CIVIL ENGINEER'S HANDBOOK

ASSOCIATION OF ENGINEERS
I&CAD DEPARTMENT
DR.K.L.RAO BHAVAN, OPP: MEDINOVA,
ERRUM MANZIL, HYDERABAD - 500 082.

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PREFACE

The Association of Engineers has the credibility of publishing number of Technical books. The necessity of a comprehensive Civil Engineer's hand book was felt by the Association and the first edition was published in the year 1987. Later as there is a demand from the members & Department Engineers for a revised and enlarged edition of Civil Engineer's Hand book, the revised edition of the Hand book was published in the year 1997.

The book was devised and planned to be a practical guide and most valuable reference book consisting of different chapters. The information was collected and compiled by a team of Engineers with specialisation in respective fields from various text books, BIS Codes and manuals. The Hand book got very much appreciation from all the Departments and all the books were sold with in no time. There is a demand from the members and particularly from the budding Engineers for the Hand book.

The Association considering the demand for the Hand Book from all the engineering sections, decided to bring a new edition duly incorporating additional information on lift Schemes, list of IS Codes required for the design of irrigation components like spillway, No F dams, earth dams, L.I. Schemes, CM & CD works and mechanical works. A new chapter on Quality control aspects (dos and don'ts) is added for the benefit of Engineers in field.

We thank all the Engineers for their continuous support and encouragement to the Association in bringing out this edition. The Association specially thank Sri I.S.N. Raju, Chief Engineer, Central Designs Organisation, Hyderabad for the suggestions given in publishing the new edition of this book.

We except the same cooperation and encouragement from all the members for publishing many more books in future relevant to the Department.

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Errum Manzil, Hyderabad.

I am very happy to know that the Association of Engineers is bringing out a new edition of the Civil Engineer's Hand Book published in the year 1998 with additions. The data and other relevant information as per IS Codes and manuals will certainly give guidance to our Engineers in their regular work.

I wish all the young Engineers joined in the Department recently to have a copy of this Hand Book and take advantage in performing their duties.

I congratulate the Association of Engineers in bringing out this revised edition

M.K. RAHAMAN

CIVIL ENGINEER'S HAND BOOK
APRIL 2010

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SECTION 1
PLAIN CEMENT CONCRETE

1.1.0.0 Concrete is one of the few building material that is manufactured at the site of construction. It is the Resident Engineer who is the sole manufacturer responsible for turning out the best quality concrete and realise its inherent worth as a field made product.

Types of Cement

1.1.1.0 The cement that is most familiar to builders and civil engineers is ordinary portland cement.Special cements are used for special purposes.The properties of certain types of cement are briefly described below:

Ordinary Portland Cement (33 Grade conforming to IS 269)
(43 Grade conforming to IS 8112)
(53 Grade conforming to IS 12269)

1.1.1.1 This cement is the basic Portland cement and is commonly manufactured in far larger quantities. It is admirably suited for use in general concrete construction when there is no exposure to Sulphates in the soil or in ground water.

Rapid hardening Portland cement (IS 8041/1990)

1.1.1.2 This cement is similar to ordinary portland cement but an adjustment of its chemical composition to give a higher content of tricalcium silicate (C_3S), and a finer grinding, enable it to attain greater strengths at early ages; that is why it is known as high early strength cement in America. The 7 - day strength of portland cement with the same water - cement ratio is attained by 3 days using this cement.

Low heat Portland cement (IS-12600/1989)

1.1.1.3 When a member is being concreted an increase in temperature occurs due to the chemical reaction which takes place while cement is setting and hardening. In ordinary concrete construction this evolution of heat is of little consequence, but in concrete dams and other massive structures such as bridge abutments, piers, and retaining walls, it is of considerable importance.

One of the ways to prevent this build-up of heat in structures and its consequential shrinkage/cracking, is to use low heat portland cement, the chemical composition of which is so designed as to obtain reduced heat evolution. Its rate of development in strength is about one-half that of concrete made of ordinary portland cement at 7-days rather more than two-thirds 28 days and at least equal at an age of 3 months and upwards. It is resistant to chemic deterioration than ordinary portland cement.

Portland - Pozzolana Cement (IS - 1489 / Part 2)

1.1.1.4 This cement is manufactured either by intergrinding portland cement clinker an pozzolana or by intimately and uniformly blending Portland cement and fine pozzolana. IS cod specifies that the latter method should be confined to places where intimate blending can b ensured through mechanical means.

The following table compares the strength at different ages of Portland -pozzolan cement and portland cement manufactured by certain factories.

Age in days	Compressive strength kg/cm2	
	Portland pozzolana cement	Ordinary portland cement
3	200-220	190-230
7	260-330	270-320
28	370-480	360-520

Although IS: 1489/1967 does not specify a minimum, compressive strength at three days, certain factories like ACC ensure that its Portland-pozzolana cement meets the minimum IS requirement for ordinary portland cement (The Indian Concrete Journal - November 1974).

Storing Cement

1.1.2.0 Modern cement is a very finely ground material and as such it is highly hygroscopic, that is to say, it readily absorbs moisture not only in the form of free water but also moisture from the air. It is therefore necessary to protect it from dampness during storage before it is used.

Every year and particularly in the monsoon, large quantities of cement get spoiled due to negligence in keeping the cement bags absolutely dry. The folowing points on the correct storage of Portland cement will therefore merit careful consideration.

The first requirement is that the bags should be stored in a building or a shed which is completely weather proof and dry. The following points are important:

i) The walls should be waterproof masonry construction

ii) The roof should be leakproof - preferably of re inforced concrete construction overlaid with a water proofing course.

iii) The windows should be few and small and kept tightly shut; this is to prevent moisture from the outside atmosphere entering the building.

iv) The best floor is 150 mm thick concrete slab laid on a dry course of soling and is 1.2 m above ground level. The ground should be drained away from the building to prevent accumulation of rain fall in its vicinity; this precution will ensure that the floor will be absolutely dry.

v) The plinth should be fairly high so that a truck can back conveniently to the door and the chassis and building floors are nearly at the same level, thus making loading and unloading of bags very easy.

Having made sure of good weather proof building, certain precautions indicated below must then be observed in storing the bags with a view to prevent them from any possible contact with moisture and to ensure systematic working of the warehouse.

vi) If the warehouse is to be nearly put into service, make sure that sufficient time has elapsed to allow the interior to dry thoroughly.

vii) Do not pile bags against the wall. A space of 300 mm all round should be left between the exterior walls and the piles.

viii) Similarly, bags should be piled off the floor on wooden planks. If however, the floor is a well constructed dry concrete floor, the bags can be placed directly on it.

ix) Place bags close together in the pile to reduce any circulation of damp air as much as possible.

x) Do not pile bags more than about 15' high, otherwise it becomes cumbersome to stack or remove them. The maximum width of pile should also be not more than about 3 m (10').

xi) If the pile is to be more than 7 or 8 bags high, arrange the bags in header and stretcher fashion, that is, alternatively lengthwise and cross wise so as to tie the piles together and lessen the danger of toppling over.

xii) For extra safety during the monsoon, or during long periods of storage, enclose the pile completely in 700 gauge polythene sheet. This can be conveniently done by making a large loose sack of the material and arranging the bags within it. The flaps will close on the top of the pile. Take care to see that the polythene is not damaged anytime during use.

xiii) Owing to the pressure on the lower layer of bags, a phenomenon known as "warehouse pack" sometimes develops in the bags. This can be removed by rolling the bags when the cement is taken out for use. Do not, on account of warehouse pack, move and restack the bags. There is no advantage in doing so.

xiv) When removing bags from storage some bags should be removed from two or three tiers back rather than all from one tier. If the piles are thus stepped back, there is less chance of over - turning them.

xv) When removing bags for use apply the "first in, first out" rule, that is take the oldest cement, out first. For this purpose, each consignment as it comes in should be stacked separately and a placard bearing the date of arrival should be pinned into the pile. The longer it is stored, the greater is its deterioration besides chances of absorption of moisture and cement "set in".

TABLE - 1.1

Reduction in strength of concrete made with stored cement	
Period of storage cement	Minimum expected reductions in strength at 28 days (%)
Fresh	0
3 months	20
6 months	30
1 year	40
2 years	50

TABLE - 1.2

Rate of hardening of concrete made with fresh and stored cement		
Test age	1:5 concrete made with cement stored in bags under normal conditions for 6 months, %	1:5 concrete made with fresh cement, %
7 days	73	100
28 days	75	100
06 months	84	100

Note : The above values are approximate.
If cement is likely to have deteriorated during storage, it should br sent to a laboratory for test.

AGGREGATES

1.1.3.0 "Aggregates" is a general term applied to those inert (that is, chemically in active) materials which, when bonded together -by cement, form concrete.

i) Natural aggregates :-Sand, crushed rock and gravel.

ii) Artificial (processed) aggregates :- Broken brick or crushed air-cooled blast-furnace slag.

iii) Light Weight aggregates :- Pumice, furnace clinker, coke breeze, sawdust, foamed slag, expanded clays and shales, expanded slates, expanded vermiculite etc.

Classification of Aggregates

1.1.3.1 Aggregates are commonly classified into two categories, fine and coarse, the dividing line being the 4.75 mm IS sieve. Material which is mainly retained on this sieve is classed as "Coarse aggregate", whereas material mainly passing it is classed as "fine aggregate". For large and important works it has become usual to separeate the coarse aggregate also into two or more sizes, and these fractions are kept seprete until the proper quantity of each has been weighed out for a batch of concrete.

"All-in-aggregate", that is to say, aggregate as it comes from the pit or river bed, is sometimes used for unimportant works.

Quality of aggregates

Natural aggregate used for concrete construction is required to comply with the norms laid down in IS : 383-1970, "Specification for coarse and the fine aggregate from natural source for concrete". It should be chemically inert, strong, hard, durable, of limited porosity, free from adherent coatings, clay lumps, coal and coal residues, and should contain no organic or other admixture that may cause corrosion of the reinforcement or impair the strength or durability of the concrete. The limits of the content of deleterious materials in aggregate are indicated in Table 1.3.

Particle shape

1.1.3.2 The particle shape of an aggregate is generally classified as rounded, irregular, angular, or flaky. Experience indicates that angular aggregates do not produce as smooth and workable a mixture as materials having more rounded and smoother particles, the grading and other conditions being similar. The sharp angular fragments of crushed rock require more sand and more cement to produce workable concrete, whereas coarser gradings can be used with gravels, with consequent decrease in mortar requirement.

Any excess of thin, flat, elongated, flaky, or splintering particles in aggregate is objectionable as it materially decreases the workability for a given water-cement ratio, and necessitates more highly sanded mixes and consequent use of more cement and water.

Grading

1.1.3.3 Almost as important as the quality of the aggregate is its gradation that is to say, its particle size distribution as determined by sieve analysis. The following Indian Standard test sieves are generally required for grading aggregates:

Fine mesh wire cloth
3-35mm, 2.36mm, 1.18mm, 600 micron, 300 micron, 75 micron.

Square hole perforated plate
80 mm, 63 mm, 50 mm, 40 mm, 31.5 mm,25 mm, 20 mm, 16 mm, 12.5 mm, 10 mm, 6.3 mm, 4.75 mm.

FIELD TEST FOR CONTAMINATION OF CEMENT (Based on Hand Book)

1.1.3.4 Out of the two simple field tests given below, any one can be performed to determine, if the cement has been contaminated.

In the first test a sample of cement is burnt for about 20 minutes on a steel plate. An adulterated sample will change in colour. The colour of an unadulterated sample will, on the other hand, remain unchanged.

In the second test : small pats of 5 x 5 x 2 cm size should be made. If the cement is unadulterated, the pat will have hardened sufficiently in 24 hours to resist any impression of the thumb nail. In 48 hours it should not be easy to break it with the pressure of the fingers.

I f the cement fails to pass these tests, it should be sent to a laboratory to determine the extent to which its properties have been affected.

FIELD TEST FOR SAND (IS 1542-1960)

1.1.3.5 To check the suitability of sand for being used in mortar or concerete it may be put to the following tests in the feild:

i) Taste of sand shall provided check for the presence of salts.

ii) Rub a little sand between the fingers, stains left on fingers will indicate the presence of clay impurities.

iii) Vigorously stir a sample of sand in a glass of warter and allow it to rest. Amount of clay or silt present in it would settle on the sand.

iv) Stir a sample of sand in a 3% solution of caustic soda and keep the bottle corked for 24 hour. If the colour of the liquid turns brown, then the presence of organic matter is indicated.

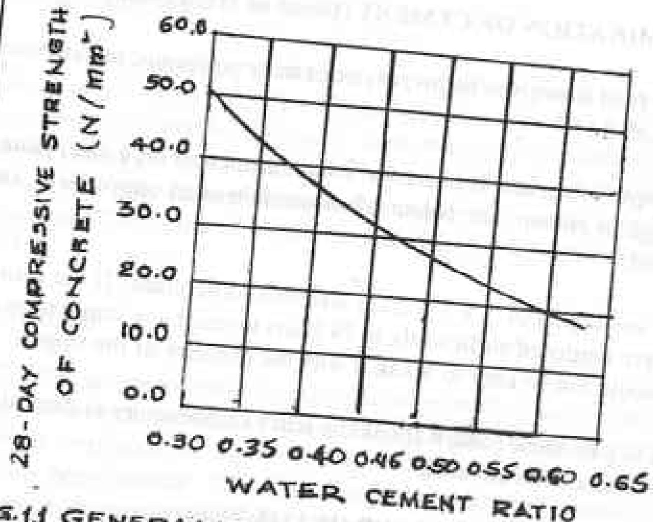

FIG.1.1 GENERALISED RELATION BETWEEN FREE WATER CEMENT RATIO COMPRESSIVE STRENGTH OF CONCRETE

TABLE - 1.3

Limits of the content of deleterious materials in aggregate (IS : 383-1970)

Deleterious substance	Fine Aggregate		Coarse Aggregate	
	Uncrushed	Crushed	Uncrushed	Crushed
Coal and ignite	1.00	1.00	1.00	1.00
Clay lumps	1.00	1.00	1.00	1.00
Soft fragments	-	-	3.00	-
Deleterious substance	**Fine Aggregate**		**Coarse Aggregate**	
	Uncrushed	Crushed	Uncrushed	Crushed
Material finer than 75 μ -IS Sieve	3.00	15.00	3.00	3.00
Shale	1.00	-	-	-
Total of percentages of all deleterious materials	+ 5.00	2.00	5.00	5.00

Percentage by weight of aggregate + Mica is excluded

1.1.3.6 <u>Maximum permissible limits for solids in water used for concreting</u> :-

Organic	200 mg/litre
Inorganic	3000 mg/litre
Sulphates (as SO_4)	500 mg/litre

Chlorides (as Cl) 2000 mg/litre for plain concrete
Chlorides 1000 mg/litre for RCC work
Suspended matter 2000 mg/litre

1.1.3.7 <u>Quality of water used in concreting</u> :-

1. Under unavoidable circumstances sea water may be used for mixing in plain or R.C concrete which are premanently under sea water.
2. Potable water with pH. value not less than 6 & at other situations 6-.9.

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3. Presence of tannic acid or iron compounds is objectionable. (Initial setting time 30 minutes for a test block.)

ADMIXTURES

1.1.3.8 Classification of Admixtures :-

Admixtures are :-

1. Air-entraining agents 2. Retarders 3. Accelerators
4. Waterproofers 5. Pozzolanas 6. Pigments and
7. Workability agents

TABLE - 1.4

CONCRETE ADMIXTURES AND THEIR PROPERTIES, CLASSIFICATION OF ADMIXTURES

Classification of concrete admixtures based on their purpose and chemical constituents is tabulated below;

Type of admixture	Materials	Desired effect
Air Entrainer	Salts of wood resins, some synthetic detergents, salts of sulphonated lignins, salts of petroleum acids, salts of protein -aceous materials, Fatty and resinous acids and their salts, Organic salts of sulphonated hydrocarbons.	Increase durability if the water cement ratio is reduced.
Shrinkage compensator	Calcium sulphate plus calcium sulpho aluminate, finely divided iron plus oxidizing agent.	Cause expansion of setting.
Gas former	Aluminium powder	
Water reducer	Lignosulphonates and their derivatives salts of hydroxylated carboxylic acids and their derivatives.	Increase strength and workability.

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Super Plasticiser	Sulphonated melamine formaldehyde condesate, Naphthalene sulphonated formal-dehyde condenstate, Modified lignosulphonate.	Increase strenthg and Work ability.
Accelerator	Calcium chloride, other solube chlorides Carbonates, Silicates, Alkali hydroxides, Triethanolamine.	Increase early strenght and accelerate set.
Retarder	Lignin, Borax sugars, Tartaric acid and Salts.	Retardation of set.
Air detrainer	Tributyl phosphate	To dissipate excess air other than gas.
Flocculating agent	Synthetic polyelectrolytes Pastes and mortars	to reduce the flow of mortar and to increase cohesion
Pumping aids	Polythene oxide, Cellulose derivatives, Alginates	Improves the handling of concrete
Pozzolanas	Flyash, Volcanic alass, Diatomite, some clays and shales	To reduce heat of liberation on hydration of economy.
Foaming agents	Sodium laury sulphate, Alkyl-aryl sulphonate.	For making light weight concrete
Water proofing petroleum products.	Some soaps Butyl stearate, Some water through concrete.	To prevent passage of agents
Bonding agents	Organic polymer emulsions.	To increase bond strenght Between old and new concrete.

Corrosion inhibitors	Sodium benzoate Calcium Ligno -sulphate, Sodium nitrate.	Corrosion protection of reinforcing steel.
Fungicide and germici- germicides properties	Copper compounds, Dieldrin emulsion, polyhal ogonated Phenels.	To impart fungicidal, dal or insecticidal to cement concrete.
pigments	colour pigment	NOTE: It is understood that the effectiveness is of temporary natrue.
	Grey - Black: Carbon black, mineral black, Balck iron oxide.	1. Colour fastness when exposed to light. 2. Chemical stability in presence of alkalinity produced in set cement.
	Blue : Ultramarine blue phthalocyanine blue	
	Red : Red iron oxide	
	Brown : Brown iron oxide, Raw or burnt Umber.	
	Cream : Yellow iron oxide.	
	Buff :	
	Green : Chromium oxide phthalocyanine green	
	White : Titanium oxide.	

TABLE - 1.5

PROPORTIONS FOR NORMAL MIX CONCRETE(By weights)

Grade of concrete	Dry aggregate by weights/ 50 kg (Bag) of cement.(Fine + coarse aggregate in Kgs.)	Maximum water content in litres 50 kg.(Bag) of cement	Remarks
M5 (1:5:10)	800 Proportions of fine	60	When machine mixing is
M7.5(1:4:8)	625 and coarse aggregates	45	employed, mixing time
M10(1:3:6)	480 (1:2) for 20 mm(By mass)	34	may be 1 1/2 to 2 to get
M15(1:2:4)	350 (1:1 1/2) for 10 mm	32	uniform consistency and
M20(1:1 1/2:3)	250 (1:2 1/2) for 40 mm	30	colour.
M25(1:1:2)) }	and coarse aggregates		
M30 }	To be worked out		
M35 }			

SIZE OF AGGREGATE : For heavily reinforced concrete Members, the aggregate should usually be restricted to 5 mm less than the minimum clear distance between the main bars OR 5 mm less than the minimum cover to the reinforcement whicever is smaller.

TABLE 1.6

Minimum Cement Content, Maximum Water-Cement Ratio and Minimum Grade of Concrete for Different Exposures with Normal Weight Aggregates of 20 mm Nominal Maximum Size

(Clauses 6.1.2, 8.2.4.1 and 9.1.2)

Sl. No.	Exposure	Plain Concrete			Reinforced Concrete		
		Minimum Cement Content Kg/m³	Maximum Free Water-Cement Ratio	Minimum Grade of Concrete	Minimum Cement Content Kg/m³	Maximum Free Water-Cement Ratio	Minimum Grade of Concrete
1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
i)	Mild	220	0.60	–	300	0.55	M 20
ii)	Moderate	240	0.60	M 15	300	0.50	M 25
iii)	Severe	250	0.50	M 20	320	0.45	M 30
iv)	Very severe	260	0.45	M 20	340	0.45	M 35
v)	Extreme	280	0.40	M 25	360	0.40	M 40

NOTES

1. Cement content prescribed in this table is irrespective of the grades of cement and it is inclusive of additions mentioned in 5.2. The additions such as fly ash or ground granulatee blast furnace slag may be taken into account in the concrete composition with respect to the cement content and water-cement ratio if the suitability is established and as long as the maximum amounts taken into account do not exceed the limit of pozzolona and slag specified in IS 1489 (Part 1) and IS 455 respectively.
2. Minimum grade for plain concrete under mild exposure condition is not specified.

Exposure	Plain Concrete		RCC	
	Minimum cement content	Maximum water cement Ratio	Minimum cement content	Maximum water cement Ratio
1	2	3	4	5
Severe - For example, to sea water, alternative wetting and drying and to freezing whilst wet subject to heavy condensation or corrosive fumes. (Marine structures)	310	0.5	360	0.45

Note : 1. When the maximum water-cement ratio can be strictly controlled, the cement content in the above table may be reduced by 10%.

Note : 2. The minimum cement content id based on 20mm aggregate. For 40mm aggregate, it should be reduced by 10%; For 12.5 mm aggregate, it should be increased by about 10%.

Note : 3. Concrete of grades lower than those given in the table may be used for plain concrete constructions, lean concrete, simple foundations for masonry walls.

Where the concrete is exposed to sulphate attack (as SO_3 with 0.2 to 1% in soil or as 30 to 250 ppm in ground water). The cement content may vary between 180 to 330 kg/m³ with water cement ratio being about 0.5 with a pH value 6 to 9 i.e., near neutral conditions. Super sulphated cement with w/c ratio of 0.4 is for mineral acids down the pH 3.5 or use portland slag cement (slag above 50%). Ordinary portland cement shall not be used in acid conditions with pH value 6 or less.

TABLE 1-7

MIX PROPORTIONS AND AGGREGATES FOR "ORDINARY CONCRETE" (by volume)

Grade of Concrete	Corresponding Nominal Mix	Sand + Coarse aggregate per bag of cement (50 Kg by volume)	*water per bag of cement	Approximate cement consumption per M3 of concrete
M-7.5	1:4:8	500 L	38 L	3 1/4 BAGS/M3
M-10	1:3:6	300 L	34 L	4 1/4 BAGS/M3
M-15	1:2:4	220 L	32 L	6 1/2 BAGS/M3
M-20	1:1 1/2:3	160 L	30 L	8 1/2 BAGS/M3
M-25	1:1:2	100 L	27 L	11 BAGS/M3

L = LITRES (1000 CC)

* The quantity of water indicated above is for dry condition of sand and coarse aggregates.

TABLE 1 - 8

PROPORTIONS OF SAND & AGGREGATES WITH FINNESS MODULES "ORDINARY CONCRETE"

Max. size of Aggregate	Proportion of Sand : total aggregates		
	Fine Sand (F M 2.2 - 2.6)	Medium Sand (F M 2.6 - 2.9)	Coarse Sand (F M 2.9 - 3.6)
15 mm	30 - 35 %	35 - 40 %	40 - 45 %
20 mm	25 - 30 %	30 - 35 %	35 - 40 %
30 mm	25 - 30 %	30 - 35 %	34 - 40 %
40 mm	25 - 30 %	25 - 30 %	30 - 35 %
50 mm	20 - 25 %	25 - 30 %	25 - 30 %

CONCRETE MIX DESIGN

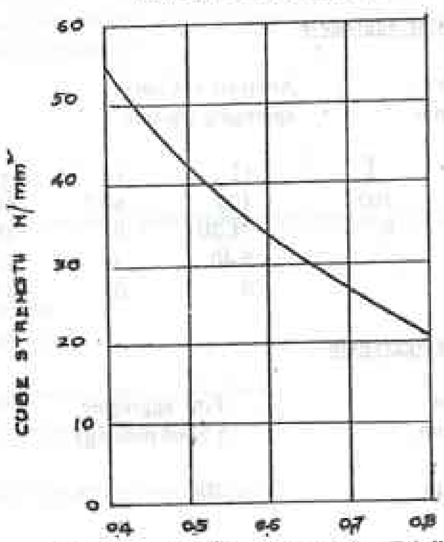

FIG.1.2 WATER/CEMENT RATIO BY WF
TYPICAL RELATION BETWEEN CUBE STRENGTH AND WATER/CEMENT RATIO

1.1.4.0. Examples of concrete MIX DESIGN

1.1.4.1. Illustrative Example on concrete MIX DESIGN (ISI Method)

An example illustrating the mix design for a concrete of M20 grade is given below

Design Stipulations

a) Characteristic compressive strength required in the field at 28 days.	20 N /mm² (200 kg/cm²)
b) Maximum size of aggregates	20 mm (angular)
c) Degree of workability	0.90 compacting factor
d) Degree of equal control	Good
e) Type of exposure	Mild

Test data for materials

a) Cement used-ordinary portland cement conforming to IS - 269/1976

b) Specific gravity of cement 3.15

c) Specific gravity

) Corase aggregate 2.60

2) Fine aggregate 2.60

d) Water absorption

1)Coarse aggregate 0.50 %

2)Fine aggregate 1.00 %

e) Free (surface) moisture

1) Coarse aggregate Nil (absorbed moisture also NIL)

2) Fine aggregate 2.00 %

f) Sieve Analysis

1) Coarse aggregate

IS sieve sizes mm	Analysis of Coarse aggregate fraction I	II	% of different fractions (% passing) I	II	COMBINED	Remarks
20	100	100	60	40	100	
10	0	71.20	0	28.5	28.5	Conforming to
4.75		9.40	0	3.70	3.7	IS 383 - 1970
2.3		0	0	0	0	(See table 1.15)

2) Fine aggregate

IS sieve sizes mm	Fine aggregate (% of passing)	Remarks
4.75 mm	100	Conforming to grading zone III of IS : 383 - 1970 (See table 1.15)
2.36 mm	100	
1.18 mm	93	
600 microns	60	
300 microns	12	
150 microns	2	

Target Strength For Mix Design :

In order that not more then the specified portion of test results are likely to fall below the characterstic strength , the concrete mix has to be designed for a some what higher target average compressive strength (f_{ck}). The margin over the characteristic strength depends upon the quality of concrete (expressed by standard deviation) and the accepted proportion of results of strength tests below the characteristic strength ((f_{ck}).given by the relations.

$$f_{ck} = f_{ck} + t \cdot s \qquad (1)$$

where f_{ck} = target average compressive strength at 28days in N/mm² (kg/cm²)

f_{ck} = Characterisitic compressive strength at 28 days

S = Standard deviation

t = a statistical constant given in table 1.9

Calculation of Aggregate Content :- With the quantities of aggregates and cement/ unit volume of concrete and the ratio of fine to total aggregate already determined , the total aggregate content/unit volume of concrete may be calculated from the following equations :

$$V = [\, W + C/S_c + (1/P.)\,(fa/S_{fa})\, \times 1/1000, \text{ and} \quad .....2$$

$$V = [\, W + C/S_c + (1/1\text{-}P.)\,(Ca/S_{ca})\, \times 1/1000 \quad ......3$$

Where V = Absolute volume of fresh concrete, whch is equal to gross volume (m³) minus the volume of entrapped air,

W = mass of water in Kg / m³ of concrete,

C = mass of cement in Kg / m³ of concrete

S_c = Sp.gr of cement

P = ratio of fine aggregate ot total aggregate by absolute volume

fa, ca = Total masses of fine aggregate and coarse aggregate in Kg/m³ of concrete.

