or 380 nos. (including wastage)		Mason	$\frac{3}{4}$
Mortar— 9 c.ft.		Labourers	$1\frac{1}{4}$

(ii) On Edge— $4\frac{1}{2}$ " thick with pointing :

Bricks— 533 nos. (without wastage)		Mason	1
or 550 nos. (including wastage)		Labourers	2
Mortar— 13 c.ft.			

(b) Flat brick flooring—Jointing in cement mortar 1 : 6 and flush pointing in cement mortar 1 : 3—Quantity of cement required = 1 cwt. per 100 sq. ft.

Comparative costs of floors, per 100 sq. ft. :—

- (i)  $1\frac{1}{2}$ " thick cement concrete 1 : 2 : 4, over 3" cement concrete 1 : 5 : 10, over 6" sand filling—Rs. 46.60
- (ii) 1" thick cement concrete 1 : 2 : 4, over 3" cement concrete 1 : 6 : 12, over  $4\frac{1}{2}$ " sand filling—Rs. 40.20
- (iii)  $\frac{3}{4}$ " thick cement concrete, ditto.—Rs. 33.00
- (iv)  $\frac{3}{4}$ " thick cement concrete 1 : 2 : 4, over  $4\frac{1}{2}$ " brick on edge, over 3" sand filling—Rs. 39.30
- (v)  $\frac{1}{2}$ " thick cement plaster 1 : 3, over  $4\frac{1}{2}$ " brick on edge, over 3" sand filling—Rs. 34.80
- (iv)  $\frac{1}{2}$ " thick cement plaster 1 : 3, over brick-bats well rammed, finished thickness 4"—Rs. 22.70

## 7. POINTING

### Cement Pointing :

(a) Cement required per 1000 sq. ft. of flush pointing on walls :—

	1 : 1	1 : 2	1 : 3	<i>Labour</i>	
Cement	8.0 c. ft.	5.0 c. ft.	3.8 c. ft.	Masons	3
Sand	8.0 c. ft.	10.0 c. ft.	11.4 c. ft.	Bhistie	1
				Labourers	4

This includes for raking joints and watering, etc.

For ruled and tuck pointing take 75 per cent extra mortar and labour.

For stone walls mortar required will be 50 per cent more and labour about  $\frac{1}{3}$  more.

(b) Requirement of mortar and labour for floors will be  $\frac{1}{4}$ th less than for walls.

Fine sand is used for pointing.



**Lime Pointing :**

Mortar required is about 10 c.ft. wet for flush pointing and 20 c.ft. dry or 15 c.ft. wet for tuck and rule pointing, per 1000 sq. ft. of wall surface. Add 10 per cent for wastage.

Proportions of mortars are given in Section 12 and also in Section 7.

**8. PLASTERING**

(See also under Sections 7 and 12.)

**(a) Mud Plaster : 1" thick per 100 sq. ft.**

<i>Materials</i>		<i>Labour</i>	
Mud	10 c.ft.	Mason	$\frac{1}{2}$
Bhusa or straw	1 c.ft.	Labourers	2

**(b) Lime Plaster :  $\frac{1}{2}$ " thick per 100 sq. ft. (one coat)**

<i>Materials</i>		<i>Labour</i>	
Dry mortar (lime, surkhi, sand, etc.)	6 c.ft.	Mason (roughwork)	$\frac{1}{2}$
or Kankar lime	5 c.ft.	Mason (fine work)	1
Chopped jute or hemp	4 oz.	Labourers	2
		Bhistie	$\frac{1}{4}$

**(c) Cement Plaster :**

Quantity of cement required per 100 sq. ft., in cwts. :

Cement-sand	1 : 2	1 : 3	1 : 4	1 : 5	1 : 6
$\frac{3}{8}$ " thick	1.4	1.1	0.8	0.7	0.6
$\frac{1}{2}$ " "	1.6	1.3	1.0	0.8	0.6
$\frac{3}{4}$ " "	2.4	1.9	1.4	1.1	0.8
1" "	3.3	2.6	2.0	1.4	...

Comparative costs of Plastering per 100 sq. ft. :—

- (i) Cement-sand mortar 1 : 2,  $\frac{1}{2}$ " thick—Rs. 16.20  
(ii) Ditto. " 1 : 3,  $\frac{1}{2}$ " thick—Rs. 14.00  
(iii) Ditto. " 1 : 4,  $\frac{1}{2}$ " thick—Rs. 12.80  
(iv) Ditto. " 1 : 5,  $\frac{1}{2}$ " thick—Rs. 12.00  
(v) Ditto. " 1 : 6,  $\frac{3}{8}$ " thick—Rs. 10.30

- (vi) Cement-lime-sand mortar 1 : 3 : 10,  $\frac{1}{2}$ " thick  
—Rs. 10.50
- (vii) Ditto. " 1 : 3 : 10,  $\frac{3}{8}$ " thick  
—Rs. 9.60
- (viii) Ditto. " 1 : 3 : 12,  $\frac{1}{2}$ " thick  
—Rs. 10.00

## 9. WHITE-WASHING

(Specifications are given in Section 7.)

1 coat	3 lbs. white lime unslaked	} Per 100 sq. ft. of surface
2 coats	5 lbs. " "	
3 coats	7 lbs. " "	

Gum may be added where desired at the rate of 1 oz. to 15 lbs. of lime. Rice water is also used instead of gum. A little quantity of indigo (blue colour) is added to kill the glaze. (See also under Section 7.)

*Labour* : One man should be able to do about 1500 sq. ft. of surface, one coat.

Shell lime is also used instead of white lime where available.

### Cement Washing

One man will do 500 sq. ft. 2 coats in one day.  
(Specifications are given in Section 8.)

### Distempering

(Specifications are given in Section 7.)

1st coat	2 $\frac{1}{2}$ lbs.	} Per 100 sq. ft. of surface.
2nd and subsequent coats	1 $\frac{1}{2}$ lbs.	

Add primer with 1st coat  $\frac{1}{2}$  gall.

Cost of labour will be about 3 times that of white-washing. (Work should be done according to the specifications of the manufacturers.)

### Water Assessment for Building Operations

Brickwork, masonry and concrete	@ 1400 gall/100 c. ft.
Plaster	@ 175 gall/100 sq. ft.
Pointing	@ 120 gall/100 sq. ft.
White-washing per coat	@ 5 gall/100 sq. ft.



## 10. WOOD-WORK for DOORS AND WINDOWS

Sizes of frames are given in Section 9.

Sawing of planks from logs of wood has not been taken in the labour items as planks are now available ready sawn. Quantity of wood includes for wastage.

Labour items are for softwood.

Add extra for fittings.

- (a) 2" thick panelled and glazed door with frames, 4' x 7' :—

Wood 5.8 c. ft. (inclusive of frames)

Labour : Carpenters 4 ; Glazier  $\frac{1}{2}$  ; Labourers 2.

- (ii) Ditto. all panelled :—

Wood 7 c. ft.

- (b)  $1\frac{1}{2}$ " thick panelled and glazed door with frame 4' x 7' :—

Wood 5 c. ft.

Frame 1.7 c. ft. + Shutters 3.3 c. ft. = 5 c. ft.

- (ii) Ditto. all panelled :—

Wood 6 c. ft.

- (c)  $1\frac{1}{2}$ " thick wire-gauze door with frame  $6\frac{1}{2}$ ' x 4'.

Wood 4.0 c. ft.

Wire gauze 13 sq. ft.

Labour : Carpenters 3 ; Labourer 1

- (ii) Ditto. shutters only : (without frame)

Wood 2.7 c. ft.

If hardwork (teak) is used, the thickness can be reduced to 1" instead of  $1\frac{1}{2}$ ", which will reduce the cost (material plus labour) by about 15%.  $1\frac{1}{2}$ " will reduce the cost by about 8%.

- (iii) Ditto. 2" thick, ditto.

Wood 3.6 c. ft.

- (d)  $1\frac{1}{2}$ " thick battened door with 1" braces, complete with frame  $6\frac{1}{2}$ ' x  $3\frac{1}{2}$ ' :

Wood 5.5 c. ft.

Labour : Carpenters 3 ; Labourer 1

- (ii)  $1\frac{1}{2}$ " thick, ditto. 7' x 4' (rebated) :—

Wood for shutters only 5.2 c. ft.

(iii) 1" thick ditto., 6' x 3' (butt jointed) :—

Wood for shutters only 2.5 c.ft.

(e) 1½" thick, venetian doors and windows :

Wood—about 18 c.ft. per 100 sq. ft. area.

Labour : about the same as for panelled and glazed doors.

(f) 1½" thick glazed windows 4' x 5' with frame :—

Wood	3.4 c.ft.	Carpenters ...	...	3
		Labourers ...	...	2
		Glazier ...	...	¼

(g) Jafri or Trellis work : per 100 sq. ft.

Wood about 7 c.ft. (inclusive of frames)

Labour : Carpenters 3 ; Labourers 2

(h) Framing labour 3 c.ft. one man per day

(i) Joinery (hardwork) 1½ " " "

4 lbs. of putty is required for 1000 sq. ft. of glass panes.

## 11. PAINTING

Ready mixed paints :

Per 100 sq. ft. on wood-work :—(Approx.)

	New work	Old work
1st coat ... ..	2.8 lbs.	1.9 lbs.
2nd coat ... ..	1.8 lbs.	1.8 lbs.
3rd coat ... ..	1.7 lbs.	1.7 lbs.

### Proportions of Paints in lbs.

Type of surface	Paint	Boiled linseed oil	Raw linseed oil	Turpentine
Outside work	112 lbs.	16 lbs.	28 lbs.	5—6 lbs.
Inside work	112 "	22 "	22 "	—

Red oxide paints for iron-work will be about :—

2.5 lbs. for 1st coat

1.8 lbs. for 2nd coat



## Painting Iron work according to Railways Specifications :—

	Red lead	Red oxide	Double boiled linseed oil	Raw linseed oil	Lamp black	Covering capacity
	lbs.	lbs.	gall.	gall.	lbs.	sq. ft.
1st coat	28	...	$\frac{3}{4}$	$\frac{1}{2}$	...	1200
2nd coat	...	12	$\frac{3}{4}$	$\frac{1}{2}$	$\frac{1}{2}$	1300

Turpentine is not added for painting massive structures such as girders of bridges.

Anti-corrosive paints made of tar are available for iron works under water.

**Labour**—One painter will do about 500 to 700 sq. ft. of surface per day, which can be up to 1000 sq. ft. if the surface is of iron sheets and the paint is running freely. For more difficult and careful work, such as doors and windows, a painter might do only about 200 sq. ft. a day.

To the above costs of paints and painters, labour required for scraping old work, (which might come to about the same wages as of the painter) and for brushes and sundries, etc., which will cost about 1/15 to 1/20 of the cost of labour and material, should be added.

(b) **Enamel Paints** : Will cover about 55 sq. ft. of surface with 1 lb. of the paint.

(c) **Additions over plane surfaces for painting** :

In measuring up painting, mouldings will, unless otherwise stated, be measured by running the tape into and over all elevations and depressions; doors and windows will be measured flat over-all including door frames, and the area multiplied by the following factors :

(i) According to Madras P.W.D. Specifications :—

Panelled or battened	$2\frac{1}{2}$ times one side
Panelled and venetianed	$3\frac{1}{4}$ „
Ditto. with glazed top	3 „
Iron barred doors	$1\frac{1}{2}$ „
Ditto. with battens and sheet	$3\frac{3}{4}$ „

Battened windows with iron bars	2½	„
Venetianed windows	3½	„
Ditto. with iron bars	4	„
Ditto. with glazed top and iron bars	4½	„
Ditto. with iron bars and glass shutters	5	„
Glazed windows with iron bars	2	„
Ditto. without iron bars	1½	„
Glazed shutters (measured over shutters only)	1	„

(ii) According to the Punjab P.W.D. Specifications :—

Panelled or battened	2 times one side
Glazed or partly glazed	2 „
Venetianed and louvred	1½ „
Wire-gauzed	1 „
Grated	¾ „
Plate glass windows (large glazed area)	1 „
Trellis work	2 „

The above additions are made where there is only one flat rate for all works, in view of the extra labour and care involved.

### Percentage additions for painting roof surfaces :

#### *Nanital pattern using flat sheets :*

Painted before erection	16 per cent
Painted after erection	12 per cent

#### *As for above using corrugated sheets :*

Painted before erection	30 per cent
Painted after erection	25 per cent

### Varnishing

1 gallon will cover about 125 sq. ft. of new surface three coats, or about 700 sq. ft. of old surface, one coat.

*Labour :* About the same as for Painting.

### Oiling (with linseed oil)

1 gallon will cover about 800 sq. ft. one coat or about 500 sq. ft. two coats.

#### H.C.F. oil 50/50 blend wood Preservative :

For 1 coat	1 gall. per 250 sq. ft.
For 2 coats	1 gall. per 150 sq. ft.



**Coal Tarring :** (*Wood-work*) per 100 sq. ft.

One coat	5 lbs.	}	Add cost of kerosene oil, etc. Prepare tar according to specifications given in Section 12.
Two coats	8 lbs.		

**Solignum Painting :** per 100 sq. ft.

One coat	$\frac{1}{2}$ gall.	Two coats	$\frac{1}{2}$ gall.
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**Bitumen Painting for Roof Surfaces or Damp-Proof Courses, etc.**

25 lbs. bitumen	}	Per 100 sq. ft. of surface
2 c.ft. sand		

## 12. JACK ARCH ROOFING

Per 100 sq. ft.— $4\frac{1}{2}$ " thick

Bricks	550 nos.
Concrete for haunches	50 c.ft.— $\frac{3}{4}$ " gauge.

Calculate quantity of cement, sand, ballast etc., and also labour from respective items. Add extra for centring.

## 13. DOUBLE ALLAHABAD TILE ROOFING

Tiles required per 100 sq. ft. of roofing :—

Flat	...	220 nos.	105 nos.
Half round	...	110 "	105 nos.
Semi-hexagonal	...	110 "	nil
Ridge flat	...	10 "	For single tiling.
Ridge semi-hexagonal	...	10 "	
Ridge elbow	...	5 "	
Eaves	...	10 "	
Ventilator	...	6 "	

Add—lime concrete 4 c. ft.

For Mangalore Tiles, see Section on "Roofs".

<i>Labour</i>	<i>D.A. Tiles</i>	<i>M. Tiles</i>
Tile layers	2	1
Mason	1	...
Labourers	5	2
Carpenter (fixing roof battens)	1	1

## 14. C.G.I. SHEET ROOFING

<i>Materials per 100 sq. ft.</i>		<i>Labour</i>	
G.I. sheet, 8' x 32"	6 nos.	Carpenter	1
Limpet washers	1 lb.	Smith	1
G.I. bolts and nuts	1½ "	Labourers	2½
G.I. screws	3 "	(wood-work and	
Iron for hook bolts, etc.	3 "	ridge is extra).	

## 15. BURNING LIME-STONE IN KILN

100 c. ft. of limestone or kankar when burnt will produce about 80 c. ft. of burnt material, and 100 c. ft. of burnt material will produce about 82 c.ft. of ground lime.

Limes have varying properties. See Section 12.

10 to 15 maunds of coal or coal dust, or 30 to 40 maunds of fairly dry wood will burn about 100 c.ft. of limestone. (This quantity is very variable)

16. BRICK BURNING (*Brick Kiln*)

About 20 tons (540 mds.) of coal is required to burn one lakh bricks, 9" size, in 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 tons of coal has been found sufficient ; where the sub-soil water level is high, a quantity as high as 24 tons may be required. Sandy soils also need more coal.)

Where wood is used instead of coal, the quantity varies from 500 mds. to 1000 mds. depending on the wood. If the wood is made into small pieces, lesser quantity will be required. (500 mds. min.) Tamarind or Babul is the best wood for burning.

About 5½ tons of coal is required for starting the firing of a 14-wall kiln and about 9 tons for a 24-wall kiln. For top firing good coal dust is required. Where wood is used the quantity required is about 300 to 400 mds. for a 14-wall kiln. 30 to 40 mds. of wood is required for first firing of a small kiln.



About one ton of coal is required for burning 1000 c.ft. of *surkhi*.

Slack 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 100 c. ft. of clay and 3 to 5 c. ft. of sand is required for making 1000 bricks.

It is not economical to burn less than 6 to 7 lakhs of bricks in a single kiln. The kiln is generally elliptical in shape. A typical economical size is :—

Length—200 ft.

Width of each trench—26 ft.

Gap between trenches—15 ft.

(This will give a radius for the semi-circle of 33 ft.)

Depth to have 16 to 18 layers.

A small size circular kiln with 3 lakhs capacity is :—

Outer dia.—73 ft. Inner dia.—48 ft. Trench—25 ft. wide, Depth—7'-6".

Size of the Chimney :

Bottom—8 ft.  $\times$  1 $\frac{1}{2}$  ft.

Height of lower part—8 ft.

Width at top of the lower part—3 $\frac{1}{2}$  ft.

(8 ft. reduces to 3 $\frac{1}{2}$  ft.)

Height of upper part—22 ft. for big size (15 ft. for small size). Total height of chimney is 35 ft.

When two chimneys are used they are 25 ft. high.

Top size—1 $\frac{1}{2}$  ft. dia.

Lower part of the chimney is generally made of 12 gauge sheet and can be used for two kilns, while upper part is generally 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—9" size.

**Materials :**

Moulding boxes	..	20 nos.
Flat wooden pieces for above		20 nos.
Sand	.. ..	100 c.ft.
Chimneys with drum and sheet		2 nos.
Feed hole covers	.. ..	36 nos.
C.I. sheets 7 ft. long	..	18 nos.
Fuel wood for first fuelling	..	10 mds.
Steam coal	" ..	8 tons
Dust coal	" ..	4 tons

Earth should be kept heaped up for seasoning for 4-5 days before moulding. 7 to 15 days are required for drying.

**Sundries :**

Cost of land ; Royalty for taking earth ; Carriage of water and cost of tube well, etc., Drums, Tins, Baskets, Rope for chimney 300 ft., Bamboos.

**Labour :**

Moulders	.. ..	5 per 1000 bricks
Loading in kiln	.. ..	3 "
Unloading and stacking	.. ..	2½ "

4 firemen, 2 labourers, 3 loaders, 1 mate and 1 Mistry, all for 3 months.

### Charcoal Burning

100 maunds of fuel-wood make about 20 to 25 maunds of charcoal. Wood is heated with a limited supply of air.

## 17. BLASTING

In small blasts 1 lb. of dynamite will loosen about 70 c.ft. of rock, and in large blasts about half of this. It will need about 7 ft. of boring,  $\frac{1}{2}$  labourer, 6 ft. of fuse and 3½ detonators. One man can bore 50 to 100 inches per day in granite and 300 to 400 inches in lime-stone.



## 18. CARRIAGE OF BUILDING MATERIALS

Material	Load for a two-bullock cart	Load for a six-wheeler motor truck
Bricks 9" ...	200 to 250 nos. ...	2000 nos.
Mangalore tiles ...	250 nos. ...	2500 nos.
Mangalore ridge tile ...	150 nos. ...	1500 nos.
Broken stone ...	12 to 15 c. ft. ...	125 c. ft.
Moorum ...	16 c. ft. ...	150 c. ft.
Earth ...	18 c. ft. ...	150 c. ft.
Gravel or Bajri ...	12 to 15 c. ft. ...	125 c. ft.
Kankar lime ...	12 c. ft. ...	150 c. ft.
Slaked lime ...	15 to 20 c. ft. ...	150 c. ft.
Stone boulders ...	10 c. ft. ...	150 c. ft.
Iron ...	12 cwts. ...	6 tons

*Note:*

(i) The loads given are only approximate and will vary according to the carriage capacity of the vehicle.

(ii) The average load for a two-bullock cart may be taken about 1400 lbs. for short distances and about 1000 lbs. for long distance (say above 5 miles).