S_{fa},S_{ca} = sp.gr. of saturated surface dry fine and coarse aggregates, respectively.

No 1 **Target mean strength of concrete :-**

For a tolerance factor of 1.65 and using table 1-9 and 1-10, the target near strength for the specified characteristic strength is

$= 20 + 4.6 \times 1.65$ from equation (1)

$= 27.6$ N/mm² (276 kg/cm²)

No 2 **Selection of water-cement ratio :-**

From Fig 1.1, the free water cement ratio required for the target mean strength of 27.6 N/m is 0.50. This is lower than the maximum value of 0.65 prescribed for "Mild" exposure in table 1-6, correction in water content and sand as per Table 1-13.

No 3 **Selection of water - and Sand content:**

From Table 1-16, for 20 mm nominal maximum size aggregate and sand conforming to gr zone ii, and sand conforming to grading zoneII, water content/m3 of concrete - 186 Kg and content 89% of total aggregate by absolute volume 35%.
For change in values in W/C ratio, compacting factor and sand belonging to zone III, the follo adjustment is required.

Change in condition (From table 1.1.3)	Adjustment required in water content and sand in total aggregate %	
For decrease in W/C ratio by (0.65-0.50)that is 0.15	0	- 3.0
For increase in compacting factor (0.9-0.8)that is 0.1	+3	0
For sand conforming to zone III of Table 1.1.2	0	- 1.5
Total	+3%	- 4.5%

Therefore, required sand content as % of total agregate by absolute volume = 35-4.5 = 30.5%

Required water content = $186 + \frac{186 \times 3}{100}$ = 186 + 5.58186

= = 191.6 l/m³

No.4 **Determination of Cement Content**

Water cement ratio	=	0.50
Water	=	191.6 Litre
Cement	=	$\frac{191.6}{0.5}$ = 383 Kg/m³

This cement content is adequate for mild exposure condition according to Table 1.6 (i.e. it is more than 200 Kg/m³) and Table (1.1.1) 310 Kg/m³).

Step No.5 **Determination of coarse and fine aggregate content**

From Table 1-13, for the specified maximum size of aggregate of 20 mm, the amount of entrapped air in the wet concrete is 2%. Taking this into account and applying equations (2) and (3)

(1-0.02)	=	(191.6+383/3.15+1/0.305x Fa/2.60)x1/1000
f_a	=	529 kg/m³
and (1-0.02)	=	(191.6 + 383/3.15+1/0.695xCa/2.60)x1/1000
ca	=	1205 Kg/m³

The MIX proportion then becomes

Water	Cement	Fa	Ca
191.6 Ltrs	383 kgs	529 kg	1205 kg
0.50	1	1.381	3.146

Step No 6 Actual quantities required for the max. per bag of Cement:
The MIX is 0.50 : 1 : 1.381 : 3.146 (by mass). For 50 kg. of cement, the quantity of materials are worked out as below:

a) Cement = 50kg
b) Sand = 69 kg
c) Coarse aggregate = 157 Kg (fraction I (60 %) = 94.2 kgs
(fraction II (40 %) = 62.8 kgs

d) Water
1. For W/C ratio of 0.50 quantity of water = 25 litres.
2. Extra quantity of water to be added for absorption incase of coarse aggregate at 0.5% by mass = (+) 0.77
3. Quantity of water to be deducted for free moisture present in sand at 2% by mass = (-1.42)

4. Actual quantity of water to be added = 25+0.77- 1.42 = 24.35 litres.
e) Actual quantity of sand required after allowing for mass of free moisture = 69+ 1.42 = 70.42 kgs.
f) Actual quantity of coarse aggrigate required
1. Fraction I = 94.2-0.5/100 x 94.2 = 94 - 0.47=93.73 kgs
2. Fraction II = 62.8-0.5/100 x 62.8 = 62.8- 0.31 = 62.49 kgs .
Therefore, the actual quantities of different constitutants required for the Mix are :-
1. Water = 24.35 kgs litres. 2. Cement = 50 kgs 3. Sand = 70.42 kgs.
Coarse aggregate = Fraction (1) = 93.73 kgs = Fraction (2) = 62.49 kgs
This completes the design of the concrete mix M 20

TABLE 1.9

VALUES OF t (STATISTICAL CONSTANTS)

Accepted proportion of test results	t
1 in 5	0.84
1 in 10	1.28
1 in 15	1.50
1 in 20	1.65
1 in 40	1.86
1 in 100	2.33

(IS : 10262 - 1982)

TABLE 1.10

Assumed Standard Deviation

(Clause 9.2.4.2 and Table 11)

Grade of	Assumed Standard Deviation N/mm²
M 10, M 15	3.5
M 20, M 25	4.0
M 30, M 35, M 40, M 45, M 50	5.0

NOTE :- The above values correspond to the site control having proper storage of cement; weigh batching of all materials; controlled addition of water, regular checking of all materials, aggregate gradings and moisture content; and periodical checking of workability and strength. Where there is deviation from the above the values given in the above table shall be increased by 1N/mm².

TABLE 1.11

APPROXIMATE CEMENT AND SAND CONTENTS PER M³ OF CONCRETE UPTO MASS MIX. M35

Nominal max. size of aggregate (mm)	Cement per M³ concrete in kgs.	Water content per M³ of concrete in kgs.	Sand as percent of total aggregate by absolute volume
10	347	208	40
20	310	186	35
40	245	165	30

Table 1.11 is to be used for concretes grade upto M_{35} and is based on the following conditions:-

a) Crushed (Angular) coarse, conforming to IS 383 - 1970.

b) Fine aggregate, consisting of natural sand conforming to grading zone II of Table 1.15

c) W/C ratio of 0.6 (by mass) and

d) Workability corresponding to compacting factor of 0.80

TABLE 1.12

A PPROXIMATE CEMENT AND SAND CONTENTS/M³ OF CONCRETE ABOVE M35 (IS: 10262 - 1982)

Nominal max. size of aggregates mm	Cement/m³ of concrete kg	water content/m³ of concrete kg	Sand as % 0f total aggregate by absolute volume
10	571	200	28
20	514	180	25

Table 1-12 is to be used for concretes of grade above M_{35} and is based on the following conditions

a) crushed (angular) coarse aggregate , conforming to IS 383 - 1970.

b) Fine aggregate consisting of natural sand confoming to grading Z one 11 of table 1.15 .

c) W/C ratio of 0.35 (By mass), and

d) workability corresponding to compacting factor of 0.80

For other conditions of workability, W/C ratio, grading of fine aggregate , and for rounded aggregates, certain adjustments in the quantity of mixing water and fine to total aggregates , certain adjustments in the quantity of mixing water and fine to total aggregates ratrio is given in Table 1.10 and 1.11 and are to be made, according to Table 1 .12

TABLE 1-13

Adjustment of values in water content and sand % for other condtions.

Change in conditions stipulated for table 1- 11 and 1 - 12	Water content	Adjustment required in % sand in total aggregate
1. For sand conforming to grading zone 1, zone 111 or zone 1V, of table 1 - 11	0	+ 1.5 % for zone 1 - 1.5 % for zone 111 - 3.0 % for zone 1V
2. Increase or decrease in the value of compacting factor by 0.1	3 %	0
3. Each 0.05 increase or decrease in free W/C ratio	0	1 %
4 For rounded aggregate	-15kg/m³	-7 %

TABLE 1.14

Approximate entrapped air content (depends on max. size of aggregate)

Nominal Max. size of aggregate mm	Approximate amount of entrapped air, as % of volume of wet concrete in non - air entrapped concrete.
10	3.00
20	2.00
40	1.00

The specific gravity bulk densuty and voids of different types of aggregates are given in Table 1.15.

TABLE 1.15

Specific gravity, bulk density and voids of different types of aggregate

Specific gravity		Bulk density (kg/ltr)		Voids (%)	
		River sand :		River sand :	
Trap	2.90	Fine	1.44	Fine	43
Granite	2.80	Medium	1.52	Coarse	35
Gravel	2.66	Coarse	1.60	Mixed moist	38
				Mixed and dry	30
Sand	2.65 to 2.60	Beach or river shingle	1.60	Broken stone graded :	
		Brokers Stone	1.60	25mm max.size	46
		Stone screenings	1.44	50mm max. size	45
		Broken Granite	1.68	63mm max. size	41
				Stone screenings	48

TABLE 1.16

Fine Aggregates

IS sieve Designation	Percentage passing for			
	Grading zone 1	Grading zone 11	Grading zone111	Grading zone 1V
10mm	100	100	100	100
4.75mm	90 -100	90 -100	90 -100	95 -100
2.36mm	60 - 95	75 -100	85 -100	95 -100
1.18mm	30 -70	55 -90	70 -100	90 -100
600 micron	15 -34	35 -59	60 -79	80 -100
300 micron	5 - 20	8 -30	12 -40	15 - 50
150 micron	0 - 10	0 - 10	0 - 10	0 - 15

NOTE :- 1. For crushed stone sands, the permissible limit on 150 micron IS Sieve is increased to 20 percent . This does not affect the 5 percent allownace permitted in 5.3 applying to other sieve sizes

NOTE :-2. Fine aggregate complying with the requirments of any grading zone in this table is suitable for concrete but the quality of concrete produced will depend upon a number of factors including

NOTE :- 3. Where concrete of high strength and good durability is required, fine aggregate conforming to any one of the four grading zones may be used, but the concrete mix should be properly designed. As the fine aggregate grading becomes progressively finer, that is, from Grading zones I to IV, the ratio of fine aggregate to coarse aggregate should be progressively reduced. The most suitable fine to coarse ratio to be used for any particular mix will, however, depend upon the actual grading practicle shape and surface texture of both fine and coarse aggregates.

NOTE :-4 It is recommended that fine aggregate conforming to grading zone IV should not be used in reinforced concrete unless tests have been made to ascertain the suitability of proportions.

PERMEABILITY AND DURABILITY OF CONCRETE

1.1.5.1. Factors Affecting Permeability

The factors which affect the permeability of concrete are

I. Cement and water,
II. Aggregate,
III. Curing,
IV Admixtures, and
V. Absorptions and uniformity of concrete.

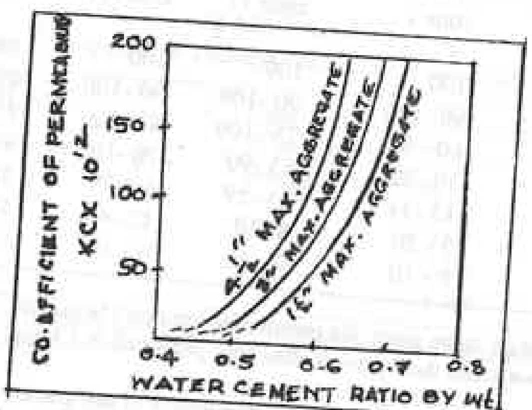

FIG. 1.3 EFFECT OF WATER/CEMENT RATIO UPON PERMEABILITY

1.1.6.0 **CURING CONCRETE** : The object of curing is to prevent or replenish the loss of necessary moisture during the (early) rapid stage of hydration .

a) Moist curing : The exposed surfaces are kept moist by sparying ponding, covering with earth,sand etc.., precast works or works in cold weather are steam cured .Early drying must be prevented especially the exposed edges/corners by suitable coverings. Pozzelane cements require 21 days curingperiod. The optimum temparature dring curing period is 15° to 38°C.

b) Membrane curing : An approved curing compound may be applied as soon as the concrete has initially set . This restricts the evaporation of thewater used in mixing. White prigmented sealing compound for curing in slopes of canal linings and related structures is being practised. It can be applied at the rate of (1 gallon/ 150 sqft) 1 litre/sq. Metre of a rea s onably smooth surface. The coating shall be protected for atleast 28 days by a 25mm thick cover of sand or earth.

c) Steam curing : By steam curing at temperatures between (130° - 160°F) 54° - 74°C,maximum strengths in precast members may be attained by 72 hours, which helps removal of forms much earlier in low pressure (atmospheric) curing. In high pressure curing at 175°c saturated steam at pressures of 8 atmospheres is employed. Certainchemical reactions take place and the strength of concrete is improved by 15% than the normally cured concrete. This precess with the chemical reactions taking place therein have the following advantages.

a) About 50% of cement is saved .
b) Leaching and efflorescene is minimised .
c) Sulphate resistance is built up
d) Initial drying shrinkage is minimised.
e) No harmful side effects on strength in this rapid build up of temperature.

Accelerarted curing methods such as infra-red radiation (USSR) and electrical curing for 3 days are also rarely employed in precast products.

1.1.7.0. **Forms - Stripping time of forms-**

Where ordinary portland cement is used (and the temperature is above 20°C) forms may be struck after the expiry of the following periods.

a)Walls, columns and vertical sides of beams	24 to 28 hours
b) Slabs (props. left under)	3 days

c) Beam soffits (props.left under)	7days
d) Removal of props. under slabs	
i) Spanning up to 4.5 m	7days
ii) Spanning over 4.5 m	14 days
e) Removal of props under beams and arches	
i) Spanning upto 6m	14 days
ii) Spanning over 6m	21 days

Note : The props left under may be sufficient to carry the full dead loads with any live load likely to occur during curing or construction .

TABLE 1-17

RECOMMENDED MIXES FOR VARIOUS TYPES OF CONSTRUCTION

Type of work	Recomm mix (by vol)	Max size of aggr	Water in its per bag of cement *	Best consis tency	Slump in cms
Long span arches heavily loaded column	1:1: 2	12 to 20	16 to 18	medium	
Heavily stressed members;small precast works,such as posts and poles for fencing, telegraphs, signals garden furniture , and decorative and other items of work of very thin sections;watertight construction for high heads; long piles.	1: 2:2	2 to 20	20 to 23	medium	
Columns and members subjected to medium, loads ;walls and floors of reservoirs and tanks; cisterns; sewers;well kerbs, platforms and other watertight constructions for moderate heads; non surfaced roof slabs;concrete deposited under water.	1:2:3 or 1:1 2/3:3 1/3	20	25	medium	2.5 to 10 mm
General building work subjected to ordinary stresses such as beams, slabs, columns,panel walls, basement and retaining walls, stairs,lintels and sills; , roads,pavements,driveways and sidewalks;floors;steps; bunkers and silos;bridges, dams, piers etc. exposed to action of water and frost; machine foundations subjected to vibrations;footings;piles	1:2:4	12 to 40 as required	27 to 30	Stiff to medium	2.5 to 5 mm
Mass concrete work in culverts,retaining walls compound walls and ordinary machine bases; foundation walls which need not be watertight	1:3:6	25 to 50	34	stiff to medium	
Mass concrete for heavy walls;foundationsunder column footings and under heavy duty floors.	1:4:8	40 to 75	45 to 48	medium	2.5 to 10 mm

* The exact quantity will depend on the method of compaction, i.e., whether the concrete is to be compected by hand or vibrated, and also on the absorption of water by aggregate.

SECTION – II

REINFORCED CEMENT CONCRETE

1.2.0.0 SYMBOLS

For the purpose of this standard, the following letter symbols shall have the meaning indicated against each; where other symbols are used, they are explained at the appropriate place:

A	Area
b	Breadth of beam, or shorter dimension of a rectangular column
bef	Effective width of slab
b_f	Effective width of flange
b_w	Breadth of web or rib
D	overall depth of beam or slab or diameter of column; dimension of a rectangular column in the direction under consideration.
D_f	Thickness of flange
DL	Dead load
d	Effective depth of beam or slab
d'	Depth of compression reinforcement from the highly compressed face
E_c	Modulus of elasticity of concrete
E_s	Modulus of elasticity of steel
e	Eccentricity
fck	Characteristic compressive strength of concrete
fcr	Modulus of rupture of concrete (flexural tensile strength)
fd	Design strength
fy	Characteristic strength of steel
K	Constant OR coefficient OR factor
Ld	Development length
LL	Live load or imposed load
l	Length of a column or beam between adequate lateral restraints or the unsupported length of a column
lef	Effective span of beam or slab or effective length of column
lex	Effective length about x-x axis
ley	Effective length about y-y axis
ln	Clear span , face to face of supports
ln'	ln for shorter of the two spans at right angles

lx	Length of shorter side of slab
ly	Length of longer side of slab
l_o	Distance between points of zero moments in a beam
l_1	Span in the direction in which moments are determined , centre to centre of supports
l_2	Span transverse to l_1, centre to centre of supports
l_2'	l_2 for the shorter of the continuous spans
M	Bending moment
m	Modular ratio
P	Axial load on a compression member
qo	Calculated maximum bearing pressure of soil
r	Radius
s	Spacing of stirrups or standard deviation
T	Torsional moment
V	Shear force
W	Total load
WL	Wind load
w	Distributed load per unit area
wd	Distributed dead load per unit area
w_i	Distributed imposed load per unit area
x	Depth of neutral axis
Z	Modulus of section
z	Lever arm
α,β	Angle or ratio
σcbc	Permissible stress in concrete in bending compression
σcc	Permissible stress in concrete in direct compression
σsc	Permissible stress in steel in compression
σst	Permissible stress in steel in tension
σsv	Permissible tensile stress in shear reinforcement
τbd	Design bond stress
τc	Shear stress in concrete
τc_{max}	Maximum shear stress in concrete with shear reinforcement
τv	Nominal shear stress
ϕ	Diameter of bar

1.2.0.1 General Design Requirements

Assumptions made :

i) Tensile stresses are taken up by reinforcement and none by concrete

ii) Stress-strain relationship of steel and concrete is a straight line

iii) Modular ratio m= 280/ 3σ cbc where

σabce = bending compressive stress in concrete in N/mm²

Ec = $5700\sqrt{fck}$ N/mm²

Tensile strength of concrete in flexure Fcr = $0.7\sqrt{fck}$ N/mm²

Loads and Forces : R.C. Concrete weight 2500 kg/m³(IRC specifies 2400 kg/m³)

E steel = 200 KN/mm²
= 2×10^6 kg/cm²

Total shrinkage strain = 0.0003

Design load :

a) It is the characteristic load in the case of working stress method.

b) It is the charateristic load with appropriate partial safety factors for limit state design.

1.2.0.2 Stability of the structure

a) Overturning : Restoring moment not less than 1.2 times over turning moment due to D.L. and 1.4 times the Over Turning Moment due to imposed loads.

b) Sliding : Minimum factor of safety :- 1.4. under most adverse combination of forces with 0.9 DL.

c) Durability : Main influencing factore is low permeability which is achieved by strong dense concrete of low water cement ratio; aggregates through compaction, proper curing (for sufficient hydration of the cement).

d) Analysis : By linear & Elastic theory

1.2.0.3. Effective Span

a) Simply supported beams or slabs : The member is not built integrally with its supports.

lef = Clear span (ln) + Effective depth of beam/ slab or c/c of supports whichever is less.

b) Continuous Beam or Slab :-

1. If width of support is less than 1/12 of clear span, then lef = same as (a)

2. If supports are wider than 1/12 of clear span or 600mm whichever is less,

i) For end span, if one end is fixed and the other continuous; OR for intermediate spans, lef= clear span between supports.

ii) For end span with one end free and other continuous : lef = ln + d/2 or half the width of discontinuous support, whichever is less.

For spans with rocker or roller bearing, lef = c/c of bearings, only.

Cantilever : The effective length of cantilever is length to its face + half to effective depth except where it forms the end of a continuous beam where the length to the centre of support shall be taken.

Frames : c/c of supports (for analysis of continuous frames).

Stiffness : Relative stiffness = Moment of Ineratia of the transformed section, taking into consideration the concrete section + area of reinforcement transformed on the basis of modular ratio, m.

Structural Frames : When the design live load does not exceed three fourths of the design dead load, the load arrangement may be for design dead load and live load on all the spans simultaneously.

1.2.0.4. BM and SF Coefficients for " Continuous Beams or Slabs "

Uniform cross sections are assumed for all members and U.D.L. is almost uniform for all spans

Individual span lengths do not differ by more than 15 % of the longest span .

NOTE: 1. For moments at supports where the unequal spans meet or in case where the spans are not equally loaded, the average of the two values for the negative moment at the support may be taken for design.

2) Beam and slabs over free end supports :-

When a member is built into a masonry wall which develops only partial restraint, the member shall be designed for a negative moment at the face of support = Wl/24 where W = the total design load and l = effective span (eg. Slab under parapet walls of roofs, etc.,).

a) B.M. Coefficients for continuous Beams (or Slabs) :-

(Multiply by WL) Total design load x Effective span to get M

Type of load	Span Moments (+) Middle of End span	Span Moments (+) Middle of Interior spa	End span support (-) (*)	Support Moments (-) At support next to end support	Support Moments (-) At other interior supports
Dead load and Imported load fixed	1/12	1/16	-1/16 or 1/10	-1/10	-1/12
Imported load fixed	1/10	1/12	-1/10 or 1/16	-1/9	-1/9

b) SF Coefficients :-

	At End support	At support Next to end support Outer side	Inner side	At all other interior supports
Dead load	0.40	0.60	0.55	0.50
Live load	0.45	0.60	0.60	0.60

Shear Force = Coefficient x Total Design Load

Note : For framed sections, T Beam or L Beam will be designed as doubly reinforced rectangular beam at the supports to resist the designed support moments.

c) Slabs spaning in two directions at right angles : BM Coefficients :-

i) Restrained slabs :- When the corners of the slabs are prevented from lifting, the slab may be designed for the Moments of :-

$$Mx = \alpha x \times WL^2x$$
$$My = \alpha y \times WL^2x$$

ii) Simply supported slabs :- When simply supported slabs do not have adequate provision to resist torsion at corners and to prevent the corners from lifting, the maximum moments per unit width at mid span are given by

$$Mx = \alpha x \times Wl^2x$$
$$My = \alpha y \times Wl^2x$$

where Mx, My, W, Lx and Ly are the same those in C(i) and α x and α y are moment coefficients to be read from the table below.

iii) BM Coefficients for slabs spanning in two directions at right angles, simply supported on four sides :-

Ly/Lx	: 1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0	2.5	3.0
α x	:0.062	0.074	0.084	0.093	0.099	0.104	0.113	0.118	0.122	0.124
αy	:0.062	0.061	0.059	0.055	0.051	0.046	0.037	0.029	0.020	0.014

Generally the main reinforcement bars are continued up to end of the supports without curtailment.

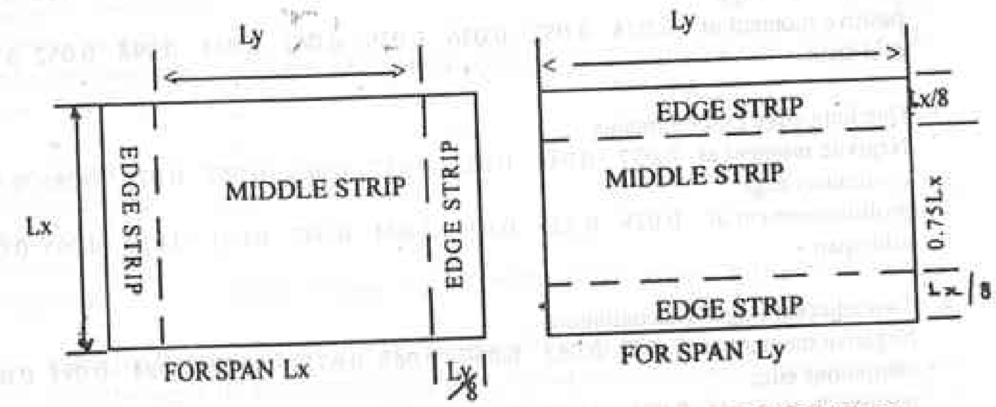

FIGURE 1.2.1 :- DIVISION OF A SLAB INTO MIDDLE AND EDGE STRIPS

TABLE 1.2.1

BENDING MOMENT CO EFFICIENTS FOR RECTANGULAR PANELS SUPPORTED ON FOUR SIDES WITH PROVISION FOR TORSION AT CORNERS IS : 456 - 1978

Case no	Type of Panel and Moments considered	short span Co efficient α x (value of ly/lx) 1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0	Long span Co efficients αy for all values of ly/lx
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
1	Interior Panels : Negative moment at continuous edge	0.032	0.037	0.043	0.047	0.051	0.053	0.060	0.065	0.032
	positive moment at mid-span	0.024	0.028	0.032	0.036	0.039	0.041	0.045	0.049	0.024

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(1)	(2)	1.0 (3)	1.1 (4)	1.2 (5)	1.3 (6)	1.4 (7)	1.5 (8)	1.75 (9)	2.0 (10)	(11)
2.	One short edge Discontinuous :									
	Nagavite moment at continuous edge	0.037	0.043	0.048	0.051	0.05	0.057	0.064	0.068	0.037
	pasitive moment at mid-span	0.028	0.032	0.036	0.039	0.041	0.044	0.048	0.052	0.028
3.	One long edge Discontinuous :									
	Negivite moment at continuous edge	0.037	0.044	0.052	0.057	0.063	0.067	0.077	0.085	0.037
	positive moment at mid-span	0.028	0.033	0.039	0.044	0.047	0.051	0.059	0.065	0.028
4.	Two adjecent Edges Discontinuous :									
	Negative moment at continuous edge	0..047	0.053	0.060	0.065	0.071	0.075	0.084	0.091	0.047
	positive moment at mid-span	0.035	0.040	0.045	0.049	0.053	0.056	0.063	0.069	0.035
5.	Two short edgs Discontinuous :									
	Negative moment at continuous edge	0.045	0.049	0.052	0.056	0.059	0.060	0.065	0.069	-
	positive moment at mid-span	0..035	0.037	0.040	0.043	0.044	0.045	0.049	0.052	0.035
6.	Two long edgs Discontinuous :									
	Nagetive moment at continuous edge	-	-	-	-	-	-	-	-	0 .045
	positive moment at continuous	0.035	0.043	0.051	0.057	0.063	0.068	0.080	0.088	0.035
7.	Three edges Discontinuous (One long edge continuous) :									
	Negative moment at continuous edge	0.057	0.064	0.071	0.076	0.080	0.084	0.091	0.097	-

	positive moment at mid-span	0.043	0.048	0.053	0.057	0.060	0.064	0.069	0.073	0.043
8.	Three Edges Discontinuous (one short edge continuous)									
	Negative moment at continuous edge	--	--	--	--	--	--	--	--	0.057
	Positive moment at mid - span	0.043	0.051	0.059	0.065	0.071	0.076	0.087	0.096	0.043
9.	Four Edges Discontinuous: Positive moment at mid-span	0.056	0.064	0.072	0.079	0.085	0.089	0.100	0.107	0.056

1.2.0.5 Effective Depths and Widths of - Beams(or Slabs)

Effective depth d= Distance between the centroid of area of tensile reinforcement and the (top of slab) Maximum compression fibre excluding finishings such as tiles, mossaic finish etc.,

T-Beams and L-Beams: Effective width of flanger :-

a) T-Beams : bf = Lo/6 + bw + 6D f (not more than bw + 1/2 clear distance between adjacent ribs(beams).

b) L-Beams : bf = Lo /12 + bw + 3Df (-- Do --)

bf = effective width of flange

Lo = Distance between zero moments in the beam (distance between two consecutive points of contraflexure = 0.7 times effective span, lef).

bw = bredth of web or rib

Df = thickness of flange = thickness of slab itself

1.2.0.6 Deflection - span / Depth Ratios

a) Deflection :- Not more than span / 350 or 20 mm which ever is less (after partitions etc., are built and finishes completed including temperature effects etc.)