(iii) For motor trucks the load should be according to the carriage capacity as approved by the Transport authorities. Loading, unloading plus traction time should be taken for calculating number of trips.

Distance and trips a bullock cart makes in a day:—

Distance	No. of trips	Distance	No. of trips
Up to $\frac{1}{2}$ mile	8	$1\frac{1}{2}$ to $2\frac{1}{2}$ miles	2
$\frac{1}{2}$ to 1 mile	7	$2\frac{1}{2}$ to $3\frac{1}{2}$ miles	$1\frac{1}{2}$
$\frac{1}{2}$ to 1 mile	5	$3\frac{1}{2}$ to $4\frac{1}{2}$ miles	$1\frac{1}{2}$
1 to $1\frac{1}{2}$ miles	4	$4\frac{1}{2}$ to $5\frac{1}{2}$ miles	$1\frac{1}{2}$
$1\frac{1}{2}$ to $1\frac{1}{2}$ miles	3	$5\frac{1}{2}$ to 8 miles	1

A truck takes about one hour to load bricks and  $\frac{1}{2}$  hour to unload with 6 labourers on job.

## 19. 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 execution 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 carefully 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, hutments for workmen, pumping of water, etc.

Make the usual provision of 5 per cent for contingencies on the estimated cost of the works for unforeseen items.



Provision for workchARGE or supervisory establishment and tools and plants.

Inclusion of lump sum provisions should be depreciated and costs should be worked out in detail as far as practicable.

Include for any tempy. works or surveys 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 inch to 1 mile will be required for most of the projects.

(Further details are given at the end of each Section.)

## 20. DEPRECIATIONS ON SCHEMES

Life of machinery and buildings is generally taken for the following numbers of years:—

Buildings	50 to 60 years	Motors; Pumps;	
Plumbing:		Generators; Boilers	
Bath tubs, lavatory			20 years
fittings, toilet bowls,		Well pumps	25 years
etc.,	25 years	Elevators	25 years
Water supply C.I. mains:		Wiring	25 years
4" dia.	50 years	Fans	15 years
6" dia.	65 years	Elec. Heaters	10 years
8" to 10" dia.	75 years	Concrete mixers	5 years
12" and over	100 years		

**Annuity, Sinking Fund, Amortization, Depreciation**

Costs of engineering works are sometimes paid in instalments. Equal periodical payments or appropriations, which are allowed to accumulate, each earning its own interest, usually compound, are called *Annuities*. Thus, a sum of money is set aside annually to accumulate compound interest and thus form a *Sinking Fund* in order to extinguish a debt. This process is called *Amortization*.

In estimating the operating expenses of engineering works, an allowance is made for *Depreciation*. In calculating this allowance, we estimate or assume the life-time,  $n$ , of the plant, and find that annuity,  $p$ , which at an assumed rate,  $r$ , of compound interest will, in the time  $n$ , amount to the cost of the plant, and thus provide a fund by means of which the plant may be replaced when worn out or superseded.

The present worth, present value, or capitalization,  $w$ , of an annuity,  $p$ , for a given number,  $n$ , of years, is that sum which, if now placed at compound interest at the assumed rate,  $r$ , will, at the end of that time, reach the same amount,  $A$ , as will be reached by that annuity.

**TABLE I. Present value  $w$ , of Annuity of Rs. 1000/-**

Years	Rate of interest (compound)					
	2%	3%	3½%	4%	5%	6%
5	4646	4580	4515	4452	4329	4212
10	8752	8530	8316	8111	7722	7360
15	12381	11938	11517	11118	10380	9712
20	15589	14877	14212	13590	12462	11470
25	18424	17413	16482	15622	14094	12783
30	20930	19600	18392	17292	15372	13765
35	23145	21487	20000	18664	16374	14498
40	25103	23115	21355	19793	17159	15046
45	26833	24519	22495	20720	17774	15456
50	28362	25730	23456	21482	18256	15762



TABLE II. **Annuity Required to Redeem Rs. 1000/-**

(The annuity which, in n years, will amount to Rs. 1000/-) :—

5	190.24	188.36	186.49	184.63	180.98	177.39
10	89.25	87.23	85.24	83.29	79.50	75.87
15	55.77	53.77	51.82	49.94	46.34	42.96
20	39.14	37.22	35.36	33.58	30.24	27.18
25	29.27	27.43	25.67	24.01	20.95	18.23
30	22.78	21.02	19.37	17.83	15.05	12.65
35	18.20	16.54	15.00	13.58	11.07	8.97
40	14.84	13.26	11.83	10.52	8.28	6.46
45	12.27	10.79	9.45	8.26	6.26	4.70
50	10.26	8.87	7.63	6.55	4.78	3.44

Comparative costs are also worked out on the basis of **Capital Recovery Factors** to compare the economics of different types of works. The true cost of work is not only the initial cost of a construction but also the expenditure to be incurred for its maintenance or essential heavy repairs or renewals and the interest on capital invested. This is done by assigning the initial cost of construction as annual charges over an assumed period of useful life. The first cost of a work is converted into annual costs by the formula :—

$$R = P \left[ \frac{r(1+r)^n}{(1+r)^n - 1} \right]$$

Where : R=annual charges ; P=first cost ; r=rate of interest per annum ; n=number of years of useful life. The capital recovery factor is a constant for a fixed rate of interest and a pre-determined life. The costs of maintenance and renewals etc. are all converted into present value. The product of these present values including the first cost and the capital recovery factor gives the annual cost.

Capital Recovery Factors for Various Lives and Interest Rates, per 1000

Years	0%	2%	3%	4%	5%	6%	8%
5	200.00	212.16	218.35	224.63	230.97	237.40	250.46
10	100.00	111.33	117.23	123.29	129.50	135.87	149.03
15	66.67	77.83	83.77	89.94	96.34	102.96	116.83
20	50.00	61.16	67.22	73.58	80.24	87.18	101.85
25	40.00	51.22	57.43	64.01	70.95	78.23	93.68
30	33.33	44.65	51.02	57.83	65.05	72.65	88.83
35	28.57	40.00	46.54	53.58	61.07	68.97	85.80
40	25.00	36.56	43.26	50.52	58.28	66.46	83.86
50	20.00	31.82	38.87	46.55	54.78	63.44	81.74



## SECTION 21

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## Athletic Fields

### Billiard :

Size of Table	Size of Room
8' × 4'	17' × 13'
10' × 5'	20' × 15'
12' × 6'	24' × 18'

### Croquet Lawn :

Size of Room 105' × 84'

### Ball Room :

Allow a minimum floor area of 16 sq. ft. for each person exclusive of ante room.

### Deck Tennis :

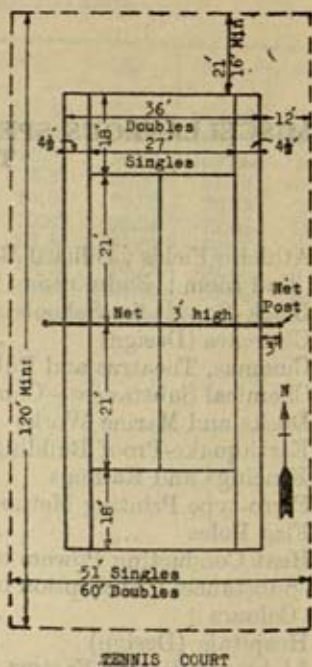
Singles size	40' × 12'
Doubles size	40' × 18'

### Badminton :

Singles size	44' × 17'
Doubles size	44' × 20'

### Basket Ball :

Max. court	94' × 50'
Ideal court	90' × 50'
For Schools	84' × 50'
	74' × 42'
	60' × 35'



TENNIS COURT

## Black-Boards for Schools

### Heights :

Primary class rooms	1'—9"	from floor to base of
Elementary class room	2'—1"	black-board
High school class room	2'—6"	

Height of Black-boards 3'—6", 4'—0", 4'—6".  
4'—0" is best.

Specifications :—Under coat of  $\frac{1}{2}$ " cement plaster of 1 cement, 2 sand and 1 charcoal powder. Finishing coat of 1 cement and 1 charcoal,  $\frac{1}{8}$ " thick.

Paint : Dissolve 1 lb. of shellac in 1 gallon of methylated spirit and add 1 lb. of ivory black,  $2\frac{1}{2}$  ounces of finest flour emery and  $\frac{1}{2}$  lb. of ultramarine blue. Mix and put in stoppered bottles. Shake well when using.



## Churches

### Size :

Accommodation including passages, communion table etc., for every person	5 to 7 sq. ft.
Width of pews	33" to 36"
Length of seats for each person	18" to 24"
Height of seats from floor	18"
Width of seats	13" to 15"
Book boards, height from floor	32"

## Cinemas, Theatres and Public Assembly Halls

### Design Requirements

(i) For approximate seating capacity in cinema halls allow 6 to 7 sq. ft. per person inclusive of passages with a minimum of 5 sq. ft. per person exclusive of passages when seats are arranged in straight rows. This will be 7 to 8 sq. ft. 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, 6 sq. ft. per sitting will usually be sufficient allowance.

(ii) The area of all doors, windows and ventilators abutting on an open air space shall not be less than  $\frac{1}{4}$ th 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. (At some places, a max : of 1 in 10 is followed.)

(iv) The clear distance between the backs of two successive seats should be 3 ft. for seats with fixed backs and 2'—9" for seats with rocking backs. (In Europe, 2'—6" is taken for seats in auditorium and 2'—8" for seats in the balcony). The centre to centre of the seats should be not less than 1'—6" for chairs without arms and 1'—8" for chairs with arms. Min. width for chairs with arms is 19 inches.

Where chairs are set on steps in balconies, the steps are made 2'—4" to 2'—8" wide and 1'—4" to 1'—8" high.

Front and rear steps are made 3 ft. wide, and the front step may be lesser in height. These steps are sometimes made 3 ft. wide and 1'—2" high.

(v) Gangways and passages should not be more than 20 ft. apart (26 ft. is prescribed by some authorities in Europe) and no seat to be more than 10 ft. from a gangway or passage (14 ft. max :).

(vi) A gangway or passage must be at least 4 ft. wide (3'-6" clear min : )

(vii) The height of the balcony ceiling or the gallery should not be less than 10 ft. from the floor of the auditorium and the depth under the balcony should not be more than 3 times the clear height.

(viii) The maximum height of the roof or ceiling at the highest step of the balcony shall be 10 ft. and at no place the distance between the nosing (top of step) and the lowest projection ray shall be less than 8 ft.

(ix) In the case of a cinema the farthest seat shall not be more than 150 ft. away from the screen. Farthest seat for natural voice is 80 ft.

(x) The angle of seating shall not be less than  $60^\circ$  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 max. angle of the line of vision from the front seat (eye level taken at 3'-6" from ground level) to the top of the screen shall not exceed  $35^\circ$ . The level of stage is kept 3 to 4 ft. from the lowest floor level.

(xii) 1 w.c. for every 200 seats and 1 urinal for every 100 seats or part thereof shall be provided for males and equal number of w.cs. for females.

(xiii) No corridor leading to any staircase or exist passage shall be less than 5 ft. in width.

(xiv) Entrance and exit doors shall be provided of size not less than 5 ft.  $\times$  8 ft. at the rate of one door for every 200 persons and one for any less number in excess. All such doors shall open outside.



(xv) 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 5 ft. 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 12" and rise not more than  $7\frac{1}{2}$ ".

### Common Names of Chemical Substances

Aqua Fortis ..	.. Nitric Acid
Blue Vitriol ..	.. Sulphate of Copper
Chalk ..	.. Carbonate of Calcium
Common Salt ..	.. Chloride of Sodium
Green Vitriol ..	.. Sulphate of Iron
Galena ..	.. Sulphate of Lead
Lime ..	.. Calcium Oxide
Nitre or Saltpeter ..	.. Nitrate of Potash
Oil of Vitriol ..	.. Sulphuric Acid
Plaster of Paris ..	.. Sulphate of Lime
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)
White Presipitate ..	.. Ammoniated Mercury
White Vitriol ..	.. Sulphate of Zinc

### Docks and Marine Works

*Dock* : An artificial basin for the use of vessels. The term is often improperly applied to a "pier" or a "marginal wharf".

*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 wharf.

*Slip*—Space between two piers; sometimes called "dock".

### **Breakwaters :** (Sea and River Protection Works)

Breakwaters are a sort of "bunds" 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 degrees. 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 levels 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 lb. weight to be about 3 per cent and stones of 50 lbs. to be not more than 12 per cent, and 1 ton 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  $1\frac{1}{2}$  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 wave-like 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 90 ft. 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, cantilevers and projections exceeding 2 ft.-6 ins. should be avoided. Chimneys should be of R.C.C. 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 R.C. 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 4 inches for single storey buildings and 8 inches 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 1 ton/sq. ft., the foundation pressure may exceed by 10 per cent, and where the bearing pressure is 2 tons/sq. ft., by 15 per cent, and where it is 4 tons/sq. ft. 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 min. allowance of 20 lbs. to the square foot.

(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 continuous-



ly 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  $\frac{1}{3}$ rd of the load for which the floor is designed with a min. of 20 lbs./sq. ft., and  $\frac{2}{3}$ rd for assembly halls, schools, and for warehouses full load should 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  $33\frac{1}{3}$  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 R.C. 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 rocky 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 superstructure. The degree of acceleration generally decreases with increasing depth below the surface.

It has been proved that a 3 ft. 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 2 tons/sq. ft. bearing value shall have footings inter-connected 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 thickness 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 considered 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, but for heavy structures it is usually more economical to adopt the inverted 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  $\frac{1}{2}$  in. dia. bars at 6 in. centres, may be provided. The cross-section of the ties or footings should be 12 ins. by 12 ins. (minimum : 8 ins. by 8 ins.) when built of reinforced concrete and should be reinforced with at least four  $\frac{1}{2}$  in. (min :  $\frac{3}{8}$  in.) dia. rods, two in the top and two in the bottom, one at each external angle, with not less than  $\frac{1}{4}$  in. round stirrups spaced not more than 12 ins. apart. If the width of the footing exceed 12 ins. one similar rod should be added at the bottom for each additional 6 ins. width or part thereof of the footings. The reinforcement should have covering of 2 ins. 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 reinforcements 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 6 in. 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 12 ins. 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 1 ft. 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 level 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 R.C. 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 6 to 12 ins. 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 6 ins. (prefer 9 ins.) 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  $\frac{1}{2}$  in. dia. for 9 ins. walls,  $\frac{5}{8}$  in. dia. for  $13\frac{1}{2}$  ins. walls, and  $\frac{3}{4}$  in. dia. for 18 ins. walls, with  $\frac{1}{4}$  in. stirrups 12 ins. centres.

The height of a masonry wall should in no case exceed  $1\frac{1}{2}$  times the total width of the building at its base. The total height of bearing wall should not exceed 40 ft., 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 min. thickness of walls in unframed buildings for one-storey building may be 9 ins., and for two-storey buildings  $13\frac{1}{2}$  ins. in ground floor and 9 ins. in first floor. For buildings more than one storey, the following heights should not be exceeded :

Thickness of wall	18"	$13\frac{1}{2}"$	9"
Storey height	15'	12'	10'

In no case the storey height of an external wall should exceed 15 ft. except that for a single storey building with trussed roof where a max. height of 22 ft. may be permitted provided the wall thickness is not less than  $1/15$ th the height of the wall. No brick wall should exceed 50 ft. in length between centres of intersecting walls. Unsupported walls of more than 16 ft. length should be stiffened by reinforced brickwork or concrete columns spaced at not more than 8 ft. 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 min. : of 5 ins. A thickness up to 8 ins. 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  $\frac{3}{8}$  in. dia. bars at 12 ins. c/c and horizontal reinforcement of  $\frac{1}{2}$  in. bars at 12 ins. c/c will be adequate for practically all storey heights encountered. The thickness of concrete hollow block masonry load bearing walls shall not be less than 8 ins. for single storeyed buildings and not less than 12 ins. 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  $\frac{1}{3}$ rd of the thickness prescribed for the ex-



ternal wall into which it bonds with a min. of 9 ins. 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. R.C. slabs shall be reinforced in both directions. All roof trusses should be anchored 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 min. distance from an external corner to the side of any opening should not be less than  $\frac{1}{2}$  of the height of the opening or  $1\frac{1}{2}$  times the thickness of the 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  $\frac{2}{3}$  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 2 ins. 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 24 ins. long and  $\frac{1}{2}$  in. 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  $1\frac{1}{4}$  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 slabs 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  $\frac{1}{4}$  in. bars at every 6th course for panels in external walls and every 8th course for panels in internal walls for spans up to 12 ft. 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 stairways, 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 3,500 sq. ft. with well braced timber frames on sound foundations can be safely constructed to withstand the stresses. Such buildings should not be more than 25 ft. in height or if two storeyed, not more than 30 ft. 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 9 ins. thick on a footing of 12 ins. width for a foundation wall up to 7 ft. in height for a single storey dwelling. Where the foundation wall exceeds 7 ft. in height,  $4\frac{1}{2}$  ins. should be added to the wall thickness for every 7 ft. or part thereof, with a proportional increase in the footings. If the foundation wall is of reinforced concrete, the thickness may be reduced to 6 ins. All corners of walls should be diagonally braced and if the length exceeds 25 ft. 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.

### **Fencings and Railings**

(See also pages 4/26, 27.)

Wires used for fencings are of different gauges, up to 14 S. W. G.. Usually they are of 4, 5 and 6 gauge in 7 strand (or 7 ply), or 4 point—3" apart barbed wire "thickest." A coil of wire as usually supplied is  $1\frac{1}{2}$  cwts. or about 450 yds. of 7-strand 14 gauge wire.

Ordinary wire posts are provided 12 ft. apart or 470 per mile, and stiffening posts 50 per mile.

The fencing posts may be of the following sizes :

**Line Posts :**

Pipe—2" dia. ; or L— $1\frac{1}{2}" \times 1\frac{1}{2}" \times \frac{1}{2}"$  or  $2" \times 2" \times \frac{1}{2}"$  ; or T— $1\frac{1}{2}" \times 1\frac{1}{2}" \times \frac{1}{2}"$

**Terminal Posts :**

Pipe—2 $\frac{1}{2}"$  dia. or L— $2\frac{1}{2}" \times 2\frac{1}{2}" \times \frac{1}{2}"$  with two stays  $1\frac{1}{2}" \times 1\frac{1}{2}" \times \frac{1}{2}"$  L for corner posts and one stay for end post.

**Gate Posts :** Not over 6 ft. wide single or 12 ft. wide double gate opening :

Pipe—3" dia. or L— $3" \times 2\frac{1}{2}" \times \frac{1}{2}"$  or T— $2\frac{1}{2}" \times 2\frac{1}{2}" \times \frac{1}{2}"$

Over 6 ft. wide to 13 ft. wide single or 12 ft. to 26 ft. wide double gate opening :

Pipe—4" dia. or L— $5" \times 3" \times \frac{3}{8}"$  or T— $3" \times 3" \times \frac{1}{2}"$

Wires are fixed to end posts by straining bolts either with an hooked or eyed end, complete with washer and nut. The sizes of such bolts are  $9" \times \frac{3}{8}"$ ,  $12" \times \frac{3}{8}"$  or  $\frac{1}{2}"$ ,  $15" \times \frac{1}{2}"$ ,  $18" \times \frac{3}{8}"$  or  $\frac{1}{2}"$  or  $\frac{3}{4}"$ . Wires are fixed to line posts by gal. staples of size  $\frac{1}{2}"$  to  $1\frac{1}{2}"$ .

For more details and also for R.C. Fence Posts see Section 8. See also under "Bridges".