Maximum span / effective depth ratios :- (Beams)

1. Satisfying the deflection lilmit upto 10m spans (beams & slabs)

40

11. Span not greater than 10m.

Cantilever	7
Simply supported	20
Continuous	26

Note :

1. For spans above 10 m, the above values may be multiplied by 10 / span in metres (in case of cantilevers, actual deflection may be calculated).
2. For tension reinforcement used further modify the above value by multiplying with the modification factor as per fig..1.2.2
3. For compression reinforcement used, modify the above values Sarab arm as per fig..1.2.3
4. For flanged beams the values of (a) or (b) be modified as per Fig. 1.2.4

b) Solid slab : It is a slab in roofs or floors or in irrigation structures as distinct from flat slabs. (Flat slabs are slabs supported by columns but not by beams with or without flared heads of columns). Generally we come accross solid slabs only.

c) Maximum span / overall depth ratios for small, shorter spans upto 3.5 m lengths :- slabs : For loadings upto 300 kgs / m² (3000 N/m² = 60 lbs / sft)

	Two way slabs with MS	with HYSD	slabs spanning in one direction with M S	with HYSD
Simply supported slabs	35	0.8x 35=28	30	24
Continuous slabs	40	0.8x 40=32	35	32
Cantilever	-		12	10

d) Slenderness limits for beams to ensure lateral stability :- A simply supported /continuous beam shall be so proportioned that the clear distance between the lateral restraints does not exceed 60 b or 250 b² / d whichever is less. (b=breadth of compression face midway between lateral restraints, d = effective depth of beam).

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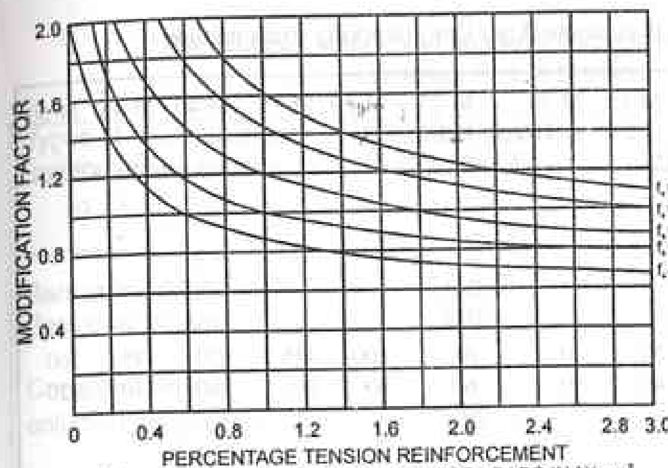

Note : f_s IS STEEL STRESS OF SERVICE LOADS IN N/mm²

$$f_s = 0.58 f_y \frac{\text{Area of cross-section of steel required}}{\text{Area of cross-section of steel provided}}$$

Fig. 1.2.2. MODIFICATION FACTOR FOR TENSION REINFORCEMENT

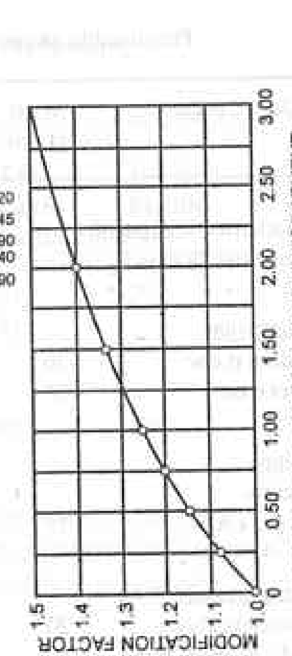

Fig. 1.2.3. MODIFICATION FACTOR FOR COMPRESSION REINFORCEMENT

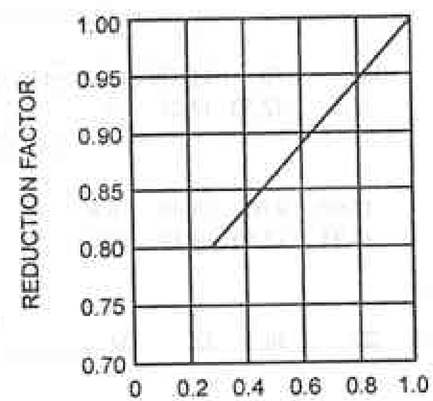

Fig. 1.2.4. RATIO OF WEB WIDTH TO FLANGE WIDTH. REDUCTION FACTORS FOR RATIOS SPAN TO EFFECTIVE DEPTH FOR FLANGED BEAMS

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TABLE 1.2.2

Permissible stresses in concrete (kgs / Cm^2) (working stress method)

Grade of Concrete	M-10 (1:3:6)	M-15 (1:2:4)	M-20 (1:1 1/2:3)	M-25 (1:1:2)	M-30	M-35	M-40	M-45	M-50
Characteristic strength of 15 cm^3 after 28 days f_{ck}									
Compression									
Bending σ cbc	30	50	70	85	100	115	130	145	160
Direct σ cc	25	40	50	60	80	90	100	110	120
Tension									
Direct σst (Ft / Ac. + Ast)	12	20	28	32	36	40	44	48	52
Bearing pressure (Concrete) only σ_{br}	20	30	40	50	60	70	80		
Bond : Average bond for plain M S bars									
in tension	4	6	8	9	10	11	12	13	14
in compression	5.0	7.5	10	11.2	12.50	13.25	15		
For HYSD (T40) bars									
in Tension	5.6	8.4	11.2	12.60	4.00	15.40	16.8		
in compression	7.0	10.5	14.0	15.75	17.50	19.25	21.0		
Modulus of rupture by beam test after 07 days	17	21	24	27	30	32	34		
Modular Ratio, m (corrected to nearest integer)	30	18	13	11	9	8	7		

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TABLE 1.2.3

Permissible stresses in steel reinforcement (in N/mm^2)
In Working Stress Method

Types of stress in steel reinforcement tension (σ_{st})	Mild steel grade1 1S:432/1966	HYSD Tor 40 **Fe 415** (IS:1786/1979)	Tor 50 **Fe 500**
Bars upto 20mm	140	230	275
Bars over 20mm	130	230	
Compression in column bars (σsc)	130	190	230
Compression in bars in a beam or slab when the compressive resistance of concrete is i) taken into account ii) not taken into account	The calculated compressive stress in the surrounding concrete multiplied by 1.5 times the modular ratio = (1.5 mσ cbc) or σsc		
upto and including 20 mm	140	190	190
over 20 mm	130	190	190

TABLE 1. 2. 4
Design stresses of Tor steel Tor 40 (In N / mm²)
(in different contexts)

Type of stress	IRC Bridge code by ministry of ransport	IS 3370 water retaining structures	IS 456 ultimate load design	General buildings (elastic design)
a)Tension	190	150 190 (On face away from liquid for members thicker than 225mm)	425	230
b) Compression	160	175	370	190
c)Tension in shear reinforcement	30	150 (thin member) lesser than 225mm	-	230
d)Tension in spiral reinforcement	150	175 (members thicker than 225 mm)	-	160

1.2.1.0 Slabs carrying concentrated load

If a solid slab supported on two opposite edges, carries concentrated loads, the maximum bending moment caused by the concentrated loads shall be assumed to be resisted by an effective width of slab (measured parallel to the suppporting edges) as follows :

a) For a single concentrated load, the effective width shall be calculated in accordance with the following equation provided that it shall not exceed the actual width of the slab :

$$bef = kx\,(1 - x / lef) + a$$

where

bef	=	effective width of slab,
k	=	constant having the values given in Table 1.2.5 depending upon the ratio of the width of the slab (1") to the effective span.lef,
x	=	distance of the centroid of the concentrated load from nearer support,
lef	=	effective span and ,
a	=	width of the contact area of the concentrated load measured parallel to the supported edge.

And provided further that in case of a load near the unsupported edge of a slab, the effective widths shall not exceed the above value nor half the above value plus the distance of the load from the unsupported edge.

b) For two or more concentrated loads placed in a line in the direction of the span, the bending moment per metre width of slab shall be calculated separately for each load according to its appropriate effective width of slab calculated as in (a) above and added together for design calculations.

c) For two or more loads not in a line in the direction of the span, if the effective width of slab for one does not overlap the effective width of slab for another load, both calculated as in (a) above, then the slab for each load can be designed separately. If the effective width of slab for one load overlaps the effective width of slab for an adjacent load, the overlapping portion of the slab shall be designed for the combined effect of the two loads.

TABLE 1.2.5

Value of k for concentrated loads in slabs

l / lef	k for simply supported slabs	k for continuous slabs
0.1	0.4	0.4
0.2	0.8	0.8
0.3	1.16	1.16
0.4	1.48	1.44
0.5	1.72	1.68
0.6	1.96	1.84
0.7	2.12	1.96
0.8	2.24	2.08
0.9	2.36	2.16
1.0 and above	2.48	2.24

1.2.2.0 <u>Development length of Bars :-</u>

The calculated tension in any bar at any section shall be developed on each side of the section by an appropriate development length or end anchorage or by a combination thereof.

$L_d = \phi \cdot \sigma_s / 4\,\tau_{bd}$

L_d = Development length including anchorage value of Hooks in tension.

ϕ = Nominal diameter of bar.

τ_{bd} = Design bond stress

σ_s = Stress in bar at section considered at design load.

1.2.3.0 <u>Lap length including anchorage value of Hooks</u>

1. In flexural tension - Shall be Ld (or) 30 ϕ which ever is greater.
2. In direct tension - 2Ld (or) 30ϕ whichever is greather.
3. in compression - shall be equal to Ld but not less than 24 ϕ
   - where Ld = development length as described in1. 2.2.0.

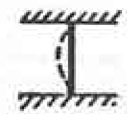
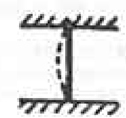
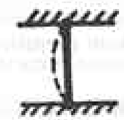
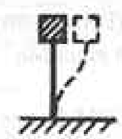
The straight length of shall not be less than 15 ϕ (or) 20 cm.

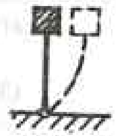
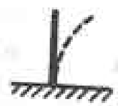
Anchorage value of Hooks : - U - Hook = 16 ϕ

L - (or) 45° bend = 4 ϕ

TABLE 1.2.6

Effective length of compression members IS : 456 - 78

Degree of End Restraint of Compression Member	Symbol	Theoretical value of effective length	Recommended value of effective length
(1)	(2)	(3)	(4)
Effectively held in position & Restrained against rotation both ends.		0.5L	0.65L
Effectively held in position at both ends restrained against rotation at one end.		0.7L	0.80L
Effectively held in position at both ends, but not restrained against rotation		1.00L	1.00L
Effectively held in position & restrained against rotation at one end, and at the other restrained against rotation but not held in position.		1.00L	1.20L
Effectively held in position and restrained against rotation at one end , and at the other partially restrained against rotation but not held in position.		—	1.50L

(1)		(2)	(3)	(4)
Effectively held in position at one end but not restrained against rotation at one end, and at the other end restrained against rotation but not held in position.			2.00L	2.00L
Effectively held in position and restrained against rotation at one end but not held in position nor restrained against rotation at the other end.			2.00L	2.00L

Note: L is the unsupported length of compression member.

1.2.4.0. Permissible loads in compression Members:

Pedastals/ shortcolumns with lateral ties :

Permissible axial load : P= σcc Ac + σsc Asc

Note: The minimum eccentricity of ((L/500 +(D or b)/30)) subject to a minimum of 20 mm may be deemed to be incorporated in the above equation.

Long columns : lef / least lateral dimension > 12(15 as per CP.110,117)

a) Correction factor for C_r =1.25 - lef/48b , b = least lateral dimension; permisible stress

b) More precisely, lef / least lateral radius of gyration > 40

Correction factor for permissible stressesC_r = 1.25 -(lef/160 r min)

r min = least radius of gyration = $\sqrt{I/A}$

L/D -	12	15	20	25	30	40	48	50	60	Remarks
a) Cr.	-	1.0	0.83	0.66	0.50	-	-	-	-	Using 15 as limit
b) Cr.	1.0	0.94	0.84	0.73	0.63	0.42	0.25	-	-	Using 12 as limit
c) Cr						1.0	0.94	0.87		Using 40 as Limit

1.2.5.0 <u>Requirement of Reinforcement for Structural Members:-</u>

A) <u>Beams:</u>

a) <u>Tension</u> - i) minimum A_{st} = 0.85 bd / f_y with HYSD bars (f_y = 2300) (0.6% to 6% in W.S. Method.

b = breadth of beam or rib

d = effective depth,

ii) Maximum 4% of gross area, (0.40 bD)

(0.8 to 8% in L.S.Method but due to space limitations and difficulty in compaction, restricted to 4% in all cases).

b) <u>Compression:</u> Minimum 0.8%; Maximum 6% of Ag (Total sectional area = Ag) but Maximum limited to 0.40 bD. Stirrups act as lateral restrainits.

c) <u>Side face reinforcement.</u>: Where the depth of the web in a beam exceeds 750 mm, side face reinforcement shall be provided along the two faces. The total area of such reinforcement shall be not less than 0.1 percent of the web area and shall be distributed equally on two faces at a spacing not exceeding 300 mm or web thickness whichever is less.

d) <u>Minimum shear reinforcement:</u> In the form of stirrups:

$$\frac{A_{sv}}{bs_v} \geq \frac{0.4}{f_y} =$$

A_{sv}= Total C.S.Area of stirrup legs effective in shear

Stirrup spacing along the length of member

f_y = 415 N/mm² in LSM or WSM for T 40 and 250 N/mm² with MS rods.

Note:0.4 may be taken as 4 kg/cm² in WSM; f_y = 4150 with T 40 or 2500 with MS rods

Note: For lintels etc., where Max shear stress < half the permissible value, this provision is not necessary.

2.5.1 (B) <u>Slabs:-</u> Minimum steel in either direction not less than 0.15% of total C.S. Area with M.S. or 0.12% with HYSD.

Minimum dia of the bar ⊁ 1/8D of the slab. Therefore eg., for a 15cm slab, ϕ (uncommon) ⊁ 150 / 8 = 18.75 mm)

C) <u>Columns Longitudinal reinforcement - compression:</u>- 0.8% to 6% but limited to 4% due to compaction placement and lapping difficulties. Minimum number of bars 4 in rectangular or square columns and 6 in circular columns. Bars shall not be less than 12 mm in dia.

In Helical reinforcement - minimum 6 bars

Spacing of main bars along periphery > 300mm

<u>Pedestals:</u> Compression members whose effective length is within three times the least lateral dimesion.

Steel area = 0.15% of cross section area.

<u>Traverse Reinforcement :</u>

Columns:

Spacing / Pitch ;

Least of the following:

i) Least lateral dimesion of compression member

ii) 16 ϕ (ϕ = smallest dia of longitudinal bar to be tied)

iii) 48 ϕ_t (ϕ_t = dia of traverse reinforcement)

Tie Dia: main /4 (1/4 maximum main bar), < 5 mm in any case.

Note: Shear Strength may be taken as 1.50 times the allowable value in flexural members.

1.2.5.1 D Members subject to combined axial load and bending (due to eccentricity of load, lateral forces ect)

a) σ cc.cal / σ cc. + σ cbc. cal / σ cbc. $\leqslant 1$

or

P/A + M/Z $\leqslant$ f $\leqslant$ σ cbc. allowable

$$\frac{\text{(calculated) Direct (axial) compressive stresses in concrete}}{\text{Permissible axial compressive stress in concrete}} + \frac{\text{Calculated bending compressive stress in concrete}}{\text{Permissible bending compressive stress in concrete}} \leqslant 1$$

b) The resultant tension in concrete 35% and 25% of the resultant compression for biaxial and uniaxial bending respectively.

75% of the 7 day modulus of rupture of concrete

(.75 x 21 = 15.75 kg/cm² for M 15 grade, say)

Note :- 1. σ cc.cal = $\dfrac{P}{Ac+1.5m}$ Asc

2 σ cbc.Cal = M/Z where M equals the moment and Z= Transformed Modulus section considering concrete and steel areas separately with application of modular ratio of m to the appropriate grade.

3. In case of sections subject to moments in two directions, the stress shall be calculated separately and added algebraically.

1. 2.5.2 Detailing Rules for Slabs , Beams and Columns :-

Right type of detailing of reinforcemnts in an essential requisite in the design and execution of RC members. Some of the factors of importance in detailing like quantum of reinforcement, range of diameter and spacing of the reinforcements are dealt with in brief in what follows.

A. Slabs

Based on the regulations in IS : 456 - 1978 and standard construction practice, the guidelines for slabs are summarised as below.

Slab reinforcements (Torsteel) (Common for Tor 40 and Tor 50)

Diameter		Min Max	6mm Least of : i) One eighth the over-all depth of slab ii) 16 mm
Quantum of Reinforcement		Min Max	0.12% of gross sectional area of slab 2% of gross sectional area of slab
Clear Spacing between	MAIN BARS	Min	Greatest of : i) 1.5 times the maximum size of coarse aggregate ii) Diameter of the needle where needle vibrators are used. iii) 5 cm
		Max	Lesser of : i) 45 cm ii) Three times the effective depth of slab
	Distribution bars (Max)		Lesser of : i) 45 cm ii) Five times the effective depth of slab.

B. Columns

Based on the regulations IS : 456-1978 and standard construction practice, the guidelines for columns are summarised in tables below.

Longitudinal Reinforcement in columns (Common for Tor 40 and Tor 50)

Area of reinf.	Min Max	0.8% of the gross sectional area of columns* 4.0% of the gross sectional area of columns
Diameter of bars	Min Max	12 mm 40 mm
Number of bars	Min	i) 4 bars for Rectangular columns ii) 6 bars for circular columns
Spacing of bars	Max	30 cm when measured along the perphery of the column

* In columns that have largest concrete cross sectional area than required, the above should be based upon the actual area of concrete required to resist the direct load acting on the column ; but in no case the reinforcement shall be less than 0.4% of the gross sectional area of the column.

Guidelines for column Ties

Min.diameter	Max.diameter	Max.spacing of ties
Greatest of i) 6 mm ii) One fourth the dia of the largest longitudinal bar	16 mm	Least of : i) Least lateral dimension of column ii) 16 times the dia of smallest longitudinal bar. iii) 48 times the dia of the ties.

SOME DETAILING RULES FOR R.C.BEAMS

Right type of detailing of reinforcements is an essential requisite in the design and execution of R.C. members. Some of the factors of importance in detailing are

(i) Quantum of reinforcement (ii) selection of suitable diameter and (iii) appropriate spacing of reinforcement. These are dealt with in brief in what follows for beams.

A. TENSION AND COMPRESSION REINFORCEMENTS

Based on the regulations in IS : 456- 2000 and standard construction practice, the requirements are furnished for various values of d/D where d is effective depth and D is overall depth. The table also gives information on side face reinforcement which should be provided when the total depth of section exceeds 75 cm.

Min. & Max. Reinforcement (Torsteel) in Beams

	Tension reinforcement with respect to gross sectional area (percentage)				Tension or compression reinforcement with respect to gross sectional area (percentage)	Side face reinforcement with respect to gross sectional area (percentage)
	MIN				MAX	MIN.
d/D	0.95	0.90	0.85	0.80	For all values	For all values (D>75 cm)
Tor 40	0.195	0.185	0.174	0.164	4.0	0.1
Tor 50	0.162	0.153	0.145	0.136	4.0	0.1

B. Shear Reinforcement :

The data summarised from the recommendations in IS : 456- 2000 are furnished below.

Minimum Shear Reinforcement (Torsteel) in Beams

	Breadth of beam (cm)					
	15	20	25	30	35	40
Tor 40	1.45	1.93	2.41	2.90	3.37	3.89
Tor 50	1.45	1.93	2.41	2.90	3.37	3.89
	(cm^2/m length of beam)					

C. The requirements in Tables above can be simplified for adoption in normal designs and below. This simplification is easy to adopt and remember and is applicable for all values of d/D.

Simplification : Min & Max (Torsteel) Reinforcement

	Tension reinforcement w.r.t. gross sectional area (Min.)	Tension or compression reinforcement w.r.t. gross sectional area (Max)	Side face reinforcement w.r.t. gross sectional area (D 75cm) (min)	Shear reinforcement w.r.t.plan area of Web (Min)
Tor 40	0.20%	4.0%	0.10%	0.10%
Tor 50	0.16%	4.0%	0.10%	0.10%

D. Spacing of longitudinal Bars and Shear Reinforcement from construction and cracking considerations, certain guide lines have been arrived at for the minimum and maximum clear spacing to be provided between bars in the beams. Similarly from construction and design considerations the minimum and maximum spacing of the stirrups have been arrived at. These are tabulated in below for the different grades of Torsteel.

Spacing to bars (Tor steel) in beams

Longitudinal reinforcement			Shear reinforcement	
Min Horizontal spacing between bars(Clear)	Max. Horizontal spacing between bars(Clear)	Min. Vertica spacing between bar layers(clear)	Min. spacing	Max. spacing
(Common for Tor 40 and Tor 50)		(Common for Tor 40 and Tor 50)	(Common for Tor 40 and Tor 50)	(Common for Tor 40 & Tor 50)
Greatest of: i) Diameter of the bar ii)Diameter of the larger bar wherein different diametersare used	For Tor 40: 18 cm Tor 50 : 15 cm (If the redistribution of moment is consideration from the section the modified values are : F or T or. 40:12 cm T or 50:10.5 cm	Greatest of: i) Diameter of the bar ii)Diameter of the larger bar where in different diameters are used	8 cm	Lesser of: i) 0.75 d ii) 45 cms
iii) 1.5 times the max size of the course aggregate		iii) 2/3 max.size of course aggregate		
iv) Dia of needle if needle vibrators are used.		iv) 1.5 cm		

1.2.5.3. Cover to Reinforcement :

1.2.5.3 (i) Reinforcement shall have concrete cover and the thickness of such cover (exclusive of plaster or other decorative finish) shall be as follows :

a) At each end of reinforcing bar not less than 25 mm nor less than twice the diameter of such bar.

b) For a longitudinal reinforcing bar in a column, not less than 40 mm nor less than the diameter of such bar. In the case of columns of minimum dimension of 200 mm or under whose reinforcing bars do not exceed 12mm a cover of 25 mm may be used.

c) For longitudinal reinforcing bar in a beam, not less than 25 mm nor less, than the diameter of such bar.

d) For tensile, compressive, shear or other reinforcement in a slab not less than 15 mm ,nor less than the diameter of such bar; and

e) For any other reinforcement, not less than 15mm nor less than the diameter of such bar.

1. 2.5.3 (ii) Special exposures-harmful chemicals, acid, saline atmosphere,Sulphurous smoke : 15 mm

b) Sea water - 40 mm more than specified in the 2.5.3 (i)

NOTE : Maximum cover - 75 mm.

1.2.5.4. Curtailment of Tension Reinforcement in Flexural Members :-

Roads to extend upto d or 12 ϕ whichever is greater from No tension zone except at supports. Where ϕ is diameter of bar.

<u>Positive moment reinforcement at supports :-</u>

At least 1/3 of + Ve moment reinforcement in simple members and 1/4 of the reinforcement in continuous beams shall extend into thesupport to a length Ld/3.

Negative moment reinforcement at supports :-

At least 1/3 of total reinforcement given for - Ve reinforcement at support shall extend beyond the point of inflection for a distance effective depth (d) or 12 or 1/16 clear span whichever is greater.

1.2.5.5. Shear (Torsion) Force :- Nominal shear stress τv, in beams or slabs of uniform depth = V/bd = S.F. due to design loads
(breadth of beam or web) x (effective depth)

Note : 1. Here effective depth is considered but not lever arm or Jd as per earlier codal practice (IS 456-1964).

2. Generally, we come across beams/slabs of uniform depth.

3. In the revised code, IS.456-1978, it may be seen, the permissible shear stress varis with percentage of steel reinforcement provided in the section along with the grade of mix. The method of design for shear and torsion by working stress method are similar to the limit State Method.

Working stress method :-

Shear- Permissible shear stress in beams using R.C.concrete in kg/cm² without shear reinforcement.

TABLE 1.2.7 (A)

i) BEAMS : Permissible shear stress τ c in kg/cm² in concrete without shear reinforcement.

100 Ast / bd (% of steel)	Grades of concrete M 15	M 20	M 25	M30	M35	M 40 (and above)
≤ 0.15	1.8	1.8	1.9	2.0	2.0	2.0
0.20	2.0	2.0	2.1	2.1	2.1	2.1
0.25	2.2	2.2	2.3	2.3	2.3	2.3
0.30	2.4	2.4	2.5	2.5	2.5	2.5
0.40	2.7	2.7	2.8	2.8	2.8	2.8
0.50	2.9	3.0	3.1	3.1	3.1	3.2
0.60	3.1	3.2	3.3	3.3	3.4	3.4
0.70	3.3	3.4	3.5	3.6	3.6	3.7
0.75	3.4	3.5	3.6	3.7	3.7	3.8
0.80	3.4	3.6	3.7	3.8	3.8	3.9
0.90	3.6	3.7	3.9	3.9	4.0	4.1

1.00	3.7	3.9	4.0	4.1	4.2	4.2
1.10	3.8	4.0	4.2	4.3	4.3	4.4
1.20	4.0	4.1	4.3	4.4	4.5	4.5
1.25	4.0	4.2	4.4	4.5	4.5	4.6
1.30	4.1	4.3	4.4	4.5	4.6	4.7
1.40	4.2	4.4	4.5	4.6	4.7	4.8
1.50	4.2	4.5	4.6	4.8	4.9	4.9
1.60	4.3	4.6	4.7	4.9	5.0	5.1
1.70	4.4	4.7	4.8	5.0	5.1	5.2
1.75	4.4	4.7	4.9	5.0	5.2	5.2
1.80	4.4	4.7	4.9	5.1	5.2	5.3
1.90	4.4	4.8	5.0	5.2	5.3	5.4
2.00	4.4	4.9	5.1	5.3	5.4	5.5
2.10	4.4	5.0	5.2	5.4	5.5	5.6
2.20	4.4	5.1	5.3	5.4	5.6	5.7
2.25	4.4	5.1	5.3	5.5	5.6	5.7
2.30	4.4	5.1	5.3	5.5	5.7	5.8
2.40	4.4	5.1	5.4	5.6	5.7	5.9
2.50	4.4	5.1	5.5	5.7	5.8	6.0
2.60	4.4	5.1	5.6	5.7	5.9	6.0
2.70	4.4	5.1	5.6	5.8	6.0	6.1
2.75	4.4	5.1	5.6	5.8	6.0	6.2
2.80	4.4	5.1	5.7	5.9	6.0	6.2
2.90	4.4	5.1	5.7	5.9	6.1	6.3
> 3.00	4.4	5.1	5.7	6.0	6.2	6.3

Note : Ast = Area of that steel in longitudinal tension reinforcement which continues to atleast one effective depth (d) beyond the section being considered except at support where the full area of tension reinforcement of a steel requirements i.e., upto d, 12 times dia of bar, or upto the point where the actual shear stress is not exceeded by 2/3 of the permissible value.