**Railing pipes for Embankments, etc.**

To consists of galv. W.I. gas pipes from  $\frac{3}{4}$  to 1 inch diameter, in 4 or 5 rows 1 ft. apart. The height above the ground level is generally from 3' to 4'-6". Posts are 6 to 8 ft. apart ; they are sunk into the ground from 1'-6" to 2'-6" and fixed in lime concrete foundations. Posts can be of Ts  $3" \times 3" \times \frac{3}{8}"$  (also called stumps or standards.) The foundations block of concrete can be of size  $1' \times 1' \times 1\frac{1}{2}'$  high. Posts at angles can be strutted by bending the second pipe from the bottom and carrying it into the lime concrete block at the base which will be increased in size by about 2 ft.

**Ferro-Type Printing (Blue Prints)**

Method of sensitizing paper :

- (i) Citerate of Iron and Ammonia  
Water ... ..

100 grs.  
1 oz.



(ii) Ferric cyanide of Potassium*			
(Red Prussiate of Potash)	...	...	70 grs.
Water	...	...	1 oz.

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) paper and done twice crossways. The paper is hung up to dry. The process should be carried out in a dark room.

About 4 ounces of the solution will suffice for coating 100 sq. ft. 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  $2\frac{1}{2}$  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 bring-

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\*and not the ferrocyanide of yellow prussiate.

ing 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	150 grs.
Gum solution	40 min.
Water	1 oz.

### Flag-Poles—Wooden

For 30 ft. to 60 ft. extending above the roof—diameter at the roof should be  $1/50$  of the height above the roof and top diameter  $\frac{1}{2}$  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 for Wards :* (See also under "Siting of Buildings and Ventilation" Section 7.)

Allow 8'-8" spacings between wall and passage for bed. Allow 10'-6" min. 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-paper	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 : 13' (prefer 16')  $\times$  9'  
 Usual size : 18' to 20'  $\times$  10' to 12' } Floor slope 1 in 36  
 Height : 7' to 9'

### Size of Motor Pit :

Length 12' } Steps can be made on one side.  
 Breadth 2'-9" } Bottom drain should be provided  
 Depth 4'-0" } if possible.  
 Planks for covering 1½" thick  
 Approach Ramp 1 in 8 to 1 in 10

(See under "Motor-Vehicles" for sizes of cars in the Section on "Roads and Highways.")

### Papers

#### Size of Drawing Papers :

	<i>Ins.</i>		<i>Ins.</i>
Antiquarian	52 $\times$ 31	Emperor	72 $\times$ 48
Atlas	34 $\times$ 26	Imperial	30 $\times$ 22
Columbier	34 $\times$ 23	Double Imperial	45 $\times$ 29
Demy	20 $\times$ 15	Medium	22 $\times$ 17
Double Elephant	40 $\times$ 27	Royal	24 $\times$ 19
Elephant	28 $\times$ 23	Super Royal	27 $\times$ 19

*Tracing Cloth*—Width : 18, 28, 30, 36, 38, 41, 42 inches  $\times$  24 yards long. Continuous Cartridge, 54 and 60 inches wide.

#### Printed Book Sizes :

Book Size	Dimensions in ins.
Foolscap (F <sup>o</sup> cap) 8 vo. ... ..	6½ $\times$ 4½
Crown 8 vo. ... ..	7½ $\times$ 5
Large crown 8 vo. ... ..	8 $\times$ 5½
Demy 8 vo. ... ..	8½ $\times$ 5½
Medium 8 vo. ... ..	9 $\times$ 5½
Royal 8 vo. ... ..	10 $\times$ 6½
Crown 4 to ... ..	10 $\times$ 7½
Demy 4 to ... ..	11½ $\times$ 8½
Royal 4 to ... ..	12½ $\times$ 10



24 or 25 sheets of paper=1 quire ;  
20 quires=1 ream=480 or 500 sheets.

### Riveting

Tables for Rivets are given in Section 4.

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.

Max. working stresses on bolts and rivets :

*For parts in Shear* : (Gross area of rivets and bolts)

On power driven (or shop) rivets  
and tight fitting turned bolts 6 tons/sq. in.

On hand driven (or field) rivets  
or bolts .. .. 5 "

On black bolts .. .. 4 "

The strength of rivets and bolts in double shear may be taken as twice that for single shear. By some engineers, in double shear the effective section of each rivet is reckoned at  $1\frac{1}{2}$  times the section area of the rivet.

*For parts in Bearing* : (Gross dia. of rivets and bolts)

On power driven rivets and tight  
fitting turned bolts ... .. 12 tons/sq. in.

On hand driven rivets or bolts ... .. 10 "

On black bolts ... .. 8 "

*For parts in Tension* : (Gross area of rivets and net area of bolts)

On power driven (or shop) rivets... 5 tons/sq. in.

On hand driven (or field) rivets ... 4 "

On bolts  $\frac{3}{4}$  in. and over in dia. ... 6 "

On bolts less than  $\frac{3}{4}$  in. dia. ... 5 "

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\*These stresses are not intended to apply to screwed tension rods.

The gross area of a rivet is the cross-sectional area of the finished rivet after driving. The net sectional area of a bolt is the area at the root of the thread.

It will be seen from above that the working stress of hand driven rivets is only 4/5th of the machine driven rivets.

Riveted joints may fail in four ways :

- (i) Shearing—single or double shear :

$$\begin{aligned} F_s &= f_s \pi r^2 & \dots & \text{single shear} \\ &= f_s 2\pi r^2 & \dots & \text{double shear} \end{aligned}$$

- (ii) Bearing or crushing : This is usually due to the plates being too thin or the dia. of the rivets being too small :

$$F_b = f_b dt$$

- (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 at 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  $\frac{5}{8}$ th that of the main plate to be jointed. In single cover, the cover plate is  $1\frac{1}{4}t$  or  $1\frac{1}{2}t$ .

Where several thicknesses 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 plate.

### *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  $d = 1.27 t$

Also, when  $t$  is less than  $\frac{1}{4}$ ",  $d = 2t$ , and when it is equal to or greater than  $\frac{1}{4}$ ",  $d = 1\frac{1}{4}t$ .  $t$  is the thinnest plate.

In plates  $\frac{1}{4}$ " to  $\frac{1}{2}$ " thick,  $\frac{1}{4}$ " to  $\frac{3}{4}$ " diameter rivets, and in plates  $\frac{3}{4}$ " to  $1\frac{1}{2}$ " thick,  $\frac{3}{4}$ " and  $1\frac{1}{2}$ " diameter rivets are used. Some engineers use a dia. of  $\frac{3}{4}$ " for a  $\frac{3}{4}$ " plate,  $\frac{7}{8}$ " for a  $\frac{1}{2}$ " plate,  $1\frac{1}{8}$ " for a  $\frac{3}{8}$ " plate. The max: size of the rivet will also depend upon the minimum width of the member. In plate girders, diameter of the rivets is taken as  $\frac{7}{8}$ " for all flanges over 6" in width and  $\frac{3}{4}$ " for all flanges 6" or less in width.

### *Pitch of Rivets :*

Number of rivets required =  $C/R$  or  $T/R$ , where  $C$  or  $T$  are direct compressive or tensional loads in tons and  $R$  is the resistance of a rivet.

Rivets should not be closer together centre to centre than 3 diameters (prefer  $3\frac{1}{2}$  dia.), (min: pitch is  $2\frac{1}{2}d$  for punched and  $2d$  for drilled holes), nor further apart than 16 times the thickness of the thinnest outside plate, or 8" for tension parts and 6" for compression parts except that, where two or more rows of staggered rivets occur in one leg of an angle, the straight line pitch may be increased to  $1\frac{1}{2}$  times the above values. In staggered riveting the distance between the transverse pitch lines should be at least  $\frac{2}{3}$ rd the transverse pitch of the rivets.

The minimum distance from the centre of a rivet to a planed or rolled edge should be  $1\frac{1}{2}$  times the diameter of the rivet for drilled holes and 2 times for punched holes ; or according to BSS 449 :—

*Rolled sawn or Planed edge*

$1\frac{3}{4}"$ for $1\frac{1}{8}"$ dia. rivet
$1\frac{1}{2}"$ for $1"$ dia. rivet
$1\frac{1}{4}"$ for $\frac{7}{8}"$ dia. rivet
$1\frac{1}{8}"$ for $\frac{3}{4}"$ dia. rivet
$1"$ for $\frac{5}{8}"$ dia. rivet
$\frac{3}{4}"$ for $\frac{1}{2}"$ dia. rivet

*Sheared edge*

$2"$ for $1\frac{1}{8}"$ dia. rivet
$1\frac{3}{4}"$ for $1"$ dia. rivet
$1\frac{1}{2}"$ for $\frac{7}{8}"$ dia. rivet
$1\frac{1}{4}"$ for $\frac{3}{4}"$ dia. rivet
$1\frac{1}{8}"$ for $\frac{5}{8}"$ dia. rivet
$1"$ for $\frac{1}{2}"$ dia. rivet

The lengths of rivet shanks are taken as follows :

For snap heads

$$L = t + t/8 + 1\frac{1}{2}d$$

For countersunk heads

$$L = t + t/8 + d$$

Deduction for rivet holes :

In tension members an area of  $t(d + \frac{1}{8})$  for hand driven rivets and  $t(d + \frac{1}{16})$  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, tons/sq. in.,

$f_b$  = allowable bearing stress in plate, tons/sq. in.,

$f_t$  = allowable tensile stress in plate, tons/sq. in.,

$d$  = dia. of rivet,

$t$  = thickness of the thinnest plate transmitting the full load,

$p$  = pitch of rivets,

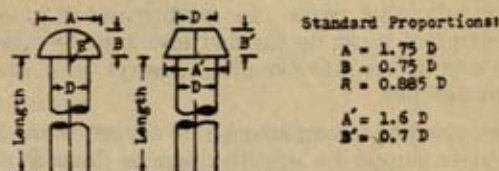
$n$  = no. of rivets per side of joint,

$b$  = breadth of the plates.