Maximum τ c (shear stress) in N/mm² allowable, even with transverse reinforcement for each mix.

TABLE - 1.2.7 (B)

Concrete Mix	M 15	M 20	M 25	M 30	M 35	M 40
τ c Max	1.6	1.8	1.9	2.2	2.3	2.5
τ c allowable in slabs in N/mm²	0.8	0.9	0.95	1.1	1.15	1.25

ii) Solid Slabs : K times τc given above where K is as given below :-

(D) Overall depth of slab in mm	300 & above	275	250	225	200	175	150 or less
k	1.0	1.05	1.10	1.15	1.20	1.25	1.3

Maximum τc allowable in slabs = Half the maximum value given above for each mix in beams;

	M15	M20	M25	M30	M35	M40
τc Max	0.8	0.9	0.95	1.1	1.15	1.25

Note :

1. For the remaining half of the shear, shear reinforcement is to be designed for beams.
2. For compressive Members in axial compression, permissible shear stree .. may be upto nearly 1.5 times the values given above for beams.
3. If they do not come under the above limits, better redesign the section with changed parameters.
4. For slabs, shear reinforcement is not usual, so the slab thickness will be modified accordingly

Table 1.2.7(c)

Design Shear strength of Concrete, τc N/mm² (IS : 456, 2000) (Limits State)

$\frac{100 Ast}{bd}$ (% of steel)	Concrete Grade M 15	M 20	M 25	M30	M35	M 40 (and above)
(1)	(2)	(3)	(4)	(5)	(6)	(7)
≤ 0.15	0.28	0.28	0.29	0.29	0.29	0.30
0.25	0.35	0.36	0.36	0.37	0.37	0.38
0.50	0.46	0.48	0.49	0.50	0.50	0.51
0.75	0.54	0.56	0.57	0.59	0.59	0.60
1.00	0.60	0.62	0.64	0.6	0.67	0.68
1.25	0.64	0.67	0.70	0.71	0.73	0.74
1.50	0.68	0.72	0.74	0.76	0.78	0.79
1.75	0.71	0.75	0.78	0.80	0.82	0.84
2.00	0.71	0.79	0.82	0.84	0.86	0.88
2.25	0.71	0.81	0.85	0.88	0.90	0.92
2.50	0.71	0.82	0.88	0.91	0.93	0.95
2.75	0.71	0.82	0.90	0.94	0.96	0.98
3.00 and above	0.71	0.82	0.92	0.96	0.99	1.01

NOTE – The term A is the area of longitudinal tension reinforcement which continues at least one effective depth beyond the section being considered except at support where the full area of tension reinforcement may be used provided the detailing conforms to 26.2.2 and 26.2.3

Note : The term As is the area of longitudinal tension reinforcement which continuesat least one effective depth beyond the section being considered except at supports where the full area of tension reinforcement may be used.

Table 1.2.7 (D)

Maximum Shear Stress, τc max N/mm2 (Limit State)

Concrete Grade	M 15	M 20	M 25	M 30	M 35	M 40
τc max, N/mm²	2.5	2.8	3.1	3.5	3.7	4.0
τc allowable in slabs	1.25	1.4	1.55	1.75	1.8	2.0

Critical section for Shear :-

i) Maximum shear generally occurs at the face of the support or

ii) At a distance less than d from the face of the support when the reaction in the direction of the applied shear force introduces compression into the end region of the member.

Generally shear reinforcement is accomplished through vertical stirrups with double legs (or 3 to 4 legs) depending on its intensity and bent up bars along with the stirrups.

Where bent up bars are provided, not more than 50% of the total shear resistance, through shear reinforcement, be shared by such bent bars.

i.e., $\frac{(V\text{-}c\ bd)}{2}$

a) Strength of shear reinforcement :- for vertical stirrups.

$$Vs = \frac{\sigma sv.\ Asv.d}{sv.}$$

b) for single bar or single group of parallel bars (more than one also)all bent up at the same cross section

Vs : σ sv . Asv Sin α

σ v in Kgs = (2300 or 1400) x Area of two legs of the stirrups (for tor 40,50 , or for M.S) or bent up bars within a distance Sv. x sin α and $\sin\alpha = 1/\sqrt{2} = 0.707$. for $\alpha = 45^o$

Note :- The spacing limitations for nominal shear reforcement are :-
1) 0.75 d, 11) 450 mm or 111) $\frac{A_{sv}}{bs_v} \geq \frac{0.4}{f_y}$ whichever is less as stated earlier.

1.2.5.6. Shear and Torsion :

Equivalent shear = equivalent shear, Ve shall be calculated from the formula :

$$Ve = V + 1.6\ Tu/b$$

where,

Ve, = equivalent shear
V = shear
Tu = torsional moment, and
b = Breadth of beam

The equivalent nominal shear stress τve , in this case shall be calculated as V = V/bd except for substituting V by Ve ie., τve = Ve/bd = (1/bd). (V+1.6T/b) (substituting V by Ve). the values of τ ve shall not exceed the value of τc max given in Table 1.2.7 (B)

If the equivalent nominal shear stress τ ve does not exceed τc, given in Table 1.2.7 A minimum shear reinforcement shall be provided as specified in 1.2.5.5 (B)

If τ ve exceeds τc given in table 1.2.7 A both longitudinal and transverse reinforcement shall be provided in accordance with para below:

<u>Reinforcement in Member Subjected to Torsion</u>

Reinforcement for torsion, when required, shall consist of longitudinal and transverse reinforcement.

<u>Longitudinal Reinforcement -</u> The longitudinal reinforcement shall be designed to resist an equivalent bending moment, M_{e1} givenby

$M_e = M + M_t$

where

M = bending moment at the cross section, and

$M_t = \frac{T(1 + D/b)}{1.7}$ where T isthe torsional moment, D is the overall depth of the beam and b is the breadth of the beam.

If the numerical value of M_t as defined in previous para exceeds the numerical value of the moment M, longitudinal reinforcement shall be provided on the flexural compression face, such that the beam can also with stand an equivalent moment M_{e2} given by $M_{e2} = M_t - M$, the moment

Me2 being taken as acting in the opposite sense to the moment M.

<u>Transverse Reinforcement -</u> Two legged closed hoops enclosing the corner longitudinal bars shall have an area of cross section Asv given by

$$Asv = Ts_v / (b_1 d_1\ \sigma sv) + V.s_v / (2.5\ d_1 \sigma\ s_v)$$

but the total transverse reinforcement shall not be less than

$$(\tau ve - \tau c)\ .\ b.S_v\ /\ \sigma sv$$

where T = torsional moment
V = shear force,
S_v = spacing of the stirrup reinforcement
b_1 = centre to centre distance between corner bars in the direction of the width,
d_1 = centre to centre distance between corner bars in the direction of the depth,
b = breadth of the member,
σ sv = permissible tensile stress in shear reinforcement
τ ve = equivalent shear stress as specified in 1.2.5.6
τ c = shear stength of the concrete as specified in Table 1.2.7 (C)

<u>Distribution of Torsion reinforcement :-</u> When a member is designed for torsion reinforcement shall be provided as below :

a) The transverse reinforcement for tension shall be rectangular closed stirrups placed perpendicular to the axis of the member. The spacing of the stirrups shall not exceed the least of x_1, $(x_1+y_1)/4$ and 300mm, where x_1 and y_1 are respectively the short and long dimensions of the stirrup.

b) Longitudinal reinforcement shall be placed as close as is practicable to the corners of the cross-section and in all cases, there shall be at least one longitudinal bar in each corner of the ties. When the cross-sectional dimension of the member exceeds 450mm, additional longitudinal bars shall be provided to satisfy the require -ments of minimum reinforcement and spacing given in side face reinforcement.

Reinforcement in flanges of T and L beams shall satisfy the requirements in 2.5.2. where flanges are in tension, a part of the main tension reinforcement shall be distributed over the effective flange width or a width equal to one tenth of a span, whcihever is smaller. If the effective flange width equal to one tenth of the span, whichever is smaller. If the effective flange width exceeds one tenth of the span, nominal longitudinal reinforcement shall be provided in the outer portions of the flange.

Slabs :-

Minimum reinforcement - The reinforcement in either direction in slabs shall not be less than 0.15 percent of the total cross-sectional area. However, this valve can be reduced to 0.12 percent when high strength deformed bars or welded wire fabric are used.

Maximum diameter - The diameter of reinforcing bars shall not exceed one eighth of the total thickness of the slab.

1.2.6.0 Design constants for critical sections :-

Table 1.2.8 : Design constants for a balance section with Mild steel IS 432-1960

(t : 1400 kg/cm² for bars upto 20 mm and 1300 kg/cm2 for bars above 20mm)

Grade of concrete	Permissible compressive stress in bending in kg/cm²	m	K	J	Q	Pt (%) of steel $\frac{100 A_s}{bd}$
M 10 (1:3:6)	30	31	0.40	0.87	5.19	0.43
M 15 (1:2:4)	50	18	0.40	0.87	8.70	0.72
M 20 (1:1 1/2:3)	70	13	0.40	0.87	12.10	0.99
M 25 (1:1:2)	85	11	0.40	0.87	14.74	1.21
M 30	100	9	0.40	0.87	17.00	1.40
M 35 Psc	115	8	0.40	0.87	19.82	1.63
M 40	130	7	0.40	0.87	22.25	1.83

These Tables 1.2.8 and 1.2.9 are only for the general guidance in design.

Table 1.2.9 : Design constants for critical section with Tor Steel (Fe 415)

	σ cbc	K	j	Q
			t = 2300 kg/cm² for all sizes	
M 10	30	0.289		
M 15	50	0.289	0.904	6.50
M 20	70	0.289	0.904	9.10
M 25	85	0.289	0.904	11.10
M 30	100	0.289	0.904	13.00
M 35	115	0.289		-
M 40	130	0.289		-

Table 1.2.10 : Design constants for balanced sections using torsteel 40 used in different contexts such as (a) tension (b) compression (c) water retaining structures etc., for M 15/ M 20 mixes.

	Mix M 15						Mix M 20					
	C	t	m	n	j	Rbd²	C	t	m	n	j	Rbd2
a)	50	2300	18	0.281	0.906	6.365	70	2300	13	0.283	0.905	8.964
b)	50	1900	18	0.321	0.892	7.158	70	1900	13	0.324	0.892	10.115
c)	50	1500	18	0.375	0.875	8.203	70	1500	13	0.378	0.874	11.563*

* 3.06 for uncreacked section

1.2.7.0 EXPANSION JOINTS :

For the purpose of general guidance, however, structures exceeding 45m in length may be divided by one or more expansion joints.

1.2.7.1 WATER RETAINING STRUCTURES - General Rules

1. Water retaining structures must be built with impermeable Walls which will not creack under storage or due to shrinkage and temperature stresses.
2. Since tensile stress in the surrounding concrete must be kept under control high strength steel may not prove to be economical, except in rectangular tanks and mild steel reinforcement is

recommended.

3. Minimum reinforcement for temperature and shrinkage shall be provided. The coefficient of expansion due to temperature may be taken 11×10^{-6} /C. Coefficient of shrinkage may be taken as 450×10^{-6} for initial shrinkageand 200×10^{-6} for drying shrinkage. When allowance is made for shrinkageallowable stresses may be increased by 33 1/3 percent.

4. Special attention should be given to flexible joints, contraction joints,expansion joints and temporary construction joints.

5. The concrete mix generally used is M200, preferably of the controlled concrete based on a good mix design.

6. Drop in water level in underground water tanks 40mm in seven days.

7. Protection against conrrosion must be ensured. For liquid faces, minimumcover is 25mm and for seawater an additional cover of 12mm shall be provided, but the same shall not be included in calculation of strength.

8. The minimum reinforcement in walls, floors and roofs in each of twodirections at right angles shall have an area of 0.3 per cent of the concrete cross section in that direction for sections upto 10 cm. thickness. For thickness greater than 10 cm and less than 45 cm the minimum reinforcement in each of the two directions shall be linearly reduced from 0.3 per cent for 10 cm thickness to 0.2 per cent for thickness greater than or equal to 45 cm thickness. In concrete sections of 225 mm thickness or more two layers of steel shall be placed, one near each face of the section to make up the minimum reinforcement specified.

1.2.7.2 GENERAL REQUIREMENTS

Design and construction of reinforced concrete liquid retaining structures shall comply with the requirements of IS : 3370 (Part I - 1965) *

1.2.7.3 DESIGN

1. General - Provision shall be made for conditions of stresses that may occur in accordance with principles of mechanics, recognised methods of design and sound engineering practice. In particular, adequate consideration shall be given to the effects of monolithic construction in the assessement of bending moment and shear.

* *Code of Practice for concrete structures for the storage of liquids Part-I- General requirements*

\+ *Code of practice for Plain and Reinforces Concrete (Fourth Revision)*

2. Before taking up the detailed design, the designer should satisfy himself on the correct estimation of loads and on the adequate statiscal equilibrium of the structure particularly in regard to safety against overturning of over hanging members; in the latter case the general arrangement should be such that statical equilibrium should be satisfied even when the overturning moment is doubled.

1.2.7.4 BASIS OF DESIGN

1. General Basis or design shall be in line with the recommendations of IS:456- 2000+ except where otherwise specified in this code. The parts of the structure neither in contact with the liquid on any face nor enclosing the space above the liquid, as in case of stating of a water tower, shall be designed in accordance with the requirements of IS : 456- 2000 +

2. Design of members other than those excluded by 3.2.1 shall be based on consideration of adequate resistance to cracking as well as adequate strength.Calculation of stresses shall be based on the following assumptions in addition to the general assumptions given in IS :456- 1978 +

a. In calculations for both flexure and direct tensions (or combination of both) relating to resistance to cracking, the concrete is capable of sustaining limited tensile stress, thewhole section of concrete including the cover, together with the reinforcement being taken into account.

b. The total shear stress given by the following equation shall not exceed the value given in 2.8.10 whatever the reinforcement provided ;

Total shear stress = Q / bjd

where Q = Total shear
b = breadth, and
jd = lever arm.

c. In strength calculations the concrete has no tensile strength.

3. Plain concrete structures : Plain concrete members of reinforced concrete liquid, retaining structures may be designed against structural failure by allowing tension in plain concrete as per the permissible limits for tension in bending specified in IS:456 + 1978. This will automatically take care in accordance with the requirement of IS:456 = 1978 shall be provided for plain concrete structural members.

1.2.7.5 PERMISSIBLE STRESSES IN CONCRETE

1. For resistance to cracking :- For calculations relating to the resistance of members to cracking, the permissible stresses in tension (direct and due to bending) and shear shall conform to the value specified in Table 1.2.11. The permissible tensile stresses due to bending apply to the face of the member in contact with the liquid. In members less than 225 mm thick

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and in contact with the liquid on one side, these permissible stresses in bending apply also to the face remote form the liquid.

Table 1.2.11 : Permissible Concrete Stress in Calculations Relating to Resistances to Cracking

Grade of concrete'	Permisible stress Kg/cm² Direct tension	Tension due to bending	Shear (Q/b jd)
M15	11	15	15
M20	12	17	17
M25	13	18	19
M30	15	20	22
M35	16	22	25
M40	17	24	27

2. For strength calculations :- In strength calculations , the permissible concrete stresses shall be in accordance with IS: 456-1978. Where the calculated shear stress in concrete alone exceeds the permissible value, reinforcement acting in conjunction with diagonal compression , in the concrete shall provided to take the whole of the shear.

1.2.8.11 PERMISSIBLE STRESS IN STEEL

1) For resistance to cracking :- When steel and concrete are assumed to act ogether for checking the tensile stress in concrete for avoidance of cracks, the tensile stress in the steel will be limited by the requirement that thepermissible tensile stres in the concrete is not exceeded; So the tensile stress in steel shall be equal to the produce of modular ratio of steel and concrete and the corresponding allowance tensile stress in concrete.

2) For strength calculations :- For strength calculations the permissible stresses in steel reinforcement shall be as given in Table 1.2.12.

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TABLE 1.2.12 PERMISSIBLE STRESSES IN STEEL REINFORCEMENT FOR STRENGTH CALCULATIONS.

S.No.	Type of stress in steel reinforcement.	Permissible stresses in kg/cm2 — Plain round mild steel bars comforming to Gr.I.of IS:432 (part.I 1966)	High yeild strength deformed bars conforming to IS:1786 or IS:1139-1966.
1	2	3	4
I	Tensile stress in members under direct tension	1150	1500
II	Tensile stress in members bending		
a)	On liquid retaining face of members	1150	1500
b)	On faceway from liquid for members less than 225mm	1150	1500
c)	On face away from liquie for members 225mm or more in thickness	1250	1900
III	tensile stress in shear reinforcement		
a)	For members less than 225mm thick	1150	1500
b)	For members 225mm or more in thickness	1250	1750
iv	Compressive stress in columns subjected to direct load	1250	1750

NOTE :

Stress limitations for liquid retaining faces shall also apply to the following.

a) Other faces within 225mm of the liquid retaining force.

b) Outside or external faces of structures away from the liquid but placed in water logged soils upto the level of the highest subsoil water level.

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SECTION III - DESIGN TABLES

TABLE 1.3.1 IMPOSED FLOOR LOADS FOR DIFFERENT OCCUPANCIES (CLAUSES 3.1, 3.1.1 AND 4.1.1) [As per IS : 875 (Part 2) - 1987]

SL No	OCCUPANCY CLASSIFICATION	UNIFORMLY DISTRBUTEED LOAD (UDL)	CONCENTRATED
1	2	3 kN/M²	4 kN
I	RESIDENTIAL BUILDINGS		
	a) Dwelling houses:		
	1)All rooms and kitchen	2.0	1.8
	2)Toilet and bath rooms	2.0	--
	3)Corridors,passages,stair -cases including fire escapes	3.0	4.5
	4) Balconies	3.0	1.5 per meter run concentrated at the outer edge.
	b) Dwelling units planned and executed in accordance with IS: 8888-1979* only.		
	1) Habitable rooms, kitchens, toilets and bathrooms	1.5	1.4
	2) Corridors,passages,and staircases including fire escapes	1.5	1.4
	3) Balconies	3.0	1.5 per meter run concentrated at the outer edge.
	c) Hotels, hostels,bording houses Lodging houses, dormitories, residential clubs:		
	1) Living rooms, bed rooms,and dormitories	2.0	1.8
	2) Kitchens,and laundries	3.0	4.5
	3) Billiards room and public lounges	3.0	2.7
	4) Store rooms	5.0	4.5
	5)Dining rooms,cafeterias and restaurants	4.0	2.7
	6) Office rooms	2.5	2.7
	7) Rooms for Indoor games	3.0	1.8
	8) Bath rooms and toilets 2.0		
	9) Corridors,passages,stair cases, fire escapes, lobbies -- as per the floor serviced (excluding stores and the like) but not less than	3.0	4.5
	10)Balconies	Same as rooms to which they give access but with a minimum of 4.0	1.5 per meter run concent-rated at the outer edge.
	d) Boiler rooms and plant rooms-- to be calculated but not less than	5.0	6.7
	e) Garages:		
	1)Garage floors(inluding parking area and repair workshops) for passengers cars and vehicles not exc -ee ding 2.5 tonnes gross wieght, including access ways and ramps - to be	2.5	9.0

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calculated but not less than		
2) Garage floors for vehicles not exceeding 4.0 tonnes gross weight (including access ways and ramps) - to be calculated but not less than	5.0	9.0
ii. EDUCATIONAL BUILDINGS		
a)Class rooms and Lecture rooms (not used for Assembly purposes) +	3.0	2.7
b)Dining rooms, cafeterias, and restaurants	3.0	2.7
c)Offices, lounges staff rooms	2.5	2.7
d) Dormitories	2.0	2.7
e)Projection rooms	5.0	-
f)Kitchens	3.0	4.5
g)Toilets and bathrooms	2.0	-
h)Store rooms	5.0	4.5
j) Libraries and Archieves :		
1) Stack room / stack area	6.0 kN/m² for a minimum height of 2.2 + 2.0 kN/m² per metre height beyond 2.2 m.	4.5
2) Reading rooms (without separate storage)	4.0	4.5
3)Reading rooms (with separate storage)	3.0	4.5
k) Boiler rooms and plant rooms-to be calculated but not less than	4.0	4.5
m) Corridors ,passages, lobbies , staircases including fire escapes as per the floor serviced (without accounting for storage and projection rooms) but not less than	4.0	4.5

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n) Balconies	Same as rooms which they give access but with a minimum of 4.0	1.5 per mtr run concentrated at the outer edge
iii) INSTITUTIONAL BUILDINGS		
a) Bad rooms, wards, dressing rooms, dormitories and lounges	2.0	1.8
b) Kitchens, laundries and laboratories	3.0	4.5
c) Dining rooms,cafeterias and restaurants	3.0+	2.7
d) Toilets and bathrooms	2.0	---
e) X-ray rooms,operating rooms, general storage areas-to be calculated but not less than	3.0	4.5
f) Office rooms and OPD rooms	2.5	2.7
g) Corridors, passages, lobbies and staircases including fire escapes- as per the floor servised but not less than	4.0	4.5
h) Boiler rooks and plant rooms- to be calculated but not less than	5.0	4.5
j) Balconies	Same as rooms which they give access but with a minimum of 4.0	1.5 per mtr run concentrated at the outer edge
(iv) ASSEMBLY BUILDINGS		
a) Assembly areas :		
1) With fixed seats #	4.0	
2) With out fixed seats	5.0	3.6
b) Restaurants (subject to assembly) museums and art gallaries and gymnasia	4.0	4.5

c) Projection rooms	5.0	-
d) Stages	5.0	4.5
e) Office rooms, kitchens and laundries	3.0	4.5
f) Dressing rooms	2.0	1.8
g) Lounges and billiards rooms	2.0	2.7
h) Toilets and bathrooms	2.0	-
j) Carridors, passages, staircases including fire escapes	4.0	4.5
k) Balconies	Same as rooms which they give access but with a minimum of 4.0	1.5 per mtr run concentrated at the outer edge
i) Boiler rooms and plant rooms including weight of machinery	7.5	4.5
m)Corridors, passages subject to loads greater than from crowds, such as wheeled vehicles, trolleys and the like, corridors, staircases and passages in grandstands	5.0	4.5

(V) BUSINESS AND OFFICE BUILDINGS (See also 3.1.2)

a) Rooms for general use with seperate storage	2.5	2.7
b) Rooms without separate storage	4.0	4.5
c) Banking halls	3.0	2.7
d) Business computing machine rooms (with fixed computers or similar equipmnet)	3.5	4.5
e) Records/files store rooms and storage space	5.0	4.5
f) Vaults and strong room - to be calculated but not less than	5.0	4.5

g) Cafeterias and dining rooms	3.0	2.7
h) Kitchens	3.0	2.7
i) Corridors, passages, lobbies and staircases including fire escapes as per the floor serviced (exlcuding stotes) butnot less than	4.0	4.5
j) Bath and toilet rooms	2.0	
k) Balconies	Same as rooms which they give access but with a minimum of 4.0	1.5 per mtr run concentrated at the outer edge
l) Stationary stores	4.0 foreach metre of storage height	9.0
m)Boiler rooms and plant rooms to be calculated but not less than	5.0	6.7
n) Libraries	See Sl.No.(ii)	

(VI) MERCANTILE BUILDINS

a) Retail shops	4.0	3.6
b) Wholesale shops - to be calculated but not less than	6.0	4.5
c) Office Rooms	2.5+	2.7
d) Dining rooms, restaurants and cafeterias	3.0	2.7
e) Toilets	2.0	-
f) Kitchens and laundries	3.0	4.5
g) Boiler rooms and plant rooms to be calculated but not les than	5.0	6.7

h) Corridors, passages, staircases including fire escapes and lobbies	4.0	4.5
i) Corridors, passages, staircases subject to loads greater than from crowds such as wheeled vehicles, trolleys and the like	5.0	4.5
j) Balconies	Same as rooms which they give access but with a minimum of 4.0	1.5 per mtr run concentrated at the outer edge

(vii) INDUSTRIAL BUILDINGS

a) Work areas without machinery/ equipmnet ++	2.5	4.5
b) work areas with macinery/ equipment		
1) Light duty } To be cal-	5.0	4.5
2) Medium duty} culated but	7.0	4.5
3) Heavy duty } not less than	10.0	4.5
c) Boiler rooms and plant rooms to be calculated but not less than	5.0	6.7
d) Cafeteries and dining rooms	3.0+	2.7
e) Corridors, passages and staircases including fire escapes	4.0	4.5
f) Corridors, passages, staircases subject to machine loads, wheeled vehicles - to be calculated but not		
g) Kitchens	3.0	4.5
h) Toilets and bathrooms	2.0	

(VIII) STORAGE BUILDINGS $

a) Storage rooms (other than cold storage)warehouses- to be calculated based on the bulk density of materials stored but not less than	2.4 Kn/m² per each meter of storage height with a min. of 7.5 kN/m²	7.0
b) Cold storage- to be calculated but not less than height with a min.	5.0 kN/m² per each metre of storage of 15 kN/m²	9.0
c) Corridors, passages and staircases including fire escapes- as per the floor serviced but not less than	4.0	4.5
d) Corridors, passages subject to loads greater than from crowds such as wheeled vehicles, trolleys and the like	5.0	4.5
e) Boiler rooms and plant rooms	7.5	4.5

* Guide for requirements of low income housing.

\+ Where unrestricted assembly of persons is enticipated, the value of UDL should be increased to 4.0 kN/m²

\# With fixed seats implies that the removal of the seating and the use of the space for other purposes is improbable. The maximum likely load in this case is, therefore, closely controlled.

++ The loading in industrial buildings (workshops and factories) varies considerably and so three loadings under the terms "light", "medium" and "heavy" are introduced in order to allow for more economical designs but the terms have no special meaning in themselves othter than the imposed load for which the relevant floor is designed. It is, however, important particularly in the case of heavy weight loads, to assess the actual loads to ensure that they are not in excess of 10 kN/m2; in case where they are ine xcess, the design shall be based on the actual loadings.

$ For various mechanical handling equipment wich are used to transport goods, as in warehouses, workshops, store roomds, etc the actual load coming fromthe use of such equipment shall be ascertained and design should cater to such loads.

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REDUCTION IN IMPOSED LOADS ON FLOORS

For Floor supporting Structural Members : The following reductions in assumed total imposed loads on floors may be made in designing columns, load bearing walls, .piers, their supports and foundations.