Rivet holes are made in three ways—Punching, Drilling, Punching and Reaming. Punching slightly injures the plates immediately about the hole and there is also possibility of inaccuracies. The punching process is frequently limited to holes of a max. of  $\frac{3}{4}"$  dia. In the third method the holes are punched about  $\frac{1}{4}"$  less than the dimension required and then the holes are enlarged with a drill to its correct dimensions. The holes are usually made  $\frac{1}{16}"$  larger than the shank of the rivet to be em-



ployed. In all cases where the total thickness to be riveted is  $3\frac{1}{2}$ " or over, the rivet holes should not be punched but should be drilled, and further, must be drilled out in place to finished diameter after the work is assembled. The work must be taken apart after drilling, and all burrs left by the drill completely removed.



Punching must be accurately done. The diameter of the punch for material not over  $\frac{3}{4}$ " thick must not be more than  $\frac{1}{16}$ " nor that of the die more than  $\frac{1}{8}$ " larger than the diameter of the rivet. The diameter of the die must not exceed that of the punch by more than  $\frac{1}{4}$ th the thickness of the metal punched. When general reaming is required, punched holes will be made with a punch  $\frac{3}{16}$ " smaller in diameter than the normal size of the rivets and will be reamed to a finished diameter of not more than  $\frac{1}{16}$ " larger than the rivet.

Rivets shall be properly heated to straw heat for the full length of the shank and quickly driven in the hole. The head of the rivet, particularly in long rivets, shall be heated more than the point and in no case shall the point be heated more than the head. Sparkling or burnt rivets must not be used. Much overheating is detected by the burnt scaly appearance of the surface of shank;  $\frac{3}{4}$ " or even 1" 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 rivets. Riveting hammers vary from 2 to 7 lbs, the holding up hammer or "dolly" from 10 to 40 lbs. 90 to 100 rivets can be put in by one gang in one working day of 9 hours.

Loose rivets can be detected by holding a finger on one head and tapping the other with a small hammer of 8 oz. 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. As a rivet shrinks in cooling, a slight vibration is not a cause for condemning a rivet. 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 cut off sideways and the rivet shank punched out.

Before riveting is commenced, every alternate hole in the joint plate should be tightly service bolted so as to ensure tight riveting. The service bolts should be re-tightened frequently as the riveting proceeds. Field riveting requires more care than shop riveting.

In important places where bolts have to be used instead of rivets, the nuts should be screwed home as tight as possible and the screw end riveted cold by hammering threads into deformation, (or burred) which prevents the nut from working loose through possible vibrations. Lock-nuts and spring washers can also be used.

For rivets of all diameters of  $\frac{1}{2}$ " and upwards, the diameter of the rivet before being heated should not be more than  $\frac{1}{16}$ " less than the diameter of the hole it is intended to fill.

All joints of riveted tanks shall be made water or oil tight by caulking only. No foreign substance shall be used in the joints.

### Ropes

The most pliable rope for hoisting or transmission contains 19 wires to the strand, while ropes of 12 or 7 wires to the strand are better adapted for use as standing ropes, guys, or rigging. The flexibility of wire rope depends largely on the size of the individual wires used in making up the strands of the rope, the smaller these wires (and



consequently the more numerous) the more flexible the rope.

Galvanized wire rope for rigging is more durable than hemp rope and does not stretch permanently under great strains. Its bulk is one-sixth and its weight one-half that of hemp rope. Crucible cast-steel wire ropes are much more durable than iron ones. They should be kept well lubricated.

### **Preservation of Ropes**

To preserve wire ropes, apply raw linseed oil (which may be mixed with an equal quantity of lamp black). Painting with equal parts of beeswax and resin melted together, will also preserve ordinary ropes. If for use in water or underground, add some fresh slaked lime to coal-tar and boil it well, saturate the rope with it while hot. Wire ropes when in use should be greased frequently. Hemp ropes can be preserved by dipping when dry into a solution of 1 oz. of sulphate of copper dissolved in  $2\frac{1}{2}$  pints of water and kept in this solution for four days and afterwards dried; the ropes should then be soaked in a solution of 5 ozs. of soap dissolved in  $2\frac{1}{2}$  pints of water and again dried.

### **Hemp or Fibre Ropes**

Weight, roughly :  $3\cdot0 \times c^2$  lbs. per 100 ft. length

Breaking load in tons, roughly :  $\frac{c^2}{20}$

$c$  is circumference of rope in inches.

Strength of ropes vary greatly; pieces from the same coil may vary 25 per cent. Good Italian hemp is considered best. The tarring of ropes is said to lessen their strength; and when exposed to weather, their durability also. A few months of exposed work weakens ropes 20 to 50 per cent.

### **Wire ropes**

Approx. breaking load for flexible steel wire ropes, in tons =  $2\cdot5 c^2$ ; for iron ropes =  $1\cdot6 c^2$ ; for plow steel ropes =  $3\cdot75 c^2$ .  $c$  is circumference of rope in inches.

Working load should be only 10 to 25 per cent. of the breaking load. Fastening reduces the working load 60 to 75 per cent.

## **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 10 sq. ft. average for secondary schools and 20 sq. ft. for higher classes. The height of the rooms should be 12 ft. minimum and if the floor area exceeds 600 sq. ft., the height should be 14 ft.

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  $\frac{1}{4}$ th 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 3 ft. from the west wall. No windows admitting light should be placed so that pupils when seated will be facing them. The light should preferably come from the left of the pupils. In rooms where pupils are seated on desks, the height of the window sill should be  $3\frac{1}{2}$  ft. to 4 ft., but where the pupils are seated on the floor, the height of the window sill should be  $2\frac{1}{2}$  ft. to 3 ft. 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 24 ft. from the window wall.

Ventilators should be fixed as near as possible of the ceiling (6" to 9") and allow an area of 50 sq. inches 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 30 ft. distant from the blackboard. Distance between rows of seats should be 2'-6".

For sanitary arrangements see "Standards for Public Sanitary Conveniences" in Section 16.

### **Hostels**

In the case of single rooms or cubicles the minimum floor space should be 96 sq. ft. Rooms for accommodating 3 or 4 students should provide 65 sq. feet of floor space per head and those for 5 or more students, a minimum of 60 sq. feet per head.

### **Steel Structures (General)**

*Bars*—Rounds, Squares, Flats etc., special sections and small shapes are classified as bars. Angles, Channels, Tees are "bar" size when their greatest dimension is under 3 inches. Flats are classified as bars when they are 6 inches or under in width and 0.25 inch or over in thickness.

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-quarter 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  $\frac{1}{4}$  in. 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 staved up in a die and afterwards annealed.

**Weights :**

Av. weight of a structure = nominal wt. of section + 10 per cent for cleats, rivets, bolts, etc.

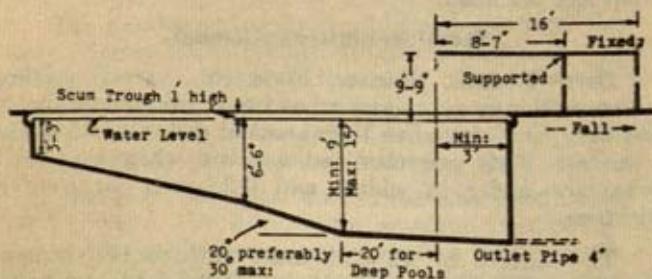
(10 per cent. is taken for wastage for bolts, nuts, washers, and rivets, etc., for field works.)

Av. weight of a beam = wt. of section including cleats + 2½ per cent. for rivets.

Av. weight of stanchions = ditto. + 5 per cent.

Av. weight of plate girder = ditto. + 10 per cent.

Weight of steel stairs 3 ft. wide, industrial type = 1 cwt. per ft. run.

**Swimming Pools**

Size	Depth	Suitable for
50' × 25'	Max. 3'—6", Min. 2'—6" ...	Schools (1'—6" depth for children)
75' × 30'	6' to 7' deep at one end and 3'—6" deep at the other end	Average capacity pools
100' × 42'	Ditto. ...	Swimming association
132' × 48'	Ditto. ...	Contests and races
165' × 60'	6' min. depth and 9' min. for diving.	Championship

Water area may be taken at 27 to 36 sq. ft. per each person using the tank. Or, 12 × 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 mini-



num of 9 ft. Copper railings  $1\frac{1}{2}$ " diameter to be provided all round.

Expansion joints are provided at about 40 ft. spacing.

*Spring Boards :*

Are generally 1'—8" wide and fixed not less than 6 ft. apart and not more than 9'—9" above water level, except for out-door pools which have boards as high as 32 ft. The board should project 3 ft. minimum from edge of tank. Where a number of boards are provided one above another, each should project at least 2 ft. further than the one next below it. The international one meter spring board is 14 ft. long and 3 meter board is 16 ft. long. Fixed running boards should be 14 to 18 ft. long or more and should give a clear run of at least 16 ft. The height of the diving board should not exceed twice the depth of the water. The ladder shall have a max : of 5 steps.

*Toilet Arrangements :*

*Lavatories :*

- 1 w.c. and 1 urinal for every 60 men
- 1 w.c. for every 40 women
- 1 basin every 60 bathers

*Dressing Boxes :*

3'—6"  $\times$  3' to 4'  $\times$  4' 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 w.cs.  | 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".)

Walls of the tanks should be designed to resist earth pressure with live load surcharge of 1 ft. Where tanks are built above ground level, there should be enough of earth well rammed all round. Top width of walls to be 1'—6".

Foundations of the long walls should preferably be stepped to make up for the sloping floor. Floor can be of 6" cement concrete over 6" 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  $\frac{3}{4}$ " filled with bitumen. Scum trough should extend 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 8 ft. (min.) width all round the pool for open air pools and given outward fall of 1 in 40 ; or, alternatively, a kerb 9" to 12" high can be constructed at the edge of the pool in order to prevent surface water draining back into the pool.