Number of Floors (Including the Roof) to be Carried by Member under consideration	Reduction in Total Distributed Imposed Load on all Floors to be carried by the Member under consideration (Percent)
1	0
2	10
3	20
4	30
5 to 10	40
Over 10	50

Table 1.3.2

STAIRCASE LOADING OVER FLIGHT AND LANDING FOR DIFFERENT SPANS AND CLASS OF LOADING

a) Live load = 300 kg/m²

Span	Slab Thickness in cm.	Landing load in kg/m²	FLIGHT LOAD IN kg/m²			
			Tread= 30 cm, rise - 15 cm		Tread = 25cm, rise= 17.5cm	
			Brick	RCC	Brick Steps	Rcc Steps
2.50	10	625	830	868	890	934
3.00	12	675	886	924	951	995
3.50	13	700	914	952	981	1025
4.00	15	750	970	1008	1042	1085
4.50	17.5	813	1040	1078	1119	1163
5.00	20	875	1110	1148	1195	1239
5.50	22.5	938	1180	1218	1272	1316
6.00	25	1000	1249	1287	1347	1391

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b) Live load = 500 kg/m²

Span	Slab Thickness in cm.	Landing load in kg/m²	FLIGHT LOAD IN kg/m²			
			Tread= 30 cm, rise - 15 cm		Tread = 25cm, rise= 17.5cm	
			Brick	RCC	Brick Steps	Rcc Steps
2.50	10	825	1030	1068	1090	1134
3.00	12	875	1086	1124	1151	1195
3.50	13	900	1114	1152	1181	1225
4.00	15	950	1170	1028	1242	1286
4.50	17.5	1013	1240	1278	1319	1363
5.00	20	1075	1310	1348	1395	1439
5.50	22.5	1138	1380	1418	1472	1516
6.00	25	1200	1450	1488	1547	1591

Note : Weight of finish and hand rails has been taken as 90 Kg/m²

1.3.2 DESIGN OF BEAMS

1.3.2.1 SINGLY REINFORCED RECTANGULAR BEAMS

Four types of problems encountered in the design of beams and their analytical solutions:

<u>Type1:To design a beam to resist a given B.M:-</u>

Treating it as a balanced section, work out

a) constants k, j and Q for given allowable stresses

b) b, is governed by supporting column or aesthetic sense

b/d ratio is 1/2 to 1/4

$M = Q\,bd^2$, threrfore, d is known

c) calculate the amount of tensile steel required from $At = M/tjd$
d) provide necessary cover for tensile steel and arrive at D. (over all depth)

Type II : To determine permissible BM or loading on beams :-
(given, the section of beam and maximum allowable stresses)

a) Find out position of hte N,A from $n = \left(-mAt + \sqrt{(mAt^2) + 2bm\,Atd}\right)/b$

b) For the given stresses determine the critical neutral axis, nc

$$nc = \frac{1}{1 + t/cm} \times d = k\ d$$

Where k = critical neutral axis depth constant and is given by

$$\frac{1}{1 + t/cm}$$

c) i) If the depth of critical NA is more thanthe depth of actual N.A., the section is under reinforced and the stress in steel will reach maximum value.

$$MR = t.\ At\ (d\text{-}n/3)$$

i) If the depth of critical NA is less thanthe depth of the actual N.A, the section will be over - reinforced and the stress in concrete will reach the maximum reach the maximum value

$$MR = b\ n\ C/2\ (d\text{-}n/3)$$

d) Depending on the nature of loading and span, the permissible loading on the beam can also be worked out.

Type III To check the design of beam for stresses :
(given- the section, Area of steel, m to determine the stress developed in concrete and steel when the section subjected to given B.M)

a) Determine n vaue
b) Evaluate LA ($jd = a$) = d-n/3
c) M.R = ta. At. a = b.n ca (a/2) ,where the actual stresses in

Steel and concrete are 'a and ^c a respectively

Type IV:- To determine the area of tensile reinforcement :-
(given the section of beam, BM to be resisted and maximum allowable stresses)

a) For the given stresses determine k, j and Q

$$\text{from k or k'} = \frac{1}{1 + ta/mca} \quad \text{or } k = \frac{1}{1 + t/mc}$$

$n = kd$, $j = d\text{-}n/3$, $Q = 1/2\ c.k.j\ bd^2$

b) For the given dimensions find out the moment of resistance or critical section, $Mc = Qbd^2$
c) i) If the actual B M is less than the M R of critical section and the section is to be designed as an under- reinforced section

for $MR = t.\ At\ [d\text{-}n/3]$

Taking moments about NA
$bn\ n/2 = mAt\ [d\text{-}n/2]$

$At = bn^2/2m(d\text{-}n)$ or

$$MR = \frac{tbn^2}{2m\,(d-n)} \times (d - n/3)$$

ii) If the actual BM is more than the moment of resistance of critical section, the section has to be designed as an over reinforced section.

MR = bn C/2 (d-n/3) from which, n, can be found out and At can be found from

$$bn \cdot n/2 = m\,At\,(d-n)$$

Altenatively putting $M = Q' bd^2$, value of Q' can be easily evaluated. From the graph or tables, for given stresses % of steel (P) can be found and

$At = pbd / 100 \quad cm^2$

1.3.2.2. Doubly reinforced Rectangular beam

Three types of problems encountered with their analytical solutions :

Type I : Design of D.R. beams (given BMax, allowable stresses, m and restricted section of beams).

i) nc, J,k and Q can be found for given stresses for a singly reinforced beam.

ii) Assume f, i.e., cover for compressive steel as 4 cms , say

iii) Moment of resistance of singly reinforced beam $M1 = Qbd^2$ balance of Moment for which compressive steel is to be provided

$$M2 = M\text{-}M1 = M\text{-}Qbd^2$$

iv) Moment resisted by compressive steel =

$$(m\text{-}1)\,Ac.c\,n\text{-}f\,(d\text{-}f)/n = M\text{-}M1 = M2 \text{ from this, Ac can be found}$$

v) $At = At_1 + At_2 = \dfrac{M1}{tjd} + \dfrac{M\text{-}M1}{t\,(d\text{-}f)}$ or $At_2 = \dfrac{m\text{-}1}{m}Ac.\dfrac{n\text{-}f}{d\text{-}n}$

or by using graphs the section can be designed.

$Q' = M/bd^2$ Take some value of f = say 4 to 5 cms.

Choose a mix, M 15 or so, and from f determine pt for tensile steel and pc for compressive steel from graph

$At = \dfrac{Pt}{100} \times bd$ and $Ac = \dfrac{Pc}{100} \times bd, (cm^2)$

Type II :- To Determine the BM or permissible loading on beams :
(given size or section, area of steel and max.allowable stresses)

i) Determine the depth of actual NA by taking moments of concrete and equivalent steel areas about N.A.

ii) For given permissible stresses, determine the critical value nc.

iii) Compare depth of actual na to critical nc and determine whether concrete reaches maximum stress or steel (over - reinforced or under- reinforced) first.
ie., na > nc, or na < nc

a) If na > nc, concrete reaches maximum stress first, then
$MR = bn\dfrac{C}{2}\,(d\text{-}n/3) + (m\text{-}1)\,Ac.C\dfrac{n\text{-}f}{n}\,(d\text{-}f)$

b) If na < nc, steel reaches maximum stress first, corresponding to, maximum stress in steel, stress in concrete will be
$Ca = \dfrac{t}{m} + \dfrac{(n)}{d\text{-}n}$.

M and R can be found taking moments about tensile steel
$MR = b.n.\dfrac{Ca}{2}\,(d\text{-}n/3) + (m\text{-}1)\,Ac.Ca\,\dfrac{n\text{-}f}{n}\,(d\text{-}f)$

Type III :- To check design of a beam for stresses developed :-
(given size of beam, area of steel and B max.)

i) Equating equivalent areas of steel and concrete determine N.A
ii) If Ca is compressive stress in top fibre of concern,

stress in compressive steel = $mca\,\dfrac{[\,n\text{-}f\,]}{n}$

iii) Taking moments of compressive forces about tensile steel and equating to external BM

$M = (m-1)\,Ac \times \frac{n-f}{n}\,Ca\,(d-f) + bn.\frac{Ca}{2}\,(d-n/3)$

from which actual stress in top fibre of concrete Ca can be found.

iv) Stress in tensile steel, $ta = m\,Ca\,(d-n)/n$

v) Stress in compressive steel $m\,Ca \times (n-f)/n$

1.3.2.3 T AND L BEAMS

Three types of problems encountered and their analytical solutions :-

Type-I :- To check the section for stresses :-

i) Assume NA to lie in flange. Find the position of NA by equating moments of equivalent areas.

$$B.\,n^2/2 = mAt\;(d-n)$$

ii) Let Ca be actual stress in concrete

If N.A lies in flange, $MR = B.n.Ca\,(d-n/3)/2 = M$, the given moment

iii) If N.A does not lie in flange but lies in the web,

$B\,ds\;(n-ds/2) = m\,At\;(d-n)$ and

$$MR = B.ds\,\frac{Ca}{2}\left[(1+ds/n)\right]\left[\left(\frac{d-ds}{2}\right) + \frac{ds^2}{6(2n-ds)}\right]$$

From this, value Ca can be found.

iv) Stress in steel, $ta = m\,Ca.\,(d-n)/n$

Type -II :- To find MR of a given T-Beam (given, the section and allowable stresses) :-

i) By taking moments of equivalent areas about NA, find the position of NA from either, (as in I (ii) and (iii) above.

a) $Bn^2/2 = m\,At\,(d-n)$ or

b) $B.ds \times [n-ds]/2 = m.At\,(d-n)$

ii) From the permissible (allowable) stresses given, determine critical N A depth constant,

$K = 1/(1+t/c.m)$ from which depth of critical N.A = nc = kd= $[1/(1+t/Cm)] \times d$

iii) If the depth of actual NA is greater than n c, section is over-reinforced and concrete reaches its maximum stress first.

The compressive stress in top fibre of concrete = C (allowable)

a) If the NA lies in the flange:

$MR = Bn\,C/2\,[d - n/3]$ as for rectangular section

b) If NA lies in the web, lever arm is given by

$d - ds/2 + ds^2/6\,(2n-ds)$

$MR = B.ds\,.\,(c+c')/2\,.\,a$

$MR = B.\,ds\,.\,c/2\,.\,(1 + (n-ds)/n)\,[(d-ds/2) + ds^2/6\,(2n-ds)]$

iv) If the actual NA is above the critical NA, the section is under reinforced and steel reaches its maximum stress first. The corresponding maximum stress in concrete in the top fibre

$$Ca = t/m\,.\,[n/(d-n)]$$

a) If NA is found to lie in the flange

$MR = Bn\;\;Ca/2\,(d-n/3)$

b) If NA found to be in web:

$MR = B.ds.\;Ca/2\,(1+\frac{n-ds}{n})/n\,[\frac{(d-ds)}{2} + ds^2/6(2n-ds)]$

Type III : - Design of T-beam (given the thickness of slab and the Mix).

1)Assume breadth of rib such that it can accomodate the steel

2) Determine width of flange from the ISI code of practices

$bf = Lo/6 + bw + 6.Df$

3) Determine effective depth of beam for balanced section from graph or from

$$MR = B.C.ds \left[d - \frac{ds(1+k)}{2k} + \frac{ds^2}{3kd} \right]$$

when k = 0.4 for all mixes with t = 1400 kg/cm2

$$M/Bc = ds[(1\text{-}1.75\,ds + ds^2 / 1.2d)] \text{ or } Bds.C(2kd\text{-}ds)\,kd \times LA$$

This is on the assumptin that NA lies in rib and the compression taken by the concrete in the rib is neglected. M/BC graphs are available against effective depth d for various slab thicknesses ranging from 8 to 20 cm s. or LA lever arm can be safely assumed to lie between 0.88 to 0.95d. or (d-ds/2)

or $M = 0.45\, Bds. \frac{C(2kd\text{-}ds)}{kd}$, when k = 0.4

M = 1.125 Bds. C(0.8d-ds) - 1.125 Bds C (2n-ds)

For Tor steel apply value of k = 0.289 (approximate) for M15 mix
Alternatively, knowing flange width, moment to be resisted, Mix used, vaue of M/BC, vlaue of d can be found for a given slab thickness of ds, from the graph.

Also determine the effective depth so that shear stress is within the permissible limits for assumed effective depth. Maximum depth of ebeam from the above, will be the effective depth of the beam, cover to it, to get D i.e. Total Depth.

4) Calculate steel area from M/t a where a is LA (Lever arm)
Can be assumed as d-ds/2 and provide steel reinforcement

5) Check the section for stresses in
(a) Steel
(b) Concrete
(c) Shear
(d) development length at supports and critical sections and provide shear reinforcement as per code through bent up bevs and vertical stirrups.

TABLE: 1.3.3 MINIMUM DEPTH-OF SLABS FOR DEFLECTION

Span (m)	Depth of Slab (mm) simply supported	Continuous	Remarks
2.00	75	75	
2.50	90	75	
2.75	100	90	
3.00	100	90	Depth/ Span = 1/30 for Simply supported
3.25	110	100	
3.50	120	100	
3.75	130	110	
4.00	140	110	1/35 for continuous
4.25	140	120	
4.50	150	130	

TABLE 1.3.4 RECTANGULAR BEAMS WITH ONLY TENSION REINFORCEMENT

Depth (cm)	Width (cm.) 23 M	23 As	30 M	30 As	REMARKS
20	41	1.2	54	1.6	Concrete M15
25	70	1.6	92	2.1	Steel HYSD Bars
30	106	2.0	138	2.5	M = Max moment capacity at working load
35	150	2.3	196	3.0	As- Corresponding Aera of Steel mm^2
40	200	2.7	261	3.5	Notes :
45	260	3.1	339	4.0	1. Clear cover to main reinf. = 2.5 cm
50	326	3.4	426	4.5	2. For design moments less than the
60	481	4.1	628	5.4	tabulated value, the area of steel can be
70	666	4.9	869	6.3	reduced in direct proportion.

TABLE 1.3.5 ONE WAY SLABS (working stress design)

STEEL = HYSD BARS

Overall Depth mm	Main Dia.mm	Steel spacing mm	Distribution Dia.mm	Steel Spacing mm	Resisting moment Kgm	REMARKS
75	8	165	6	250	200	
100	8	200	6	190	420	
110	8	180	6	170	530	Stress
	10	270	6			σcb= 50kg/cm²
120	8	160	6	160	650	σst= 2300Kg/cm² m = 18
	10	250	6			
130	10	230	6	140	790	Constnats
	12	320				'k" = 0.29
140	10	210	6	130	940	d = 0.90
	12	300				Q = 6.55
150	10	190	6	120	1100	M.R = 6.55 bd² Cover = 15 mm

NOTE: For two way slabs, calculate moments in both directions and read the main steel from the Table for moment in each direction and provide.

TABLE 1.3.6 T-BEAMS WITH TENSION REINFORCEMENT

M = Maximum moment capacity of T-beam in 1 m width of flange(Tonnes)
As= Corresponding area of steel (Sq mm)

Overall depth (cm)		Flange thickness (cm)					
20	M	177	177	177	177	177	177
	As	5.2	5.2	5.2	5.2	5.2	5.2
25	M	301	301	301	301	301	301
	As	6.8	6.8	6.8	6.8	6.8	6.8

Overall depth cm		Flange thickness (cm)					
30	M	458	458	458	458	458	458
	As	8.4	8.4	8.4	8.4	8.4	8.4
35	M	647	647	647	647	647	647
	As	9.9	9.9	9.9	9.9	9.9	9.9
40	M	869	869	869	869	869	869
	AS	11.5	11.5	11.5	11.5	11.5	11.5
45	M	1100	1121	1124	1124	1124	1124
	As	12.7	13.0	13.1	13.1	13.1	13.1
50	M	1337	1376	1402	1410	1410	1410
	As	13.7	14.2	14.5	14.7	14.7	14.7
60	M	1815	1897	1964	2017	2054	2078
	As	15.1	15.9	16.5	17.4	17.4	17.7
70	M	2297	2427	2538	2632	2712	2777
	As	16.1	17.1	18.0	18.7	19.4	19.9

TABLE 1.3.7 Shear Carrying capacity for 2 legged vertical stirupps (Connes) (WORKING STRESS METHOD)

Overall beam depth (cm)	6 mm dia (Mild steel) 6	8	10	15	20	25	30	8mm dia (HYSD) bars 6	8	10	15	20	25	30
20	2.1	1.6						6.3	4.7					
25	2.8	2.1	1.7					8.2	6.2	4.9				
30	3.5	2.6	2.1	1.4				10.4	7.6	6.1	4.1			
35	4.1	3.1	2.5	1.6				12.1	9.1	7.2	4.8			
40	4.8	3.6	2.9	1.9	1.4			14.0	10.5	8.4	5.6	4.2		
45	5.4	4.1	3.3	2.2	1.6			15.9	11.9	9.5	6.4	4.8		
50	6.1	4.6	3.6	2.4	1.8	1.5		17.8	13.4	10.7	7.1	5.3	4.3	
60	7.4	5.5	4.4	3.0	2.2	1.8	1.5	21.7	16.2	13.0	8.7	6.5	5.2	4.3
70	8.7	6.5	5.2	3.5	2.6	2.1	1.7	25.5	19.1	15.3	10.2	7.6	6.1	5.1

NOTE : Maximum spacing is limited to 0.5 x depth of beam

TABLE 1.3.8 RECTANGULAR COLUMNS Working Stress Method

Concrete = M - 15 ; Steel = HYSD BARS;

Size mm x mm	Gross Area cm^2	Load Concrete (T)	Load on Bars (T) Min : 0.08Sc Ac Min	Max : 0.04 Sc Ac Max	Total capacity (Tonnes) Min	Max.	Minimum Recom - -mended ties
230 x230	530	21	8	40	29	61	
x 300	690	28	10	52	38	80	
350	800	32	12	60	44	92	
400	920	37	14	70	51	107	6 mm φ
450	1030	41	15	78	56	119	@150c/c
300 x300	900	36	13	68	49	104	
350	1050	42	16	80	58	102	
400	1200	48	18	91	66	139	8 mm φ
450	1350	54	20	102	74	150	@ 200 c/c

TABLE - 1.3.9

Working Stress Design
Steel : HYSD BARS

LOAD CARRYING CAPACITY OF LONGITUDINAL STEEL IN COLUMNS (TONNES)

Bar Dia mm.	Area Cm^2	Load on each Bar	No. of Bars 4	6	8	10	12	14
12	1.13	2.1	8.6	13	17	21	26	30
16	2.01	3.8	15	23	31	38	46	53
20	3.14	5.9	24	36	48	60	71	83
22	3.80	7.2	29	43	58	72	87	101
25	4.91	9.3	37	56	75	93	112	131

TABLE 1.3.10

SQUARE FOOTING

Safe bearing capacity of soils = 5 t/m²

Footing Width (A) (m)	Total load (Tonnes)	a cm	D cm	As cm^2
1.0	4.5	23	25	0.80
1.2	6.5	23	25	1.50
1.4	8.9	23	25	2.60
1.6	11.6	23	30	3.3
1.8	14.7	23	35	3.9
2.0	18.1	23	40	4.9
2.2	22.0	23	45	5.8
2.4	26.2	23	50	6.8
2.6	30.7	23	60	7.2

2.8	35.6	23	65	8.4
3.0	40.9	23	75	9.0
3.2	46.6	23	80	10.30
3.4	52.60	23	90	11.00
3.6	58.9	23	95	12.40
3.8	65.60	23	105	13.20
4.0	72.70	23	115	14.10
4.2	80.20	23	120	15.70
4.4	88.00	23	130	16.70
4.6	96.20	23	140	17.80
4.8	104.70	23	150	18.90
5.0	113.60	23	160	20.10

The steel (As) is to be provided in each direction.

TABLE 1.3.11

SQUARE FOOTING

Safe bearing capacity of soils = 10 t/m²

Footing Width (A) (m)	Total load (Tonnes)	a cm	D cm	Areas of Steel As (cm²)
1.0	9.10	23	25	2.10
1.2	13.10	23	25	2.90
1.4	17.81	23	30	3.50
1.6	23.30	23	40	4.73
1.8	29.45	23	45	5.75
2.0	36.35	23	55	6.84
2.2	44.00	23	60	7.98
2.4	52.36	23	70	8.18
2.6	61.45	23	80	10.45
2.8	71.30	23	90	11.75
3.0	81.80	23	100	13.10
3.2	93.10	23	110	14.52
3.4	105.10	23	120	15.97
3.6	117.80	23	130	17.47
3.8	131.30	23	145	19.02
4.0	145.45	23	155	20.61
4.2	160.35	23	165	22.23

TABLE 1.3.12

SQUARE FOOTING

Safe bearing capacity of soil = 15 t/m²

Footing Width (A) (m)	Total load (Tonnes)	a cm	D cm	Areas of Steel As (cm²)
1.0	13.70	23	25	2.69
1.2	19.30	23	30	3.55
1.4	26.70	23	35	3.91
1.6	34.90	23	45	5.80
1.8	44.20	23	55	7.05
2.0	54.50	23	65	8.37
2.2	66.00	23	75	9.77
2.4	78.55	23	85	11.21
2.6	92.20	23	95	12.79
2.8	106.90	23	110	14.39
3.0	122.72	23	120	16.06
3.2	139.60	23	135	17.78
3.4	157.60	23	150	19.56

TABLE 1.3.13

SQUARE FOOTING

Safe bearing capacity of soil = 20 t/m²

Footing Width (A) (m)	Total load (Tonnes)	a cm	D cm	Areas of Steel As (cm²)
1.0	18.20	23	25	2.97
1.2	26.20	23	35	4.10
1.4	35.63	23	45	5.35
1.6	46.54	23	55	6.69
1.8	58.90	23	65	8.14
2.0	72.70	23	75	9.67
2.2	88.00	23	85	11.29
2.4	104.70	23	100	12.99
2.6	122.90	23	110	14.77
2.8	142.50	23	125	16.62
30.0	163.60	23	140	18.54

1.3.2 DESIGN OF STAIR CASES

1.3.2.1 Desired width of stair cases and balconies

a) Residential buildings : minimum 85 cms (75 - 95 cm)

	No.of users	width of stair in cms
	10	
	11-20	110
	21-100	125
		140
b) Public buildings ware houses	upto 200 -	
Industrial buildings	200 - 300 -	155
public houses		185

Note : Increase by 25 mm for every additional 15 persons until a maximum of 2.75 metres is reached.

c) Tread (width of step)

Residences	Non-residential buildings ; factories / cinema theatres	
23 cms	27 - 30 cms	25 - 27 cms

d) Rise (vertical distance between two steps)

Residences	Non-residential buildings ; factories / cinema theatres	
15 - 18 (20) cms	12 - 15 cms	19 - 25 cms

Thumb rule : - 2 R+T = 60 cms, Tr = 400 --450 (500)

Tread	23	25	28	30	33	35	38
Rise	19	17	16	15	14	12	11

Total number of steps in a single flight	$\ngtr$ 12
Minimum clear head room over stairs	2.10
Maximum heights of flight without landings or winders	2.4 m

e) Winders : Provided if unavoidable. Tread should be at least 23 cms, at about 45 cms from the (inner small end of the tread) handrail, generally located in the lower reaches but not on the top of the stairs. For quarter space landing, not more than three winders shall be used.

1.3.2.2 Design aspects

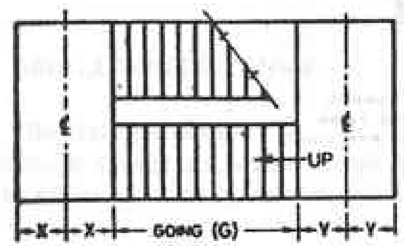
Stairs spanning horizontally :- Stairs supported at each side by walls, stringer beamsor at one side by wall and the other side by a beam.

1.3.2.3 Effective span of Stairs :- The effective span of stairs without stringer beams shall be taken as the following horizontal distances :

a) Where supported at top and bottom risers by beams spanning parallel with the risers, the distance centre-to-centre of beams,

b) Where spanning on to the edge of landing slab, which spans parallel with the risers (see Fig.), a distance equal to the going of the stairs plus at each end either half the width of the landing or one metre, whichever is smaller, and

c) Where the landing slab spans in the same direction as the stairs, they shall be considered as acting together to form a single slab and the span determined as the distance centre to centre of the supporting beams or walls, the going measured horizontally.



X	Y	SPAN IN METRES
<1 m	<1 m	G + X + Y
<1 m	≥ 1 m	G + X + 1
≥ 1 m	<1 m	G + Y + 1
≥ 1 m	≥ 1 m	G + 1 + 1

Fig. EFFECTIVE SPAN FOR STAIRS SUPPORTED AT EACH END BY LANDINGS SPANNING PARALLEL WITH THE RISERS.

EFFECTIVE SPAN FOR STAIRS SUPPORTED AT EACH END BY LANDINGS SPANNING PARALLEL WITH THE RISERS

1.3.2.4 Distribution of laoding on Stairs :- In the case of stairs with open wells where spans partly crossing at right angles occur, the laod on areas common to any two such spans may be taken as one-half in each direction as shown in fig. Where flights or landings are embedded into walls for a length of not less than 110 mm and are designed to span in the direction of the flight, a 150 mm strip may be deducted from the loaded area and the effective breadth of the section increased by 75 mm for purposes of design (see fig.)

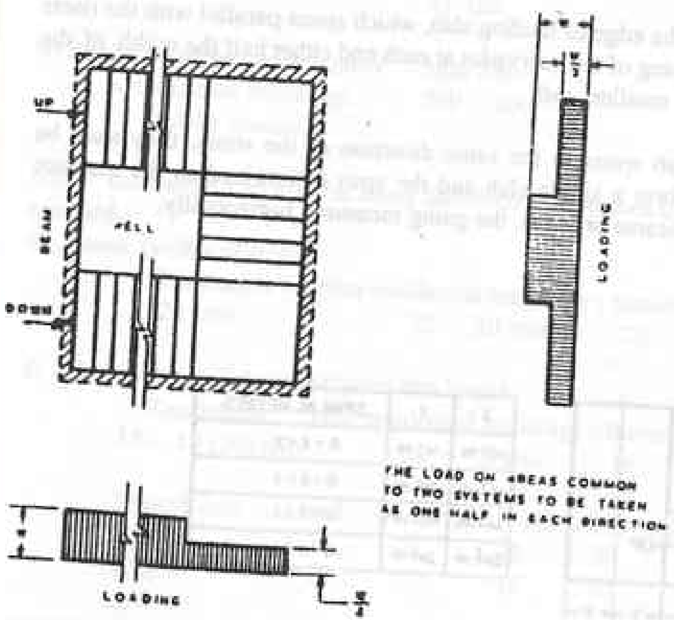


Fig. LOADING ON STAIRS WITH OPEN WELLS

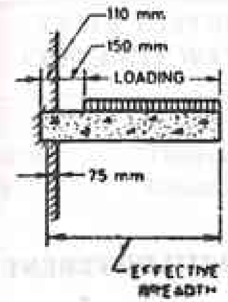


Fig. LOADING ON STAIRS BUILT INTO WALLS

Stairs spanning longitudinally : - Staire span between supports and the ,bottom and top of flight and remain unsupported at sides.

Bending Moment : Coefficients $WL^2 / 8$ if simply supported and $WL^2 / 10$ if continuous
Loading : See Table 1.3.1 I.S 875 - 1964 (Revised)
300 kg/m² for residences
00 Kg/m² for offices and balconies
Mix M 15

To arrive at teh corresponding load on plan, for waiste slab multiply L by a factor :

$$= \frac{(R^2 + T^2)}{T}$$

1.3.3.0 SMALL WATER TANKS

1.3.3.1 Illustrative Example

Let us consider a water tank with a storage capacity of 7,000 liters. From Table1.3.14 and 1.3.15 the following particulars may be set out.