### Surveying

Table of Angles Corresponding to Openings of a 2-foot Rule

In.	Deg.-Min	In.	Deg.-Min.	In.	Deg.-Min.	In.	Deg.-Min.
$\frac{1}{4}$	1—12	3	14—22	$5\frac{3}{4}$	27—44	$8\frac{1}{2}$	41—29
$\frac{1}{2}$	2—24	$3\frac{1}{2}$	15—34	6	28—59	$8\frac{3}{4}$	42—46
$\frac{3}{4}$	3—36	$3\frac{1}{2}$	16—46	$6\frac{1}{2}$	30—11	9	44—3
1	4—47	$3\frac{1}{2}$	17—59	$6\frac{1}{2}$	31—26	$9\frac{1}{4}$	45—21
$1\frac{1}{4}$	5—58	4	19—12	$6\frac{3}{4}$	32—40	$9\frac{1}{2}$	46—38
$1\frac{1}{2}$	7—10	$4\frac{1}{2}$	20—24	7	33—54	$9\frac{3}{4}$	47—56
$1\frac{3}{4}$	8—22	$4\frac{1}{2}$	21—37	$7\frac{1}{2}$	35—10	10	49—15
2	9—34	$4\frac{3}{4}$	22—50	$7\frac{1}{2}$	36—25	$10\frac{1}{4}$	50—34
$2\frac{1}{4}$	10—46	5	24—3	$7\frac{3}{4}$	37—41	$10\frac{1}{2}$	51—53
$2\frac{1}{2}$	11—58	$5\frac{1}{2}$	25—16	8	38—57	$10\frac{3}{4}$	52—33
$2\frac{3}{4}$	13—10	$5\frac{1}{2}$	26—30	$8\frac{1}{2}$	40—13	...	...



### Chaining Over Sloping Grounds

Table of Deductions or Additions to be made  
per 100 feet Chaining over Sloping Grounds

Slope in degrees	Deduction in feet	Rise in feet per 100 ft. horizontal	Slope in degrees	Deduction in feet	Rise in feet per 100 ft. horizontal
1	·015	1·746	11	1·887	19·44
2	·061	3·492	12	2·185	21·26
3	·137	5·251	13	2·563	23·09
4	·244	6·993	14	2·970	24·93
5	·381	8·749	15	3·407	26·79
6	·548	10·51	16	3·874	28·57
7	·745	12·28	17	4·370	30·57
8	1·73	14·05	18	4·894	32·43
9	1·231	15·34	19	5·448	34·43
10	1·519	17·63	20	6·031	36·40

### Thermometers

Three thermometric scales are in use :

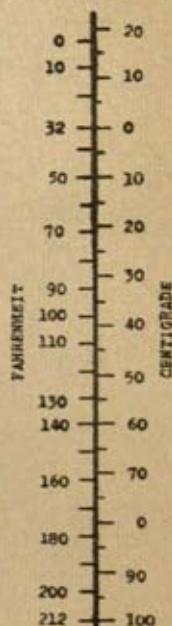
$$\begin{array}{l|l}
 C = 5/4 \ R = 5/9 \ (F - 32) & C = \text{Centigrade} \\
 & \text{(Celsius)} \\
 F = 9/5 \ C + 32 = 9/4 R + 32 & F = \text{Fahrenheit} \\
 R = 4/5 \ C = 4/9 (F - 32) & R = \text{Reaumur}
 \end{array}$$

Freezing point, or Melting point of ice  
= 32°F. = 0°C. = 0°R

Boiling point of water = 212°F.  
= 100°C. = 80°R

Human temperature = 98.4°F.  
= 37°C. = 29.5°R.

For most purposes mercury in glass  
thermometers are used. For recording the  
interior temperature of a dam, thermo-  
couples are used.



**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.

**ADDENDA**



## SECTION 22

## ELECTRIC SERVICES

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## TERMS AND MEASURES

**A.C. (Alternating Current):** Is a current which alternatively reverses its direction in a circuit in a periodic 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.

**D.C. (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  $\times$  1 amp.  $\text{watts/volts} = \text{amperes}$ .

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 (B.T.U.) 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:* (I.H.P.) 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:* (B.H.P.). 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:* (S.H.P.) Is the same as B.H.P., but is the term used for large engines where the output is measured by a torsionmeter rather than by a brake.

*Nominal Horse-power:* (N.H.P.) 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  $= \text{B.H.P.} / \text{I.H.P.}$  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 H.P.*  $= \text{I.H.P.} \times \text{engine efficiency} \times \text{generator efficiency.}$

One H.P. motor consumes about 500 watts.

One H.P.  $= 33,000 \text{ ft. lbs./min.} = 550 \text{ ft. lbs./sec.}$   
 $= 0.7457 \text{ kilowatts} = 746 \text{ watts.}$

1 kilovolt  $= 1000 \text{ volts}$

1 kilowatt  $= 1000 \text{ watts} = 1.341 \text{ H.P.} = 737 \text{ ft. lbs./sec.}$

1 kilowatt-hour  $= 1000 \text{ watt-hours} = 1.341 \text{ H.P.-hours.}$

1 millivolt  $= \text{one-thousandth of a volt.}$

1 milliamperere  $= \text{one-thousandth of an ampere.}$

1 megohm  $= \text{one million ohms.}$

- $E=IR$ : To find the voltage, multiply amperage by number of ohms resistance.
- $W=IE$ : To find number of watts, multiply voltage by amperage.
- $W=I^2R$ : 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.

In a D.C. circuit  $R=E/I$ .  
 In an A.C. circuit  $Z=E/I$  } Ohm's Law  
 $Z$  is impedance in ohms.

### Advantages of A.C. Over D.C.:

- (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.

D.C. current is necessary for some industrial processes, such as electro-plating, and also for the charging of storage batteries. D.C. is also required for cinema work. A.C. is converted to D.C. through *convertors*. Speed control is simpler with D.C. than with A.C., and D.C. motors in general operate with less noise.

A.C. can be changed to uni-directional current required for battery charging etc., by means of a *rectifier*.

### A.C. Constant-Potential Systems

There are two systems:

(i) Single-phase system. The term phase is used in connection with A.C. 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 but three-phase service is given to larger power consumers.



**Current Taken by D.C. Motors***Approx. full load current in Amperes assuming average efficiency:*

H.P.	Voltage			H.P.	Voltage		
	110	220	440		110	220	440
$\frac{1}{8}$	3.0	1.5	..	20	154	77	38
$\frac{1}{4}$	5.4	2.7	..	25	193	97	48
$\frac{1}{2}$	10	5	2	30	227	114	57
1	18	9	4	40	300	150	75
2	26	13	6	50	375	187	93
3	42	21	10	60	452	226	113
5	49	25	13	75	560	280	139
6	60	30	15	80	590	295	148
7½	80	40	20	100	740	370	195
10	95	48	24	150	..	550	275
12	117	59	29	200	..	..	365

**Current Taken by A.C. Motors***Approx. Amperes per phase taken by modern induction motors, allowing reasonable efficiencies and power factor:—*

B.H.P. of Motor	Single Phase	Three	Phase	B.H.P. of Motor	single Phase	Three	Phase
	230 Volts	400 Volts	440 Volts		230 Volts	400 Volts	440 Volts
$\frac{1}{8}$	1.8	.7	.6	5	24	8	7.5
$\frac{1}{4}$	3.5	1.2	1.0	7½	36	12	11
$\frac{1}{2}$	4.8	1.7	1.4	10	47	15	14
1	6.2	2.0	1.7	12½	59	19	18
1½	7.4	2.5	2.2	15	70	22	21
2	8.7	2.8	2.5	20	91	29	28
3	10.0	3.2	2.8	30	135	42	39
4	11.8	3.5	3.2	40	183	56	53
5	14.0	4.3	4.0	50	227	70	66
7½	17.5	5.0	4.5	75	..	104	94
10	20.0	6.5	6.0	100	..	136	125

To find the current taken by each terminal of a three-wire, three-phase A.C. motor, divide the current taken by a single-phase A.C. motor of the same size and voltage by 1.73.

### Quantity by Heat

The British Thermal Unit (B.Th.U.) is the quantity of heat required to raise the temperature of 1 lb. of water by 1 F. deg. Similarly the caloric or the gram cloric (cal.) is the quantity of heat required to raise the temperature of one gram of water by 1 C. deg.

### Power Available from Water Falls

H.P. available in kilowatts  $= Qh/12$  (or  $Qh/16$  with 75% efficiency)

$Q$ =discharge of water in cusecs,  $h$ =fall of water in ft.

**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— $\frac{1}{2}$  to  $\frac{2}{3}$  of installed load; other services — $\frac{1}{3}$  to  $\frac{1}{2}$  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 distance of about 3 ft. 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 3 to 6 inches 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 crankshaft centre to about 27 inches 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 1 ft. 3 ins. for single panel and not less than 2 ft. 6 ins. 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:* (C.P.) 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 C.P. = 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 120° F.

Apart from discharge and fluorescent tubular lamps, standard type gasfilled 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.

Radios are rated at 30 to 100 watts. Refrigerator 3 c. ft. at 150 watts and 9½ c. ft. at 250 watts.

All glow lamps may be hung at a height of 8 ft. above the floor level and fans 9 ft. 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 say. 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.	S.W.G.		
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-S.W.G.	27	23	21

Fuse wires should not be kept un-enclosed 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 60°F., to allow for the higher temperature 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 120°F., 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  $\frac{1}{3}$  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 avoid the need for separate down leads for earthing.

The buried ends of steel poles should be lapped with tarred gunny as a preservative. Special consideration must be given to the earthing of R.C. poles. In soft soils a small base plate is usually necessary under the foot of the pole.