Capacity of tank	......	7,000 Litres
Inside diameter	......	2.5 m
Depth (including free board)	......	1.6 m
Wall thickness	......	10 cm.
Floor thickness	......	10 cm,
(Reinforcement in wall, hoop bars conforming to bars c in fig1.3.1)	......	10 mm dia at 20 cm c/c

Vertical bars (a and b fig.3.11)	......	10 mm dia at 20 cm c/c
Reinforcement in floor (bars d in Fig.3.1)	......	10 mm dia at 20 cm c/c mesh at top and bottom

TABLE 1.3.14 RECOMMENDED SIZES OF CIRCULAR TANKS WITH DIFFERENT CAPACITIES

Capacity of tank Litres	Internal diameter m	Depth including free board of 15 cm m
2,500	1.75	1.20
5,000	2.25	1.40
7,000	2.50	1.60
10,000	2.75	1.85
20,000	3.50	2.25
30,000	4.00	2.55
40,000	4.50	2.65
50,000	5.00	2.70

TABLE 1.3.15 STRUCTURAL DETAILS OF CIRCULAR REINFORCED CONCRETE WATER TANKS OF DIFFERENT CAPACITIES (Figure 1.3-1)

(On ground / level platform)

Capacity litres	Thickness of concrete in Wall	Floor	Reinforcement in wall Hoop bars*	Vertical bars**	Mesh reinforced cement in floor at top and bottom ++
2,500	10	10	10 mm dia @20 cm c/c	10 mm dia @20 cm c/c	10 mm dia @20 cm c/c
5000	10	10	10 mm dia @20 cm c/c	10 mm dia @20 cm c/c	10 mm dia @20 cm c/c
7,000	10	10	10 mm dia @20 cm c/c	10 mm dia @20 cm c/c	10 mm dia @20 cm c/c
10,000	13	13	10 mm dia @20 cm c/c	10 mm dia @20 cm c/c	10 mm dia @20 cm c/c
20,000	15	15	10 mm dia @18 cm c/c	10 mm dia @18 cm c/c	12 mm dia @18 cm c/c
30,000	15	15	10 mm dia @18 cm c/c	10 mm dia @18 cm c/c	12 mm dia @18 cm c/c
40,000	18	18	12 mm dia @20 cm c/c	12 mm dia @20 cm c/c	12 mm dia @20 cm c/c
50,000	18	18	12 mm dia @19 cm c/c	12 mm dia @19 cm c/c	12 mm dia @19 cm c/c

* Shape conforming to bars c
** Shape conforming to bars a and b
++ Shape conforming to bars d

Grade of concrete = M 20

NOTE :
1. This table applies only to those tanks, the sizes of which conform to those given in table 3-16 for the respective capacities.
2. Only hydrostatic pressure is considered in the design.
3. Wind pressure resistance has to be separately designed.

TABLE 1.3.16

RECOMMENDED SIZES OF SQUARE TANKS WITH DIFFERENT CAPACITIES

Capacity of 15cm litres	Internal dimensions m			Depth including free board m
2,500	1.5	x	1.5	1.25
5,000	2.0	x	2.0	1.40
7,000	2.3	x	2.3	1.50
10,000	2.6	x	2.6	1.65
20,000	3.3	x	3.3	2.00
30,000	3.8	x	3.8	2.25
40,000	4.2	x	4.2	2.45
50,000	4.5	x	4.5	2.65

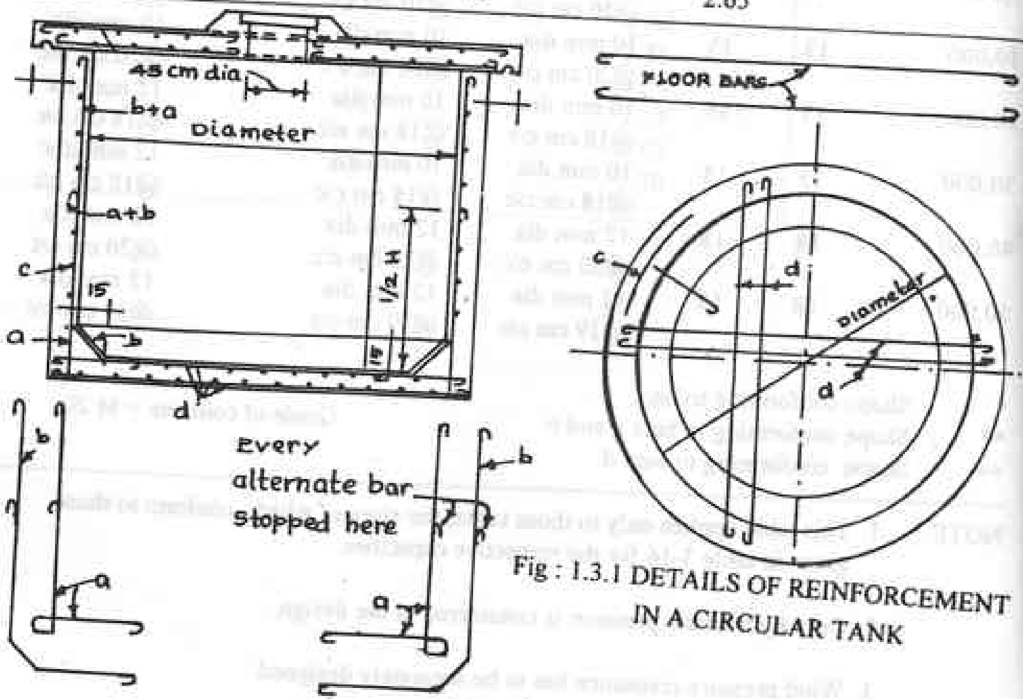


Fig : 1.3.1 DETAILS OF REINFORCEMENT IN A CIRCULAR TANK

TABLE 1.3.17 STRUCTURAL DETAILS OF SQUARE REINFORCED CONCRETE WATER TANKS OF DIFFERENT CAPACITIES (FIG 1.3-2)

Capacity	Thickness		Reinforcement in wall					Mesh reinforcement in floor, alternate bars bent up ends
	Wall,	floor,	Vertical bars in bottom half *	Vertical bars in bottom half **	Vertical bars+	Vertical bars in Tophalf++	Horizontal bars	
litres	cm	cm						
2500	10	10	10 mm dia @20 cm oc	10 mm dia @20 cm oc	10 mm dia @ 20 cm oc	—	10 mm dia @ 20 cm oc	10 mm dia @20 cm oc
5000	10	10	10 mm dia @20 cm oc	10 mm dia @20 cm oc	10 mm dia @20 cm oc	—	10 mm dia @ 20 cm oc	10 mm dia @20 cm oc
7000	10	10	12 mm dia @20 cm oc	10 mm dia @20 cm oc	10 mm dia @20 cm oc	—	10 mm dia @20 cm oc	10 mm dia @20 cmoc
10000	13	13	16 mm dia @20 cm oc	12 mm dia @20 cm oc	12 mm dia @ 40 cm oc	12 mm dia @40 cm oc	10 mm dia @20 cm oc	10 mm dia @20 cm oc
20000	15	15	16 mm dia @20 cm oc	12 mmdia @20 cm oc	12 mm dia @ 40 cm oc	12 mm dia @40 cm oc	12 mm dia @20 cm oc	12 mm dia @20 cm oc
30000	18	18	16 mm dia @20 cm oc	12 mm dia @20 cm oc	12 mm dia @40 cm oc	12 mm dia @40 cm oc	12 mm dia @20 cm oc	10 mm dia @20 cm oc
40000	18	18	20 mm dia @20 cm oc	12 mm dia @20 cm oc	12 mm dia @ 40 cm oc	12 mm dia @40cm oc	12mm dia @20cm oc	10mm dia @20cmoc
50000	20	20	22 mm dia @20 cm oc	12 mmdia @20 cm oc	12 mm dia @ 40 cm oc	12mm dia @40cm oc	12 mm dia @20 cm oc	10mm dia @20cmoc

* Shape conforming to bars a   ** Shape conforming to bars b

\+ Shape conforming to bars c   ++ Shape conforming to bars d

Grade of concrete M20

NOTE : This table applies only to those tanks, the size of which conform to those given in Table 1-3-16 for the respective capacities.

1.3.3.2 Uses of Accoproof

To make the concrete used in building water storage tanks water tight, it is necessary to mix 2 % of Accoproof integrally with the concrete, which should consist of 1 part of cement, 1½ parts of clean graded sand 4.75 mm down, and 3 parts of broken stone graded from 20 to 4.75 mm.

For rendering newly built concrete storage tanks water tight, their walls and floors should, depending on their unevenness, be treated with two or three caots, each of cement , mortar comprising 1 part of cement admixed with 2 % of Accoproof and 3 parts of clean graded sand from 3mm down.

Where repairs for stopping leakage have to be carried out to an existing concrete masonry tank, the base of finishing coats of plaster, each 12.5 mm thick, should be made water proof by admixing 2 % of Accoproof with the cement mortar 1:2 1/2 mix.

Number of Coats : If the surface of newly built wall to be renderd is fairly even, two coats should be sufficient - the backing coat being 10 to 12.5 mm thick and the finishing coat required. If the surface of the wall is rough and uneven, three coats should be used, the first coat about 10 mm thick being applied to true up the surface.

TABLE 1.3.18 ESTIMATED QUANTITIES OF ACCOPROOF REQUIRED FOR 100 M² OF CEMENT MORTAR FOR VARIOUS THICKNESS

Accoproof assumed at 2 % by weight of Cement

Mix.	Acco proof in Kg for 100 M² of cement mortar for thickness in mm of					
	10	12.5	15	20	25	30
1 : 2½	11.8	14.7	17.8	23.6	29.4	35.4
1 : 3	10.1	12.6	15.2	20.2	25.2	30.3

ROOF

It is desirable to provide a roof over concrete tanks to prevent pollution of water. By referring to Table 1.3.19 and 1.3.20 the thickness of the roof slab and reinforcement required for circular and square water tanks, respectively, for capacities varying from 2,500 to 50,000 litres can be obtained.

TABLE 1.3.19 STRUCTURAL DETAILS OF REINFORCED CONCRETE ROOFS FOR CIRCULAR WATER TANKS

Grade : M - 15

Capacity litres	Internal diameter of tank, m	Thickness of roof cm	Mesh reinforcement in roof (alternate bars bent up at ends)	
2,500	1.75	10	10 mm dia	@ 20 cm c/c
5,000	2.25	10	10 mm dia	@ 20 cm c/c
7,000	2.55	10	10 mm dia	@ 20 cm c/c
10,000	2.75	10	10 mm dia	@ 20 cm c/c
20,000	3.50	10	10 mm dia	@ 20 cm c/c
30,000	4.00	10	10 mm dia	@ 15.5 cm c/c
40,000	4.50	10	10 mm dia	@ 15.5 cm c/c
50,000	5.00	10	10 mm dia	@ 12.5 cm c/c

Note : Design live load assumed as 150 kg / m²

TABLE1.3.20 STRUCTURAL DETAILS OF REINFORCED CONCRETE ROOFS FOR SQUARE WATER TANKS

Capacity litres	Internal size of tank, m	Thickness of roof cm	Mesh reinforcement in roof (alternate bars bent up at ends	
2,500	1.5 x 1.5	10	10 mm dia	@ 20 cm c/c
5,000	2.0 x 2.0	10	10 mm dia	@ 20 cm c/c
7,000	2.3 x 2.3	10	10 mm dia	@ 20 cm c/c
10,000	2.6 x 2.6	10	10 mm dia	@ 20 cm c/c
20,000	3.3 x 3.3	10	10 mm dia	@ 20 cm c/c
30,000	3.8 x 3.8	10	10 mm dia	@ 17.5 cm c/c
40,000	4.2 x 4.2	10	10 mm dia	@ 16.5 cm c/c
50,000	4.5 x 4.5	10	10 mm dia	@ 14.0 cm c/c

Note : Design live load assumed as 150 kg/ m². Grade of concrete : M 15

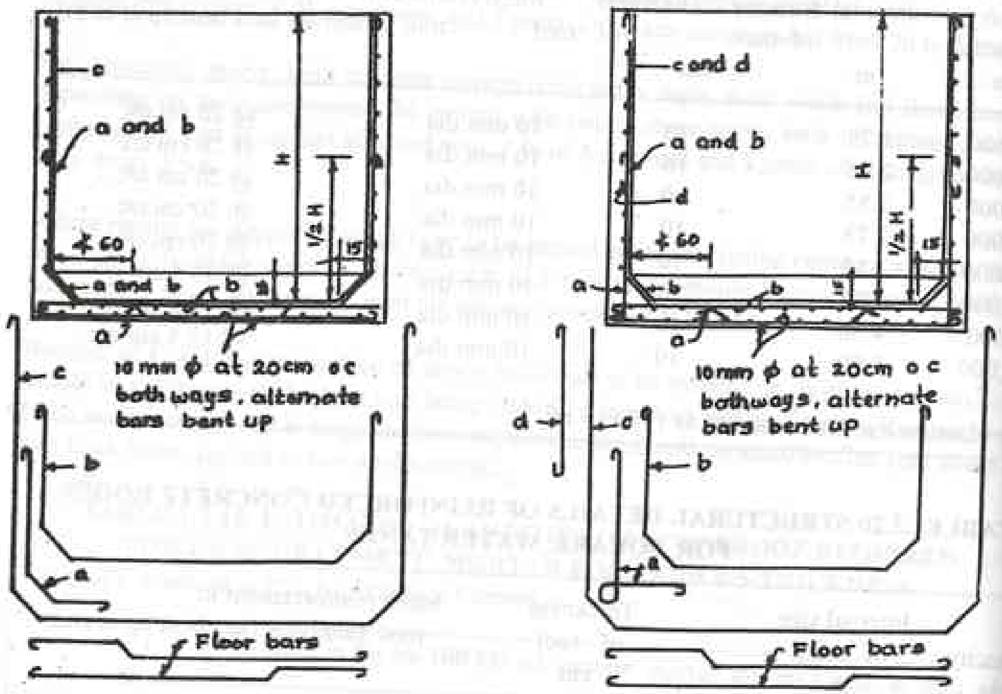


FIG 1.3-2

SECTION OF SQUARE TANKS SHOWING DETAILS OF REINFORCEMENT

1.3.4 DESIGN AIDS FOR WORKING STRESS METHOD :

1.3.4.1 Flexural Members :

(1) Singly reinforced rectangular members :-

Modular ratio

$$m = \frac{2800}{3\sigma cbc} = \frac{933.33}{\sigma\ cbc}$$

Therefore, for all values of m, σ cbc = 933.33

Balanced section : σ st : m σ cbc(1/k - 1)

$$(1/k - 1) = \frac{\sigma\ st}{m\ \sigma\ cbc} = \frac{\sigma\ st}{933.33}$$

$$1/k = \frac{\sigma\ st}{933.33} + 1 = \frac{\sigma\ st + 933.33}{933.33}$$

$$k = 933.33\ /\ (\ \sigma\ st + 933.33\ )$$

Note :- Value of K for balanced section depends on σst only and it is independent of σ cbc

MR for balanced section. M bal = (bd² / 2) . σ cbc k (1 - k/3)

The values of Mbal / bd² for different values of σ cbc and σ st are given below.

Concrete grade	σ cbc kg/cm²	σ st in kg/cm² MS 4500	Fe 415- Tor40 2300	Fe.500-Tor50 2750
M15	50	8.7	6.50	5.80
M20	70	12.10	9.1	8.10
M25	85	14.70	11.10	9.90
M30	100	17.30	13.0	11.60

Percentage of steel (pt) for balanced section :-

Compressive force C : Tensile force = T

$$\sigma cbc \frac{kbd}{2} = \frac{Pt.bal.}{100} \times bd.\ \sigma st$$

$$Pt\ bal = \frac{50\ K\ \ \sigma\ cbc}{\sigma\ st}$$

Pt.of steel (tensile reinforcement) for a balanced Singly reinforced section

Concrete grade	σ cbc kg/cm²	σ st in Kg/cm² M.S. 1400	Tor40=Fe415	Tor50=Fe 500
M15	50	0.71%	0.31	0.23
M20	70	1.00%	0.44	0.32
M25	85	1.21%	0.53	0.39
M30	100	1.43%	0.63	0.46

Under reinforced section : Singly reinforced section

Equating moments :- N A =

$$b.kd.\frac{kd}{2} = \frac{pt}{100}\ bd\ \ m\ (d\text{-}kd)\ \text{or}\ \frac{bn^2}{2} = Ast.m(d\text{-}n)$$

$$bd^2\frac{k^2}{2} = bd^2\frac{ptm}{100}(1\text{-}k)$$

$$k^2 = \frac{ptm}{50}(1\text{-}k)$$

$$k^2 + \frac{pt\ mk}{50} - \frac{ptm}{50} = 0$$

$$k = \frac{-ptm}{100} + \sqrt{\frac{pt\ m^2}{(100)^2} + \frac{ptm}{50}}$$

MR for under reinforced section :

M : bd^2 pt. σst (1-k/3) / 100 or Ast. σst (d-n/3)

Table 1.3.2 1 M15 Mix - MR factors

Working stress Method

Flexure - Moment of resistance factor for a singly reinforced Reinforced Rectangular section with different % of steel for different grades of steel and concrete (unbalancedSection)

σ Cbc = 50Kg/cm², grade = M=15

Moment of Resistance factor = M/bd²(in kg/cm²)

pt % of steel (100 Ast)/bd	σst kg/cm² Ms/1400	Tor . 40 Fe 415 2300	Tor . 50 Fe 500 2750
0.12	1.57	2.58	3.09
0.13	1.70	2.79	3.34
0.14	1.83	3.00	3.59
0.15	1.95	3.21	3.84
0.16	2.08	3.41	4.08
0.17	2.20	3.62	4.33
0.18	2.30	3.83	4.57
0.19	2.45	4.03	4.82
0.20	2.58	4.23	5.06
0.21	2.70	4.44	5.31
0.22	2.82	4.64	5.55
0.23	2.95	4.84	5.79
0.24	3.07	5.05	
0.25	3.19	5.25	
0.26	3.32	5.45	

σst in Kg/cm² Pt.	M.S	T40	T50
0.47	5.83		
0.48	5.95		
0.49	6.07		
0.50	6.19		
0.51	6.30		
0.52	6.42		
0.53	6.54		
0.54	6.65		
0.55	6.77		
0.56	6.89		
0.57	7.00		
0.58	7.12		
0.59	7.24		
0.60	7.35		
0.61	7.47		

pt % of steel (100 Ast)/bd	σst kg/cm² Ms/1400	Tor . 40 Fe 415 2300	Tor . 50 Fe 500 2750	σst in Kg/cm² Pt.	M.S	T40	T50
0.27	3.44	5.65		0.62	7.58		
0.28	3.56	5.85		0.63	7.70		
0.29	3.68	6.05		0.64	7.81		
0.30	3.80	6.25		0.65	7.93		
0.31	3.92	6.45		0.66	8.04		
0.32	4.05			0.67	8.16		
0.33	4.17			0.68	8.27		
0.34	4.29			0.69	8.39		
0.35	4.41			0.70	8.50		
0.36	4.53			0.71	8.62		
0.37	4.65			0.72			
0.38	4.77			0.73			
0.39	4.89			0.74			
0.40	5.00			0.75			
0.41	5.12			0.76			
0.42	5.24			0.77			
0.43	5.36			0.78			
0.44	5.48			0.79			
0.45	5.60			0.80			
0.46	5.72						

Table 1.3.22 M.20 - MR Factor Working Stress Method

Flexure: Moment of Resistance factor M/bd²/ in kg/cm² for singly reinforced Rectangular section using different % steel for different grades of steel and concrete (Unbalanced section)

pt % of steel (100 Ast)/bd	σst kg/cm² Ms/1400	T 40 2300	T 50 2750	σst in Kg/cm² Pt	M.S.	T 40	T 50
0.20	2.81	4.28	5.12	0.71	8.78		
0.22	2.86	4.70	5.62	0.72	8.90		
0.24	3.11	5.11	6.11	0.73	9.01		
0.26	3.36	5.52	6.60	0.74	9.13		
0.28	3.61	5.93	7.09	0.75	9.25		
0.30	3.86	6.33	7.57	0.76	9.36		
0.32	4.10	6.74	8.08	0.77	9.48		
0.34	4.35	7.14		0.78	9.60		
0.36	4.59	7.55		0.79	9.71		
0.38	4.84	7.95		0.80	9.83		
0.40	5.08	8.35		0.81	9.94		
0.42	5.32	8.75		0.82	10.06		
0.43	5.45	8.95		0.83	10.18		
0.44	5.57			0.84	10.29		
0.45	5.69			0.85	10.41		
0.46	5.81			0.86	10.52		
0.47	5.93			0.87	10.64		
0.48	6.05			0.88	10.75		
0.49	6.17			0.89	10.87		
0.50	6.29			0.90	10.99		
0.51	6.41			0.91	11.10		
0.52	6.53			0.92	11.22		
0.53	6.65			0.93	11.33		

pt % of steel (100 Ast)/bd	Ms/1400	σst kg/cm² T 40 2300	T 50 2750
0.54	6.77		
0.55	6.89		
0.56	7.01		
0.57	7.13		
0.58	7.24		
0.59	7.36		
0.60	7.48		
0.61	7.60		
0.62	7.72		
0.63	7.84		
0.64	7.95		
0.65	8.07		
0.66	8.19		
0.67	8.31		
0.68	8.43		
0.69	8.54		
0.70	8.66		

Pt	σst in Kg/cm² M.S.	T 40	T 50
0.94	11.45		
0.95	11.56		
0.96	11.68		
0.97	11.79		
0.98	11.90		
0.99	12.02		
1.00			
1.01			
1.02			
1.03			
1.04			
1.05			
1.06			
1.07			
1.08			
1.09			
1.10			
1.11			
1.12			
1.13			

1.3.4.2 Flexrual Members - Doubly reinforced rectangular section

Working design

When the Moment to be resisted exceeds the balancing moment, these section are used upto certain extent. Afterwards, they will be replaced by T & L Beams or the sections will be redesigend changing the parameters if permissible.

M total = M for balanced section + M' to be taken by compressive steel and additional tensile reinforcement over the steel used in balanced section.

The Stress in compression reinforcement is taken as 1.5 m times the actual stress in the surrounding concrete.

$$\text{Additional R.M } M' = \frac{Pc\ bd}{100}(1.5m-1)\,\sigma cbc \times \frac{(kd-d')}{kd}(d-d')$$

$$= \frac{Pc}{100}(1.5m-1)\,\sigma cbc \times \left(1 - \frac{d'}{kd} - \frac{d'}{d}\ b'd^2\right)$$

(by taking moments about centroid of tensile steel)

Equating Addl. tensile force and addl. compressive force (by virtue of addl. steel provided in the section)

bd (Total pt)

$$\frac{(Pt,\ bal\ )}{-100}\sigma st = \frac{Pc.\ bd}{100}(1.5m-1)\ \sigma cbc\ \frac{(1-d')}{kd}$$

or $(pt - pt,\ bal)\sigma st = pc\,(1.5m-1)\,\sigma cbc\,(1-d'/kd)$

$$\text{Total R.M} = M\ bal + \frac{(pt-pt,\ bal)}{100}\ \sigma st\,(1-d'/d)\,bd^2$$

Where $M\ bal = k\ bd^2\ \sigma cbc\ (1-k/3)$

$$\text{or } pt.bal\ .\ \frac{bd^2}{100}\ .\ \sigma st\ (1-k/3)$$

$$A_{st} = (Ast1 + Ast2) = Pt,\ bal \times \frac{bd}{100} + \frac{M'}{\sigma st\,(d-d')}$$

To simplify matters, the compression reinforcement, Asc can also be expressed as a ratio of the additional tensile steel Ast_2 as

$$\frac{Asc}{Ast2} = \frac{Pt}{(Pt-Pt,bal)} = \frac{\sigma st}{\sigma cbc} \times \frac{1}{(1.5m-1)\ (1-d'/kd)}$$

Values of ratios Asc/As2 for four different alues of d'/d (cover to compressive steel/Effective depth) for four grades of concrete and two grades of steel

σst kg/cm² grade of Steel	σcbc kg/cm² and grade of mix	Value of the Ratios Asc/Ast2 Rations of cover/depth = d'/d 0.05	0.10	0.15	0.20
1400 M.S	M15= 50	1.19	1.38	1.66	2.07
	M20= 70	1.20	1.40	1.68	2.11
	M25= 85	1.22	1.42	1.70	2.13
	M30= 100	1.23	1.44	1.72	2.15
2300 T-40	M15= 50	2.06	2.61	3.55	5.54
	M20= 70	2.09	2.65	3.60	5.63
	M25= 85	2.12	2.68	3.64	5.69
	M30= 100	2.14	2.71	3.68	5.76

TABLE - 1.3.23

M15 MIX - DOUBLY REINFORCED SECTIONS - M.R. FACTORS

Flexure - doubly reinforced beams - % of steel in tension (pt) and compression (Pc) for given M/bd², kg/cm² and compressive steel cover/beam depth ratios using different grades of concrete and steel.

M15 (1:2:4) Mix, σcbc : 50 kg/cm², Ms,(σst)= 1400 kg/cm²

$\frac{M^2}{bd^2}$	d'/d=0.05 Pt	Pc	d'/d=0.10 Pt	Pc	d'/d=0.15 Pt	Pc	d'/d= 0.20 Pt	Pc
8.70	0.717	0.003	0.717	0.004	0.717	0.005	0.717	0.006
9.00	0.739	0.030	0.741	0.037	0.742	0.046	0.744	0.062
9.50	0.777	0.070	0.780	0.091	0.784	0.116	0.789	0.154
10.0	0.815	0.119	0.820	0.146	0.826	0.186	0.833	0.247
10.50	0.852	0.163	0.860	0.201	0.868	0.256	0.878	0.340
11.0	0.89	0.208	0.899	0.256	0.910	0.325	0.923	0.432
11.50	0.927	0.252	0.939	0.311	0.952	0.395	0.967	0.525

M15 (1:2:4) Mix, σcbc : 50 kg/cm², Ms,(σst)= 1400 kg/cm²

$\frac{M^2}{bd^2}$	d'/d=0.05 Pt	Pc	d'/d=0.10 Pt	Pc	d'/d=0.15 Pt	Pc	d'/d= 0.20 Pt	Pc
12.0	0.965	0.297	0.979	0.366	0.994	0.465	1.042	0.617
12.50	1.003	0.342	1.019	0.421	1.036	0.534	1.057	0.710
13.0	1.040	0.386	1.058	0.476	1.078	0.604	1.101	0.802
13.50	1.078	0.431	1.098	0.530	1.120	0.674	1.146	0.895
14.00	1.115	0.475	1.138	0.585	1.162	0.744	1.190	0.988
14.50	1.153	0.520	1.177	0.640	1.204	0.813	1.235	1.080
15.0	1.190	0.564	1.217	0.695	1.247	0.883	1.280	1.173
15.50	1.228	0.609	1.257	0.750	1.289	0.953	1.324	1.265
16.00	1.226	0.653	1.296	0.805	1.331	1.023	1.369	1.358
16.50	1.303	0.698	1.336	0.860	1.373	1.092	1.414	1.451
17.0	1.341	0.743	1.376	0.914	1.415	1.162	1.458	1.543
17.50	1.378	0.787	1.415	0.969	1.457	1.232	1.503	1.636
18.00	1.416	0.832	1.455	1.024	1.499	1.301	1.548	1.728
18.5	1.454	0.876	1.495	1.079	1.541	1.371	1.592	1.821
19.0	1.491	0.921	1.534	1.134	1.583	1.441	1.637	1.914
19.50	1.529	0.965	1.574	1.189	1.625	1.511	1.682	2.006
20.0	1.566	1.010	1.614	1.244	1.667	1.580	1.726	2.099
20.50	1.604	1.054	1.653	1.299	1.702	1.650	1.771	2.191
21.0	1.642	1.099	1.693	1.353	1.751	1.720	1.815	2.284
21.5	1.679	1.144	1.733	1.408	1.793	1.789	1.860	2.377
22.00	1.717	1.188	1.772	1.463	1.835	1.859	1.905	2.469
22.50	1.754	0.233	1.812	1.518	1.877	1.929	1.949	2.562
23.0	1.792	1.277	1.852	1.573	1.919	1.999	1.994	2.654
23.50	1.830	1.322	1.892	1.628	1.961	2.068	2.039	2.747
24.0	1.867	1.366	1.931	1.683	2.003	2.138	2.083	2.840
24.50	1.905	1.411	1.971	1.738	2.045	2.208	2.128	2.932
25.0	1.942	1.455	2.011	1.792	2.087	2.277	2.17	3.025
25.50	1.980	1.500	2.050	1.847	2.129	2.347	2.217	3.117

26.00	2.018	1.545	2.090	1.902	2.171	2.417	2.262	3.210
26.50	2.055	1.589	2.130	1.957	2.213	2.487	2.307	3.302
27.00	2.093	1.634	2.169	2.012	2.255	2.556	2.351	3.395
27.50	2.130	1.678	2.209	2.067	2.297	2.626	2.396	3.488
28.0	2.168	1.723	2.249	2.122	2.339	2.696	2.440	3.580
28.50	2.206	1.767	2.288	2.177	2.381	2.765	2.485	3.673
29.00	2.243	1.812	2.328	2.231	2.423	2.835	2.530	3.765
29.50	2.281	1.857	2.368	2.286	2.465	2.905	2.514	3.858
30.00	2.318	1.901	2.407	2.341	2.507	2.975	2.619	3.951
30.50	2.356	1.946	2.447	2.396	2.549	3.044	2.664	4.043

Note - Pt = % Total Tensile Steel (100 At / bd)
Pc = % Compressive Steel (1000 Ac/bc)

TABLE 1.3.24

M20 - Mix - Doubly Reinforced Section - MR Factor Flexure - Doubly Reinforced Beams - % of steel in tension(Pt) and compression (Pc) for given M/bd², (kg/cm²) and compressive steel cover beam depth ratios using different grades of concrete and steel.

M20 (1:½:3) Mix, σcbc : 70 kg/cm², MS,(σst)= 1400 kg/cm²

$\frac{M}{bd^2}$	d' /d=0.05		d ' /d=0.10		d ' /d=0.15		d ' /d= 0.20	
	Pt	Pc	Pt	Pc	Pt	Pc	Pt	Pc
12.20	1.005	0.006	1.005	0.007	1.006	0.089	1.006	0.013
12.50	1.028	0.033	1.029	0.041	1.031	0.052	1.033	0.069
13.0	1.065	0.078	1.069	0.097	1.073	0.123	1.077	0.163
13.50	1.203	0.124	1.108	0.152	1.115	0.193	1.122	0.257
14.0	1.140	0.169	1.148	0.208	1.157	0.264	1.167	0.351
14.50	1.178	0.214	1.188	0.264	1.199	0.335	1.211	0.445
15.00	1.216	0.259	1.228	0.319	1.241	0.406	1.256	0.539
15.50	1.253	0.305	1.267	0.375	1.283	0.476	1.301	0.633
16.00	1.291	0.350	1.307	0.431	1.325	0.547	1.345	0.727
16.50	1.328	0.395	1.341	0.486	1.367	0.618	1.390	0.821
17.00	1.366	0.440	1.386	0.542	1.409	0.689	1.435	0.915

M20 (1:½:3) Mix, σcbc : 70 kg/cm², MS,(σst)= 1400 kg/cm²

$\frac{M}{bd^2}$	d' /d=0.05		d ' /d=0.10		d ' /d=0.15		d ' /d= 0.20	
	Pt	Pc	Pt	Pc	Pt	Pc	Pt	Pc
17.50	1.404	0.485	1.426	0.598	1.451	0.760	1.479	1.009
18.00	1.444	0.531	1.465	0.653	1.493	0.830	1.240	1.103
18.50	1.429	0.576	1.505	0.709	1.535	0.901	1.568	1.197
19.00	1.516	0.621	1.545	0.765	1.577	0.972	1.613	1.291
19.50	1.554	0.666	1.585	0.821	1.619	1.043	1.658	1.385
20.00	1.591	0.712	1.824	0.876	1.661	1.113	1.702	1.479
20.50	1.629	0.757	1.664	0.932	1.703	1.184	1.747	1.573
21.00	1.667	0.802	1.704	0.988	1.745	1.255	1.792	1.667
21.50	1.704	0.847	1.743	1.043	1.787	1.326	1.876	1.761
22.00	1.742	0.892	1.783	1.099	1.829	1.396	1.881	1.855
22.50	1.779	0.938	1.823	1.155	1.871	1.467	1.926	1.949
23.00	1.817	0.983	1.862	1.210	1.913	1.538	1.970	2.043
23.50	1.855	1.028	1.962	1.266	1.955	1.609	2.015	2.137
24.00	1.892	1.073	1.942	1.322	1.977	1.680	2.060	2.231
24.50	1.930	1.119	1.981	1.378	2.039	1.750	2.104	2.325
25.00	1.967	1.164	2.021	1.433	2.081	1.821	2.145	2.419
25.50	2.005	1.209	2.061	1.489	2.123	1.892	2.193	2.513
26.00	2.043	1.254	2.101	1.545	2.165	1.963	2.238	2.608
26.50	2.080	1.299	2.140	1.600	2.207	2.033	2.283	2.701
27.00	2.118	1.345	2.180	1.656	2.249	2.104	2.327	2.795
27.50	2.155	1.390	2.220	1.712	2.291	2.175	2.372	2.888
28.00	2.193	1.435	2.259	1.767	2.333	2.246	2.417	2.982
28.50	2.231	1.480	2.299	1.823	2.375	2.316	2.461	3.076
29.00	2.268	1.526	2.339	1.879	2.417	2.387	2.506	3.170
29.50	2.306	1.571	2.378	1.934	2.459	2.458	2.554	3.264
30.00	2.343	1.616	2.418	1.990	2.501	2.529	2.595	3.358
30.50	2.381	1.661	2.458	2.046	2.543	2.599	2.640	3.452
31.00	2.149	1.707	2.497	2.102	2.585	2.670	2.685	3.546

31.50	2.456	1.752	2.537	2.157	2.627	2.741	2.729	3.640
32.00	2.494	1.797	2.577	2.213	2.669	2.812	2.774	3.734
32.50	2.531	1.842	2.616	2.269	2.711	2.883	2.818	3.828
33.00	2.569	1.887	2.656	2.324	2.754	2.953	2.863	3.922
33.50	2.607	1.933	2.696	2.380	2.796	3.024	2.908	4.016
34.00	2.644	1.978	2.735	2.436	2.838	3.097	2.952	4.120

TABLE 1.3.25

M15 - Mix - % Steel required in Dubly reinforced BeamsWorking stress method - flexure - Doubly reinforced Rectangularsections - % of steel in tension (pt) and compression (pc) for given M/bd², (kg/cm²) and compresive steel cover/beamdepth ratios using didfferent grades of concrete and steel.

M15 (1:2:4) Mix, σcbc : 50 kg/cm², Fe 415 ot T-40 ,(σst)= 2300 kg/cm²

$\frac{M}{bd^2}$	d'/d=0.05 Pt	Pc	d'/d=0.10 Pt	Pc	d'/d=0.15 Pt	Pc	d'/d=0.20 Pt	Pc
6.60	0.317	0.007	0.318	0.010	0.318	0.014	0.318	0.023
7.00	0.336	0.045	0.337	0.060	0.338	0.087	0.340	0.144
7.50	0.359	0.092	0.361	0.123	0.364	0.177	0.367	0.295
8.00	0.381	0.139	0.385	0.186	0.389	0.268	0.394	0.446
8.50	0.404	0.187	0.409	0.249	0.415	0.359	0.421	0.596
9.00	0.427	0.234	0.433	0.312	0.441	0.450	0.448	0.747
9.50	0.450	0.281	0.458	0.375	0.466	0.540	0.476	0.898
10.00	0.473	0.328	0.482	0.438	0.492	0.631	0.503	1.048
10.50	0.496	0.375	0.506	0.501	0.517	0.722	0.530	1.199
11.00	0.519	0.422	0.530	0.564	0.543	0.812	0.557	1.350
11.50	0.542	0.469	0.554	0.627	0.568	0.903	0.584	1.501
12.00	0.564	0.577	0.578	0.690	0.594	0.994	0.611	1.651
12.50	0.587	0.564	0.603	0.753	0.620	1.085	0.639	1.802
13.00	0.610	0.611	0.627	0.816	0.645	1.175	0.666	1.953
13.50	0.633	0.658	0.657	0.879	0.671	1.266	0.693	2.104
14.00	0.656	0.705	0.675	0.942	0.696	1.357	0.720	2.254

M15 (1:2:4) Mix, σcbc : 50 kg/cm², Fe 415 ot T-40 ,(σst)= 2300 kg/cm²

$\frac{M}{bd^2}$	d'/d=0.05 Pt	Pc	d'/d=0.10 Pt	Pc	d'/d=0.15 Pt	Pc	d'/d=0.20 Pt	Pc
14.50	0.679	0.752	0.699	1.005	0.722	1.447	0.747	2.405
15.00	0.702	0.800	0.723	1.068	0.747	1.538	0.775	2.556
15.50	0.725	0.847	0.747	1.131	0.773	1.629	0.802	2.707
16.00	0.748	0.894	0.722	1.194	0.799	1.712	0.829	2.857
16.50	0.770	0.941	0.796	1.257	0.824	1.810	0.856	3.008
17.00	0.793	0.988	0.820	1.319	0.850	1.901	0.883	3.159
17.50	0.816	1.035	0.844	1.382	0.875	1.992	0.910	3.309
18.00	0.839	1.082	0.868	1.445	0.901	2.082	0.938	3.460
18.50	0.862	1.130	0.892	1.508	0.826	2.173	0.965	3.611
19.00	0.885	1.177	0.917	1.571	0.952	2.264	0.992	3.762
19.50	0.908	1.224	0.941	1.634	0.978	2.354	1.019	3.912
20.00	0.931	1.271	0.965	1.679	1.003	2.445	-	-
20.50	0.953	1.318	0.989	1.760	1.029	2.536	-	-
21.00	0.976	1.365	1.013	1.823	1.054	2.627	-	-
21.50	0.999	1.413	1.037	1.886	1.080	2.717	-	-
22.00	1.022	1.460	1.061	1.949	1.105	2.808	-	-
22.50	1.045	1.507	1.086	2.012	1.131	2.899	-	-
23.00	1.068	1.554	1.110	2.075	1.157	2.989	-	-
23.50	1.091	1.601	1.134	2.138	1.182	3.080	-	-
24.00	1.114	1.648	1.158	2.201	1.208	3.171		
24.50	1.137	1.695	1.182	2.264	1.233	3.262		
25.00	1.159	1.743	1.206	2.327	1.259	3.352		
25.50	1.182	1.790	1.231	2.390	1.284	3.443		
26.00	1.205	1.837	1.255	2.453	1.310	3.534		
26.50	1.228	1.884	1.279	2.516	1.336	3.624		
27.00	1.251	1.931	1.303	2.579	1.361	3.715		
27.50	1.274	1.978	1.327	2.642	1.387	3.806		
28.00	1.297	2.026	1.351	2.705	1.412	3.897		
28.50	1.320	2.073	1.375	2.768	1.438	3.987		

TABLE 1.3.26

M20 Mix - % Steel required for Doubly reinforced BeamsWorking Stress method - flexure - doubly reinforced Rectangular sections - % of steel in tension (pt) and compression (pc) for given M/bd², (kg/cm²) and compressive steel cover/beam depth ratios using different grades of concrete and steel.

M20 (1:1½:3) Mix, σcbc : 70 kg/cm², Fe 415 ot T-40 ,(σst)= 2300 kg/cm²

$\frac{M^2}{bd^2}$	d'/d=0.05		d'/d=0.10		d'/d=0.15		d'/d=0.20	
	Pt	Pc	Pt	Pc	Pt	Pc	Pt	Pc
9.20	0.442	0.007	0.443	0.009	0.443	0.013	0.443	0.021
9.50	0.456	0.035	0.457	0.047	0.458	0.068	0.459	0.113
10.00	0.479	0.083	0.481	0.111	0.484	0.160	0.486	0.266
10.50	0.502	0.131	0.505	0.175	0.509	0.252	0.514	0.419
11.00	0.525	0.179	0.530	0.239	0.535	0.344	0.541	0.572
11.50	0.548	0.227	0.554	0.303	0.560	0.436	0.568	0.725
12.00	0.571	0.275	0.578	0.367	0.586	0.528	0.595	0.878
12.50	0.593	0.323	0.602	0.431	0.612	0.620	0.622	1.031
13.00	0.616	0.370	0.626	0.495	0.637	0.712	0.650	1.184
13.50	0.639	0.418	0.650	0.558	0.663	0.805	0.677	1.337
14.00	0.662	0.466	0.674	0.622	0.688	0.897	0.704	1.490
14.50	0.685	0.514	0.699	0.686	0.714	0.989	0.731	1.643
15.00	0.708	0.562	0.723	0.750	0.739	1.081	0.758	1.796
15.50	0.731	0.610	0.747	0.814	0.765	1.173	0.785	1.949
16.00	0.754	0.658	0.771	0.878	0.791	1.265	0.813	2.012
16.50	0.777	0.705	0.795	0.942	0.816	1.357	0.840	2.255
17.00	0.799	0.753	0.819	1.006	0.842	1.449	0.867	2.408
17.50	0.822	0.801	0.844	1.070	0.867	1.541	0.894	2.561
18.00	0.845	0.849	0.868	1.134	0.893	1.633	0.921	2.714
18.50	0.868	0.897	0.892	1.198	0.919	1.725	0.948	2.867
19.00	0.891	0.945	0.916	1.262	0.944	1.817	0.976	3.020

M20 (1:1½:3) Mix, σcbc : 70 kg/cm², Fe 415 ot T-40 ,(σst)= 2300 kg/cm²

$\frac{M^2}{bd^2}$	d'/d=0.05		d'/d=0.10		d'/d=0.15		d'/d=0.20	
	Pt	Pc	Pt	Pc	Pt	Pc	Pt	Pc
19.50	0.914	0.993	0.940	1.325	0.970	1.909	1.003	3.173
20.00	0.937	1.040	0.964	1.389	0.995	2.002	1.030	3.326
20.50	0.960	1.088	0.988	1.453	1.021	2.094	1.057	3.479
21.00	0.982	1.136	1.013	1.517	1.046	2.186	1.084	3.632
21.50	1.005	1.184	1.037	1.581	1.072	2.278	1.111	3.785
22.00	1.028	1.232	1.061	1.645	1.098	2.370	1.139	3.978
22.50	1.051	1.280	1.085	1.709	1.123	2.462		
23.00	1.074	1.328	1.109	1.773	1.149	2.554		
23.50	1.097	1.376	1.133	1.837	1.174	2.646		
24.00	1.120	1.423	1.158	1.901	1.200	2.738		
24.50	1.143	1.471	1.182	1.965	1.225	2.830		
25.00	1.166	1.519	1.206	2.028	2.251	2.922		
25.50	1.188	1.567	1.230	2.092	1.277	3.014		
26.00	1.211	1.615	1.254	2.156	1.302	3.106		
26.50	1.234	1.663	1.278	2.220	1.328	3.198		
27.00	1.257	1.711	1.303	2.284	1.353	3.291		
27.50	1.280	1.758	1.327	2.348	1.379	3.383		
28.00	1.330	1.806	1.351	2.412	1.404	3.475		
28.50	1.326	1.854	1.375	2.476	1.430	3.567		
29.00	1.349	1.902	1.399	2.540	1.456	3.659		
29.50	1.371	1.950	1.423	2.604	1.481	3.751		
30.00	1.394	1.998	1.447	2.668	1.507	3.843		
30.50	1.417	2.046	1.472	2.731	1.532	3.935		
31.00	1.440	2.093	1.496	2.795	1.558	4.027		

1.3.4.2 Shear-Design of vertical stirrups - values of Vs/d = for two legged stirrups

Shear reinforcement through stirrups/depth in kg/cm² for two grades of steel (M.S and T-40)

Shear reinforcement shall be provided to carry a shear equal to $V - \tau c.bd$

$Vs = \sigma Sv\ Asv.d/sy$

Asv. Total area of C.S of stirrup legs in the spacing Sv.

σSv. allowable tensile stress in shear reinforcement 1400 kg/cm² (OR) 2300 Kg/cm²

TABLEI3-27 values of Vs/d for two legged stirrups is T/metre

Stirrup spacing (Sv)in cms	σSv = 1400 kg/cm² (M.S) For bars < 20 mm ϕ and 1300 Kg/cm2 for bars > 20 mm ϕ				Sv = 2300 kg/cm² (Tor 40) for bars of all sized			
	6	8	10	12	6	8	10	12
5	15.83	28.15	43.98	63.33	26.01	46.24	72.26	104.05
6	13.14	23.46	36.65	52.78	21.68	38.54	60.21	86.71
7	11.31	20.11	37.42	45.24	18.58	33.03	51.61	74.32
8	9.90	17.59	27.49	39.58	16.26	28.90	45.16	65.03
9	8.80	15.64	24.43	35.19	14.45	35.69	40.14	57.81
10	7.92	14.07	21.99	31.67	13.01	23.12	36.13	52.02
11	7.20	12.79	19.99	28.79	11.82	21.02	32.84	47.30
12	6.60	11.73	18.33	26.39	10.84	19.27	30.12	43.35
13	6.09	10.83	16.92	24.36	10.00	17.79	27.79	40.02
14	5.65	10.05	15.71	22.62	9.20	16.52	25.80	37.16
15	5.28	9.38	14.66	21.11	8.67	15.41	24.09	34.68
16	4.95	8.80	13.74	19.79	8.13	14.45	22.58	32.52
17	4.66	8.26	12.94	18.63	7.65	13.60	21.25	30.60

Stirrup spacing (Sv)in cms	σSv = 1400 kg/cm² (M.S) For bars < 20 mm ϕ and 1300 Kg/cm2 for bars > 20 mm ϕ				Sv = 2300 kg/cm² (Tor 40) for bars of all sized			
	6	8	10	12	6	8	10	12
18	4.40	7.82	12.22	17.59	7.23	12.85	20.07	28.90
19	4.17	7.41	11.57	16.67	6.50	12.17	19.01	27.38
20	3.96	7.04	11.00	15.83	6.05	11.56	18.06	26.01
25	3.17	5.63	8.80	12.67	5.20	9.25	14.45	20.81
30	2.64	4.69	7.33	10.56	4.32	7.71	12.04	17.34
35	2.26	4.02	6.26	9.05	3.72	6.61	10.32	14.86
40	1.98	3.52	5.50	7.92	3.25	5.78	9.03	13.01
45	1.76	3.13	4.89	7.04	2.89	5.14	8.03	11.56

1.3.4.3 Shear design of Bent up bars

Shear reinforement shall be provided to carry shear equal to $V - \tau c.bd$. The strength of Vs shall be calculated as

For inclined stirrups or a series of bars bent up at different (intervals) cross sections.

$$Vs = \sigma sv . \frac{Asv}{sv} . d (\sin\alpha + \cos\alpha)$$

Asv = total sectional area of bent up bars within a spacing Sv.
Sv = spacing of bent up bar along the length of member
α = angle between inclined bent up bar and axis of member, not less the 45°.

For a single bar or a single group of parallel bars, all bent up at the same cross section.

$$Vs = \sigma sv . Asv . \sin\alpha$$

TABLE 1.3.28 Shear - Bent up Bars

Bar dia in mm (ϕ) (or Fe 415)	Values of Vs. for a single bar in kgs			
	σs. 1400 kg cm² upto 20 mmϕ and 1300 kg/cm² for MS. bars - (0) beyond 20 mmϕ		σ Sv:2300 kg/cm² for T 40	
	α=45°	α =60°	α =45°	α =60°
10	778	952	1277	1564
12	1120	1371	1839	2253
16	1990	2438	3270	4005
18	2519	3086	4139	5069
20	3110	3809	5109	6258
22	3494	4280	6182	7572
25	4512	5526	7983	9777
28	5660	6932	10014	12265
32	7393	9054	13080	16019
36	9357	11460	16554	20275

1.3.4.4 Development Length and Anchorage:

The method of calculating the development length is the same as given limit state design. The difference is only in the values of the bond stresses applicable for each method.

Development length for plain bars (in Cms)

σst 1400kg/cm² for bar upto 20 mm ϕ (In tension)

σst 1300kg/cm² beyond 20 mmϕ

σst 1300kg/cm² for all diameters (in compression).

TABLE 1.3.29 DEVELOPMENT LENGTH FOR PLAIN BARS

Dia (ϕ) of bar in mm	Tension bars Grade of concrete				Compression bars Grade of concrete			
	M15	M20	M25	M30	M15	M20	M25	M30
6	35.0	26.3	23.3	21.0	26.0	19.5	17.3	15.6
8	46.7	35.0	31.1	28.0	34.7	26.0	23.1	20.8
10	58.3	43.8	38.9	35.0	43.3	32.5	28.9	26.0
12	70.0	52.5	46.7	42.0	52.0	39.0	34.7	31.2
16	93.3	70.0	62.2	56.0	69.3	52.0	46.2	41.6
18	105.0	78.8	70.0	63.0	78.0	58.5	52.0	46.8
20	116.7	87.5	77.8	70.0	86.7	65.0	57.8	52.0
22	119.2	89.4	79.4	71.5	95.3	71.5	63.6	57.2
25	135.4	101.6	90.3	81.3	108.3	81.3	72.2	65.0
28	151.7	113.8	101.1	91.0	121.3	91.0	80.9	72.8
32	173.3	130.0	115.6	104.0	138.7	104.0	92.4	83.2
36	195.0	146.3	130.0	117.0	156.0	117.0	104.0	93.6

(The above values are in cms)

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TABLE - 1.3.30
DEVELOPMENT LENGTH FOR DEFORMED BARS
(Tabulated Values are in cms)

Development length for deformed Bar (in cms) for Fe 415
σst : 2300 kg/ cm² } for all diameters
σst : 1900 kg/ cm² }

Bar Dia	Tension bars/ Grade of concrete				Compression bars Grade of concrete in mm			
	M15	M20	M25	M30	M15	M20	M25	M30
6	41.1	30.8	27.4	24.6	27.1	20.4	18.1	16.3
8	54.8	41.1	36.5	32.9	36.2	27.1	24.1	21.7
10	68.5	51.3	45.6	41.1	45.2	33.9	30.2	27.1
12	82.1	61.6	54.8	49.3	54.3	40.7	36.2	32.6
16	109.5	82.1	73.0	65.7	72.4	54.3	48.3	43.4
18	123.2	92.4	82.1	73.9	81.4	61.1	54.3	48.9
20	136.9	102.7	91.3	82.1	90.5	67.9	60.3	54.3
22	150.6	112.9	100.4	90.4	99.5	74.6	66.3	59.7
25	171.1	128.3	114.1	102.7	113.1	84.8	75.4	67.9
28	191.1	143.8	127.8	115.0	126.7	95.0	84.4	76.0
32	219.0	164.3	146.0	131.4	144.8	108.6	96.5	86.9
36	246.4	184.8	164.3	147.9	162.9	122.1	108.6	97.7

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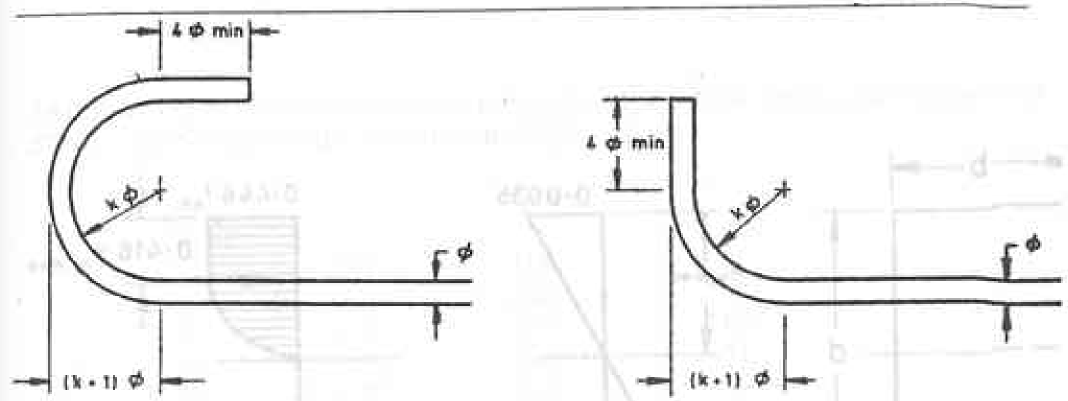
TABLE - 1.3.3DA
ANCHORAGE VALUES OF HOOKS AND BENDS
(Tabulated values are in cms)

Bar dia in mm	6	8	10	12	16	18	20	22	25	28	32	36
Anchorage value of hook	9.6	12.8	16.0	19.2	25.6	28.8	32	35.2	40	44.8	51.2	57.6
Anchorage value of 90° bend	4.8	6.4	8.0	9.6	12.8	14.4	16	17.6	20	22.4	25.6	28.8



STANDARD HOOK STANDARD 90° BEND

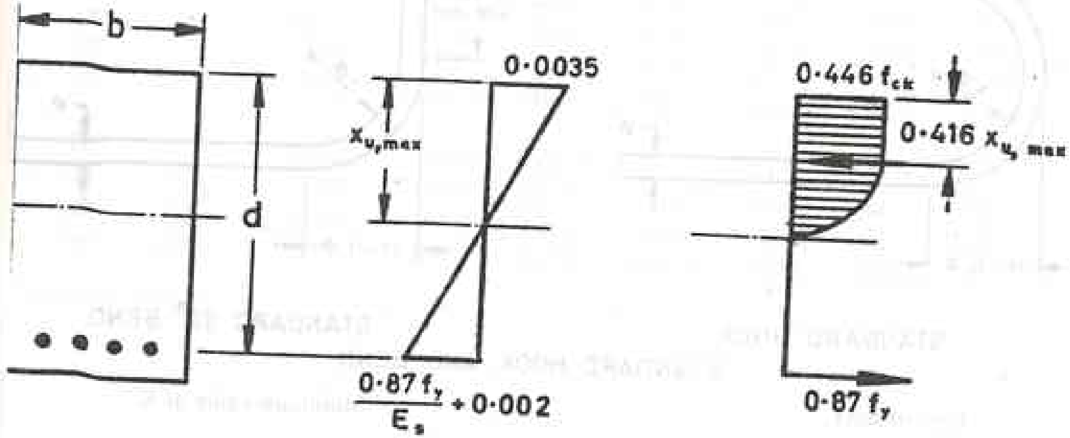
STANDARD HOOK AND BEND

Type of steel	Minimum value of K
Mild steel	2
Cold worked steel	4

Note : 1 Table is applicable for all grades of reinforcement bars.
2. Hookes and bends shall conform to the details given above.