## Weight of Single Copper Wires per 1000 yds. in lbs.

Gauge No.	S.W.G.	B.W.G.	B.&S.G.	Gauge No.	S.W.G.	B.W.G.	B.&S.G.
1	817.0	817.0	760.0	24	4.395	4.395	3.669
2	691.8	732.4	602.6	25	3.632	3.632	2.911
3	576.7	609.2	477.9	26	2.943	2.942	2.295
4	488.8	514.4	379.0	27	2.442	2.325	1.831
5	408.1	439.5	300.5	28	1.989	1.780	1.442
6	334.8	374.3	238.3	29	1.680	1.535	1.160
7	281.3	294.3	189.0	30	1.396	1.308	.9116
8	232.5	274.2	150.0	31	1.222	.9080	.7192
9	188.3	198.9	118.9	32	1.059	.7350	.5789
10	148.8	163.0	94.29	33	.9080	.5810	.4578
11	122.2	130.8	74.71	34	.7686	.4449	.3603
12	98.20	107.9	59.30	35	.6408	.2270	.2848
13	76.90	81.90	47.06	36	.5249	.1453	.2270
14	58.12	62.57	37.31	37	.4199	..	.1800
15	47.09	47.08	29.61	38	.3269	..	.1429
16	37.20	38.37	23.46	39	.2456	..	.1132
17	28.48	30.54	18.60	40	.2092	..	.0898
18	20.92	21.81	14.75	41	.1758	..	.0712
19	14.53	16.12	11.70	42	.1453	..	..
20	11.77	11.12	9.275	43	.1177	..	..
21	9.300	9.300	7.354	44	.0930	..	.0363
22	7.120	7.120	5.813	45	.0712	..	..
23	5.231	5.676	4.638				

(Wire gauges are given in Section 4.)

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 annealed at the same stage of its being drawn and reduced in section. The cold drawing increases the tensile strength but reduces elongation. The min. tensile strength of soft or annealed wire is 36000 lbs./sq. in. with 35 per cent of elongation in 10-inch length. The max. tensile strength of hard drawn copper wire is 65000 lb./sq. in.

## SYSTEMS OF WIRING &amp; THEIR SUITABILITY

**Screwed Steel Conduits.** Are made up of comparatively heavy gauge mild steel tubes of varying thicknesses. No conduit less than 6/10 in. 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 fires; 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 impossible 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 3 inches.

**C.T.S. or T.R.S.** (tough-rubber sheathed) cables will 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. T.R.S. cables have a tendency to crack and open after some time where the wires have been bent to a sharp angle.

**P.V.C.** 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 is 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 damage, or places where inflammable materials are stored. Cleated wiring is not suitable for permanent domestic installations. V.R.I. (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-sheathed 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 overheating 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, steam 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 220 ft. (I. E. Rule 67) ; 150 ft. 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 20 ft. from the ground, and in private grounds or at a consumer's premises, it shall not be less than 15 ft. In the case of outdoor sub-stations in public places, proper fencing to a height of 8 ft. and a minimum clearance to the live conductors of 5 ft. 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 S.W.G. 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 S.W.G. and they must be composed of wires twisted together on a short lay, no wire being smaller than a No. 40 S.W.G.

Where conductors pass through floors, they shall be carried in a heavy gauge insulated conduit or a porcelain tube. The floor tube shall be carried 12 inches above the floor line and 1 inch below ceiling line. Where the supply is A. C. 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, and also after erection.

Insulated wires exposed to sun and rain deteriorate quickly though the weather proof quality has over double the life of V.I.R. wires. Lime and distemper gradually ruin the insulation of V.I.R. wires and oil attacks the rubber covering of cables. P.V.C. cables stand up well to oil and petrol.

Plugs (for fixing wires) for ordinary walls or ceilings should be of well seasoned teak or other hardwood not less than 2 inches long by 1 inch square on the inner and  $\frac{3}{4}$  inch square on the outer end, except for use with metal-sheathed wiring. They should be cemented into the walls to within  $\frac{1}{4}$  inch 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  $\frac{1}{4}$  inch diameter and  $\frac{1}{8}$  inch deep. Wooden plugs used for metal-sheathed wiring should not be less than  $1\frac{1}{2}$  inches long and  $\frac{1}{4}$  inch 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 possible.

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 circuits should be balanced. Circuits 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 neutral 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 3 ft. in front, and either less than 9 inches behind or a gangway of over 30 inches width and 6 ft. 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 megohm. When PVC cables are used the values of insulation resistance may be relaxed to  $\frac{1}{2}$ .

*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 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 installation.

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 S. P. Switches*—Tests should be made to verify that all non-linked S. P. 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, Vol. 87, 1940, page 390) : Ohms.-cm.

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
Sandy 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  $0^{\circ}\text{C}$ . 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 6 ft. 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, 1933).

(i) Wet marshy ground and ground containing refuse such as ashes, cinders and brine waste. (ii) Clay, loamy soil, arable land, clayey soil and loam mixed with small quantities of sand. (iii) Clay and loam mixed with varying proportions of gravel and stones. (iv) 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  $\frac{3}{4}$  in. to  $\frac{1}{2}$  in. diameter G.I. pipe is enough, driven down to at least 5 to 6 ft.; 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 6 ft. below ground and should not also be less than 6 ft. from any building. Size need not be more than 25 to 30 sq./ft.

In rocky areas where it is not possible to drive down the earth rods to sufficient depths to good moisture level, copper or G. I. strips or wires are used in long lengths, which are buried as deep as possible (but not less than 2 ft.) G. I. strips can be about 1 in.  $\times$   $\frac{1}{8}$  in. and copper strips 1 in.  $\times$   $\frac{1}{16}$  in. 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 as 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 feet 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 G. I. bolts and nuts using G. I. washers. (Also see Lightning conductors in the Section on "Masonry Structures"). Not less than the equivalent (solid or stranded) of one No. 8 S. W. G. (No. 12 min.) copper or G. I. wire should

be used for making an earth connection with max. size up to 0.1 sq. inch. (Also see "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 100 ft. 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.  $\frac{1}{2}$  in. dia. iron rod 3 ft. 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 of 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 one mile 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 accidentally 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 flowing 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, short-circuit and over-current. (Also see further).

## 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 A. C. 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 A. C. voltages. The max. safe D. C. voltage is about 5 times that for A. C. and D. C. is less fatal than A. C. until about 1000 volts when it is more fatal. With D. C. systems there are fewer accidents with distribution voltages of about 220, but fatalities have, however, occurred with D. C. 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 accidentally 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

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\*This is based generally on "Electrical Accidents by K.V. Karantha".



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.

The min. safe clearance from power lines should be 8 ft. vertical and 4 ft. horizontal for L. T. lines and  $8\frac{1}{2}$  ft. and 5 ft. for H. T. 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 at once, 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 6 ft. of any overhead H.T. 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 discharge wires should be kept at least 2 ft. 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 inadequate size in an H. T. line ; it can have a flash-over melting the copper of the conductor at the point of flash and breaking the conductor.

(iii) Accidentally energised poles and stays due to insulated wires dangling from the lines, losing their insulation and making accidental contact with the poles or stays. 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 losing 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 straddling 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 steadily 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 unusual 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 avoided. Joints in wiring cause overheating and sometimes fire. Presence of readily combustible materials or explosive gases near "faults" are very dangerous. 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 4 ft.-6 ins. above the floor. Main switches and cut-outs of dwelling houses should not be fixed in garages. Conduit wiring and C. T. S. 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 occurred 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.

## THE END



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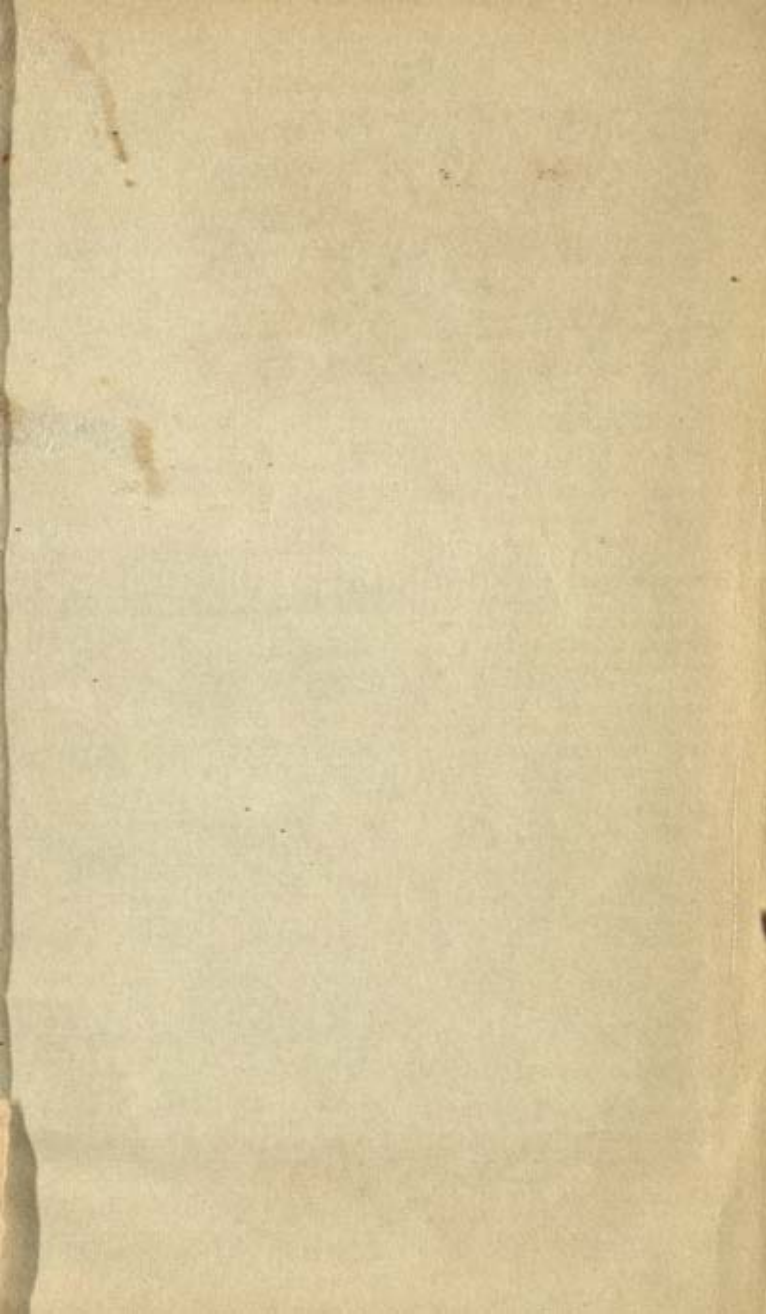
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