1.3.5 DESIGN TABLES (LIMIT STATE METHOD)

Extract of SP 16 (Design Aid to IS 456 - 1978)



STRAIN DIAGRAM

STRESS DIAGRAM

SINGLY REINFORCED SECTION

TABLE A - VALUES OF X_{u} max /d FOR DIFFERENT GRADES OF STEEL

f_y, N/mm²	250	415	500
X_{u0} max /d	0.531	0.479	0.456

TABLE B - LIMITING MOMENT OF RESISTANCE AND REINFORCEMENT INDEX FOR SINGLY REINFORCED RECTANGULAR SECTIONS

f_y N/mm²	250	415	500
M_{u1} lim / f_{ck} bd²	0.149	0.138	0.133
P_{t} lim $\frac{f_y}{f_{ck}}$	21.97	19.82	18.87

TABLE C - LIMITING MOMENT OF RESISTANCE FACTOR Mu1lim/bd², N/mm² FOR SINGLY REINFORCED RECTANGULAR SECTIONS

fck N/mm²	fy N/mm²		
	250	415	500
15	2.24	2.07	2.00
20	2.98	2.76	2.66
25	3.73	3.45	3.33
30	4.47	4.14	3.99

TABLE D - MAXIMUM PERCENTAGE OF TENSILE REINFORCEMENT Pt lim FOR SINGLY REINFORCED RECTANGULAR SECTIONS

fck N/mm²	y N/mm²		
	250	415	500
15	1.32	0.72	0.57
20	1.76	0.96	0.76
25	2.20	1.19	0.94
30	2.64	1.43	1.13

Example 1 Singly Reinforced Beam

Determine the main tension reinforcement required for a rectangular beam section with the following data :

Size of beam	30 x 60 cm
Concrete mix	M 15
Characteristic strength of reinforcement	415 N/mm²
* Factored moment	170 k N.m

Assuming 25 mm dia bars with 25 mm clear cover,

Effective depth = 60 - 2.5 - 2.5/2 = 56.25 cm

From table C, for $f_y = 415$ N/mm² and $f_{ck} = 15$ N/mm²

$Mu,lim/bd^2 = 2.07$ N/mm²

$= \frac{2.07}{1000} \times (1000)^2$

$= 2.07 \times 10^3$ kN/m²

Therefore $Mu,lim = 2.07 \times 10^3 bd^2$

$= 2.07 \times 10^3 \times 30/100 \times (56.25/100)^2 = 196.5$ kN.m

* Factored moment means the moment due to characteristic loads multiplied by the appropriate value of partial safety factor (usually 1.50)

Actual moment of 170 kN.m is less than Mu,lim. The section is therefore to be designed as a singly reinforced (under reinforced) rectangular section.

METHOD OF REFERRING TO TABLES

For referring to tables, we need the value of Mu/bd^2

$$\frac{Mu}{bd^2} = \frac{170 \times 10^6}{30 \times 56.25 \times 56.25 \times 10^3}$$

$= 1.79$ N/mm²

From Table 1.3.31

Percentage of reinforcement, pt = 0.594

Therefore As = $\frac{0.594 \times 30 \times 56.25}{100}$ = 10.02 cm² = 10.02 cm²

For designing of slabs also similar procedure may be adopted, assuming appropriate value of d (depth of slab) and width b as 100 cm.

*Factored moment means the moment due to characteristic loads multiplied by the appropriate value of partial safety factor (usually 1.50)

$Mu_2 = MU - Mv_1,Lim$

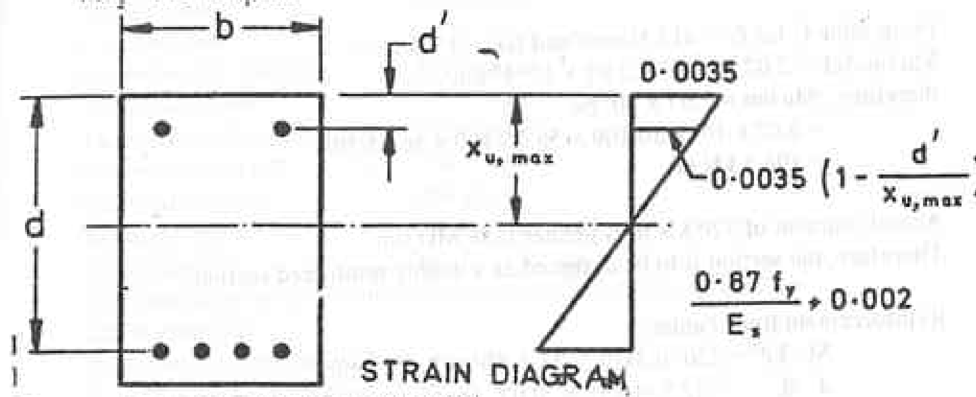


FIG. DOUBLY REINFORCED SECTION

TABLE E

STRESS IN COMPRESSION REINFORCEMENT f_{sc} N/mm² IN DOUBLY REINFORCED BEAMS WITH COLD WORKED BARS

f_{y0} N/mm²	d' / d 0.05	0.10	0.15	0.20
415	355	353	342	329
500	424	412	395	370

Example 2 Doubly Reinforced Beam

Determine the main reinforcement required for a rectangular beam section with the followingdata:

Size of beam	30 x 60 cm
Concrete mix	M 15
Characteristic strength of Reinforcement	415 N/mm²

Factored moment 320 kN.m

Assuming 25 mm dia bars with 25 mm clear cover,
d= 60 - 2.5 - 2.5/2 =56.25 cm

From table C for fy = 415 N/mm² and fck = 15 N/mm²
Mu lim/bd² = 2.07 N/mm² = 2.07 x 10³ kN/m²,
therefore, Mu lim = 2.07 x 10³ bd²
= 2.07 x 10³ x 30/100 x 56.25/100 x 56.25/100
= 196.5 kN.m

Actual moment of 320 kN.m is greater than MU_l lim
Therefore, the section is to be designed as a doubly reinforced section .

Reinforcement from Tables :-

Mu/bd² = 320/ 0.3x(0.5625)²x10³ = 3.37 N/mm²

$d'/d = \frac{(2.5+1.25)}{(56.25)} = 0.07$

Next higher value of d'/d = 0.1 will be used for referring to Tables.

Referring to Table 1.3.33 corresponding to Mu/bd² = 3.37 and d'/d = 0.1,

pt = 1.117, pc = 0.418

Therefore Ast = 18.85 cm², Asc = 7.05 cm²

EXAMPLE 3 Square Column with Uniaxial Bending

Determine the reinforcement to be provided in a square column subjected to uniaxial bending, with the following data :

Size of column	45 x 45 cm
Concrete mix	M 25
Characteristic strength of reinforcement	415 N/mm²
Factored load(characteristic load multiplied by)	2500 kN
Factored moment	200 kN.m
Arrangement of reinforcement	(a) On two sides (b) On four sides

(Assume moment due to minimum eccentricity to be less than the actual moment).

Assuming 25 mm bars with 40 mm cover,
d' = 40 + 12.5 = 52.5 mm = 5.25 cm
d'/D = 5.25 / 45 = 0.12
Charts for d'/D = 0.15 will be used
Pu / fck bD = 2500 x 10³ / 25 x 45 x 45 x 10² = 0.494
Mu / fck. bD² = 200 x 10⁶/ 25x45x45x45x10³= 0.088

a) Reinforcement on two sides,

Referring to Chart 3

p/f_{ck} = 0.09

Percentage of reinforcement, p = 0.09 x 25 = 2.25

As = p b D/100 = 2.25x45x45/100 = 45.56 cm²

b) Reinforcement on four sides from Chart 7,

p/f_{ck} = 0.10

p = 0.10 x 25 = 2.5

As = 2.5 x 45 x 45/100 = 50.63 cm²

Example 4 Circular Column with Uniaxial Bending

Determine the reinforcement to be provided in a circular column with the following data:

Diameter of column	50 cm
Grade of concrete	M 20
Characteristic strength of reinforcement	250 N/mm² for bars upto 20 mmϕ

240 N/mm² for bars over 20 mmϕ

Factored load	1600 kN
Factored moment	125 kN.m

Lateral reinforcement :

a) Hoop reinforcement

b) Helical reinforcement

(Assume moment due to minimum eccentricity to be less than the actual moment).

Assuming 25 mm bars with 40 mm cover,

$d' = 40 + 12.5 = 52.5$ mm $= 5.25$ cm

$d'/D = 5.25/50 = 0.105$

Charts for $d'/D = 0.10$ will be used.

Column with hoop reinforcement

$Pu/fck\ D^2 = 1600x10^3 / 20x50x50x10^2 = 0.32$

$Mu/fck\ D^3 = 125x10^6 / 20x50x50x50x10^3 = 0.05$

Referring to Chart 15, for fy = 250 N/mm²

$p/fck = 0.87$

$p = 0.87 \times 20 = 1.74$

$As = p\pi D^2/400 = 1.74x\pi x50x50/400 = 34.16$ cm²

For fy=240 N/mm²,

$As = 34.16 \times 250/240 = 35.58$ cm²

Example 5 Rectangular Column with Biaxial Bending

Determine the reinforcement to be provided in a short column subjected to biaxial bending, with the following data:

Size of column	40 x 60 cm
Concrete mix	M 15
Charecteristic strength of reinforcement	415 N/mm²
Factored load, Pu	1600 kN
Factored moment acting parallel to the larger dimension, Mux	120 kN
Factored moment acting parallel to the shorter dimension, Muy	90 kN

Moments due to minimum eccentricity are less than the values given above.

Reinforcement is distributed equally on four sides.

As a first trial assume the reinforcement percentage, p=1.2

$p/fck = 1.2/15 = 0.08$

Uniaxial moment capacity of the section about xx-axis :

$d'/D = 5.25 / 60 = 0.0875$

Chart for $d'/D = 0.1$ will be used.

$Pu/fck\ bD = 1600x10^3 / 15x40x60x10^2 = 0.444$

Referring to Chart 6, $Mu/fck\ b\ D^2 = 0.09$

∴ $Mux1 = 0.09x15x40x60^2x10^3 / 10^6 = 194.4$ k N.m

Uniaxial moment capacity of the section about yy-axis :

$d'/D = 5.25/40 = 0.131$

Chart for $d'/D = 0.15$ will be used.

Referring to Chart 7, $Mu/fck\ b\ D^2 = 0.083$

$Muy1 = 0.083x15x60x40^2x10^3 / 10^6 = 119.52$ kN.m

Calculation of Puz :

Referring to Chart 13 corresponding to p=1.2, fy=415 and fck = 15,

$Puz / Ag = 10.3$ N/mm²

$Puz = 10.3\ Ag = 10.3x40x60x10^2 / 10^3$ kN

$= 2472$ kN

$P_u/P_{uz} = 1600 / 2472 = 0.647$

$M_{ux}/M_{ux1} = 120 / 194.4 = 0.617$

$M_{uy}/M_{uy1} = 90 / 119.52 = 0.753$

Referring to Chart 14, the permissible value of Mux/Mux1 corresponding to the above values of Muy/Muy1 and Pu/Puz is equal to 0.58.

The actual value of 0.617 is only slightly higher than the value read from the Chart. This can be made up by slight increase in reinforcement.

$As = 1.2x40x60 / 100 = 28.8$ cm²

12 bars of 18 mm will give As = 30.53 cm²

Reinforcement percentage provided,

$p = 30.53 \times 100 / 60 \times 40 = 1.27$

With this percentage, the section may be rechecked as follows

$p/f_{ck} = 1.27 / 15 = 0.0847$

Referring to Chart 6,

$M_u / f_{ck}\, b\, D^2 = 0.095$

$M_{ux1} = 0.095 \times 15 \times 40 \times 60^2 \times 10^3 / 10^6$

$= 205.2$ kN.m

Referring to Chart 7,

$M_u / f_{ck}\, b\, D^2 = 0.085$

$M_{uy1} = 0.085 \times 15 \times 60 \times 40^2 \times 10^3 / 10^6 = 122.4$ kN.m

Referring to Chart 13

$P_{uz}/A_g = 10.4$ N/mm²

$\therefore P_{uz} = 10.4 \times 60 \times 40 \times 10^2 / 10^3 = 2496$ kN

$P_u / P_{uz} = 1600 / 2496 = 0.641$

$M_{ux} / M_{ux1} = 120 / 205.2 = 0.585$

$M_{uy} / M_{uy1} = 90 / 122.4 = 0.735$

Referring to Chart 14

Corresponding to the above values of Muy /Muy1 and Pu/Puz, the permissible value of Mux / Mux1 is 0.6.

Hence the section is O.K.

Example 6 Shear

Determine the shear reinforcement (vertical stirrups) required for a beam section with the following data:

Beam size	30 x 60 cm
Depth of beam	60 cm
Concrete grade	M 15
Characteristic strength of stirrup reinforcement	250 N/mm²
Tensile reinforcement percentage	0.8
Factored shear force, Vu	180 k N

Assuming 25 mm dia bars with 25 mm cover,

$d = 60 - 2.5/2 - 2.5 = 56.25$ cm

Shear stress, $\tau_v = V_u / bd = 180 \times 10^3 / 30 \times 56.25 \times 10^2$

$= 1.07$ N/mm²

From Table 1.2.7(D) for M15, $\tau_{c,max} = 2.5$ N/mm²

τ_v is less than $\tau_{c,max}$

From Table 1.2.7(C) for Pt = 0.8, $\tau_c = 0.55$ N/mm²

Shear capacity of concrete section = $\tau_c\, bd$

$= 0.55 \times 30 \times 56.25 \times 10^2 / 10^3 = 92.8$ kN

Shear to be carried by stirrups, $V_{us} = V_s - \tau_c bd$

$= 180 - 92.8 = 87.2$ kN

$V_{us} / d = 87.2 / 56.25 = 1.55$ kN/cm

Referring to Table 1.3.37 for steel fy = 250 N/mm².

Provide 8 mm diameter two legged vertical stirrups at 14 cm spacing.

Example 7 Torsion

Determine the reinforcements required for a rectangular beam section with the following data :

Size of the beam	30 x 60 cm
Concrete grade	M 15
Characteristic strength of steel	415 N/mm²
Factored shear force	95 kN
Factored torsional moment	45 kN.m
Factored bending moment	115 kN.m

Assuming 25 mm dia bars with 25 mm cover,

$d = 60 - 2.5 - 2.5/2 = 56.25$ cm

Equivalent shear, $V_c = V + 1.6\,(T / b)$

$= 95 + 1.6 \times 45/0.3 = 95 + 240 = 335$ kN

Equivalent shear stress, $\tau_{vc} = V_c / bd$

$= 335 \times 10^3 / 30 \times 56.25 \times 10^2$

$= 1.99$ N/mm²

For Table 1.2.7(D) for M15, $\tau_{c,max} = 2.5$ N/mm²

τ_{vc} is less than $\tau_{c,max}$; hence the section does not require revision.

From Table 1.2.7(C) for an assumed value of pt = 0.5

$\tau_c = 0.46$ N/mm2 $< \tau_{vc}$.

Hence longitudinal and transverse reinforcements are to be designed

<u>Longitudinal reinforcement</u>

Equivalent bending moment, $M_{e1} = M_u + M_t$

$= M_u + T_u(1 + D/b) / 1.7$

$= 115 + 45(1+60/30)/1.7$
$= 115 + 79.4$
$= 194.4$ kN.m

$Mc1/bd^2 = 194.4 \times 10^6 / 30 \times (56.25)^2 \times 10^3$
$= 2.05$ N/mm^2

Referring to table 1.3.31 corresponding to $Mu/bd^2 = 2.05$
pt = 0.708
$Ast = 0.708 \times 30 \times 56.25/100 = 11.95$ cm^2

Provide 4 bars of 20 mm dia (Ast = 12.56 cm^2) on the flexural tensile face. As Mt is less than Mu, we need not consider Mc2

Therefore, provide only two bars of 12 mm dia on the compression face, one bar being at each corner.

As the depth of the beam is more than 45 cm, side face reinforcement of 0.05% on each side is to be provided

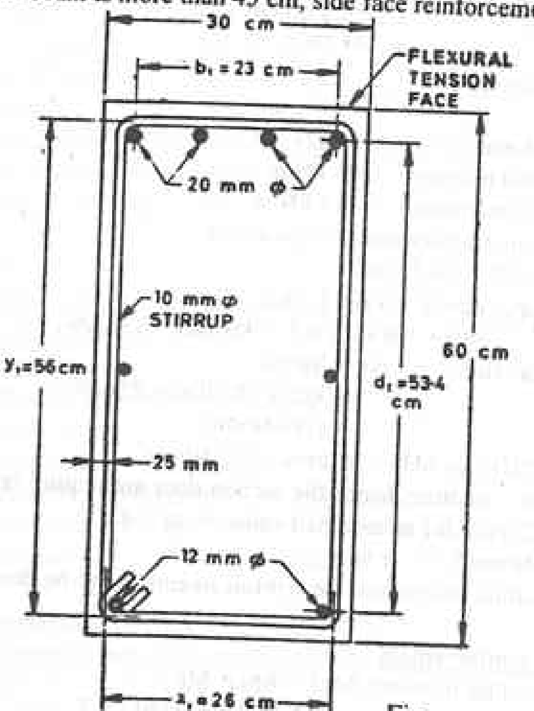


Fig.

Providing one bar at the middle of each side,

Spacing of bar = 53.4/2 = 26.7 cm
Area required for each bar = $0.05 \times 30 \times 26.7 / 100 = 0.40$ cm^2
Provide one bar of 12 mm dia ~~on each~~ side.
Transverse reinforcement
Area of two legs of the stirrup should satisfy the following :
$Asu = Tu\,Sv / b1d1(0.87\,fy) + Vu\,Sv / 2.5\,d1\,(0.87\,fy)$

Assuming Diameter of stirrups as 10mm
$= 60 - (2.5+1.0) - (2.5+0.6) = 53.4$ cm
$30 - 2(2.5+1.0) = 23$ cm

$Asv\,(0.87\,fy) / Sv = 45 \times 10^6 / 23 \times 53.4 \times 10^2$
$+ 95 \times 10^3 / 2.5 \times 53.4 \times 10$
$= 366.4 + 71.2 = 437.6$ N/mm
$= 4.38$ kN/cm

Area of all the legs of the stirrup should satisfy the condition that Asv/Sv should not be less than $(\tau ve - \tau c)b / 0.87\,fy$
From Table 1.2.7(C) for tensile reinforcement percentage of 0.71,the value of τc is 0.53 N/mm^2
$Asv\,(0.87\,fy) / Sv = (\tau vc - \tau c)b$
$= (1.99 - 0.53)\quad 30/10$
$= 438$ N/mm $= 4.38$ kN/cm

Note :- It is only a coincidence that the values of Asv(0.87fy)/Sv calculated by the two equations are the same.
Referring Table 1.3.37 (for fy = 415 N/mm^2).
Provide 10mm ϕ two legged stirrups at 12.5 cm spacing.
According to IS 456-1978 the spacing of stirrups shall not exceed x_1, $(x_1+y_1)/4$ and 300 mm, where x_1 and y_1 are the short and long dimensions of the stirrup.
$x_1 = 30 - 2(2.5-0.5) = 26$ cm
$y_1 = 60 - 2(2.5 - 0.5) = 56$ cm
$(x_1+y_1)/4 = (26 + 56)/4 = 20.5$ cm
10mm ϕ two legged stirrups at 12.5 cm spacing will satisfy allthe codal requirements.

TABLE 1.3.31

FLEXURE - REINFORCEMENT PERCENTAGE, PI FOR SINGLY REINFORCED SECTIONS

fck = 15N / mm²

Mu/bd² N/mm²	fy' N/mm² 240	250	415	480	500
0.30	0.147	0.141	0.085	0.074	0.071
0.35	0.172	0.166	0.100	0.086	0.083
0.40	0.198	0.190	0.114	0.099	0.095
0.45	0.224	0.215	0.129	0.112	0.107
0.50	0.250	0.240	0.144	0.125	0.120
0.55	0.276	0.265	0.159	0.138	0.132
0.60	0.302	0.290	0.175	0.151	0.145
0.65	0.329	0.316	0.190	0.164	0.158
0.70	0.356	0.342	0.206	0.178	0.171
0.75	0.383	0.368	0.221	0.191	0.184
0.80	0.410	0.394	0.237	0.205	0.197
0.82	0.421	0.405	0.244	0.211	0.202
0.84	0.433	0.415	0.250	0.216	0.208
0.86	0.444	0.426	0.257	0.222	0.213
0.88	0.455	0.437	0.263	0.227	0.218
0.90	0.466	0.448	0.270	0.233	0.224
0.92	0.477	0.458	0.276	0.239	0.229
0.94	0.489	0.469	0.283	0.244	0.235
0.96	0.500	0.480	0.289	0.250	0.240
0.98	0.512	0.491	0.296	0.256	0.246
1.00	0.523	0.502	0.303	0.262	0.251
1.02	0.535	0.513	0.309	0.267	0.257
1.04	0.546	0.524	0.316	0.273	0.262
1.06	0.558	0.536	0.323	0.279	0.268
1.08	0.570	0.547	0.329	0.285	0.273
1.10	0.581	0.558	0.336	0.291	0.279
1.12	0.593	0.570	0.343	0.297	0.285
1.14	0.605	0.581	0.350	0.303	0.290
1.16	0.617	0.592	0.357	0.309	0.296
1.18	0.629	0.604	0.364	0.315	0.302
1.20	0.641	0.615	0.371	0.321	0.308
1.22	0.653	0.627	0.378	0.327	0.314

1.24	0.665	0.639	0.385	0.333	0.319
1.26	0.678	0.650	0.392	0.339	0.325
1.28	0.690	0.662	0.399	0.345	0.331
1.30	0.702	0.674	0.406	0.351	0.337
1.32	0.715	0.686	0.413	0.357	0.343
1.34	0.727	0.698	0.420	0.364	0.349
1.36	0.740	0.710	0.428	0.370	0.355
1.38	0.752	0.722	0.435	0.376	0.361
1.40	0.765	0.734	0.442	0.382	0.367
1.42	0.778	0.747	0.450	0.389	0.373
1.44	0.790	0.759	0.457	0.395	0.379
1.46	0.803	0.771	0.465	0.402	0.386
1.48	0.816	0.784	0.472	0.408	0.392
1.50	0.829	0.796	0.480	0.415	0.398
1.52	0.842	0.809	0.487	0.421	0.404
1.54	0.856	0.821	0.495	0.428	0.411
1.56	0.869	0.834	0.503	0.434	0.417
1.58	0.882	0.847	0.510	0.441	0.423
1.60	0.896	0.860	0.518	0.448	0.430
1.62	0.909	0.873	0.526	0.455	0.436
1.64	0.923	0.886	0.534	0.461	0.443
1.66	0.936	0.899	0.542	0.468	0.449
1.68	0.950	0.912	0.550	0.475	0.456
1.70	0.964	0.925	0.558	0.482	0.463
1.72	0.978	0.939	0.566	0.489	0.469
1.74	0.992	0.952	0.574	0.496	0.476
1.76	1.006	0.966	0.582	0.503	0.483
1.78	1.020	0.980	0.590	0.510	0.490
1.80	1.035	0.993	0.598	0.517	0.497
1.82	1.049	1.007	0.607	0.525	0.504
1.84	1.064	1.021	0.615	0.532	0.511
1.86	1.078	1.035	0.624	0.539	0.518
1.88	1.093	1.049	0.632	0.546	0.525
1.90	1.108	1.063	0.641	0.554	0.532
1.92	1.123	1.078	0.649	0.561	0.539
1.94	1.138	1.092	0.658	0.569	0.546

1.96	1.153	1.107	0.667	0.576	0.553
1.98	1.168	1.121	0.676	0.584	0.561
2.00	1.184	1.136	0.685	0.592	
2.02	1.199	1.151	0.693		
2.04	1.215	1.166	0.703		
2.06	1.231	1.181	0.712		
2.08	1.247	1.197			
2.10	1.263	1.212			
2.12	1.279	1.228			
2.14	1.295	1.243			
2.16	1.312	1.259			
2.18	1.328	1.275			
2.20	1.345	1.291			
2.22	1.362	1.308			
2.24	1.379				

Note : Blanks indicate inadmissible reinforcement percentage.

TABLE 1.3.32

FLEXURE - LIMITING MOMENT OF RESISTANCE FACTOR Mu lim /bwd² fck FOR SINGLY REINFORCED T-BEAMS, N/mm²

fy = 415 N/mm²

Df/d	bf / bw									
	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0
0.06	0.138	0.164	0.190	0.216	0.242	0.268	0.294	0.320	0.346	0.372
0.07	0.138	0.168	0.198	0.228	0.259	0.289	0.319	0.349	0.379	0.409
0.08	0.138	0.172	0.207	0.241	0.275	0.309	0.344	0.378	0.412	0.446
0.09	0.138	0.176	0.215	0.253	0.291	0.330	0.368	0.406	0.445	0.483
0.10	0.138	0.180	0.223	0.265	0.308	0.350	0.392	0.435	0.477	0.519
0.11	0.138	0.184	0.231	0.277	0.324	0.370	0.416	0.463	0.509	0.555
0.12	0.138	0.188	0.239	0.289	0.339	0.390	0.440	0.490	0.541	0.591
0.13	0.138	0.192	0.247	0.301	0.355	0.409	0.463	0.518	0.572	0.626

0.14	0.138	0.196	0.254	0.312	0.370	0.429	0.487	0.545	0.603	0.661
0.15	0.138	0.200	0.262	0.324	0.386	0.448	0.509	0.571	0.633	0.695
0.16	0.138	0.204	0.269	0.335	0.401	0.466	0.532	0.598	0.663	0.729
0.17	0.138	0.207	0.277	0.346	0.416	0.485	0.554	0.624	0.693	0.762
0.18	0.138	0.211	0.284	0.357	0.430	0.503	0.576	0.649	0.723	0.796
0.19	0.138	0.215	0.291	0.368	0.445	0.522	0.598	0.675	0.752	0.828
0.20	0.138	0.218	0.299	0.379	0.459	0.540	0.620	0.700	0.780	0.861
0.21	0.138	0.221	0.305	0.388	0.471	0.554	0.638	0.721	0.804	0.887
0.22	0.138	0.224	0.309	0.395	0.480	0.566	0.651	0.737	0.822	0.908
0.23	0.138	0.226	0.314	0.402	0.489	0.577	0.665	0.753	0.841	0.928
0.24	0.138	.0228	0.318	0.408	0.498	0.588	0.678	0.768	0.859	0.949
0.25	0.138	0.230	0.323	0.415	0.507	0.600	0.692	0.784	0.876	0.969
0.26	0.138	0.233	0.327	0.422	0.516	0.611	0.705	0.800	0.894	0.989
0.27	0.138	0.235	0.331	0.428	0.525	0.622	0.718	0.815	0.912	1.008
0.28	0.138	0.237	0.336	0.435	0.534	0.632	0.731	0.830	0.929	1.028
0.29	0.138	0.239	0.340	0.441	0.542	0.643	0.744	0.845	0.946	1.047
0.30	0.138	0.241	0.344	0.448	0.551	0.654	0.757	0.860	0.963	1.066
0.31	0.138	0.243	0.349	0.454	0.559	0.664	0.770	0.875	0.980	1.085
0.32	0.138	0.245	0.353	0.460	0.568	0.675	0.782	0.890	0.997	1.104
0.33	0.138	0.248	0.357	0.466	0.576	0.685	0.795	0.904	1.014	1.123
0.34	0.138	0.250	0.361	0.473	0.584	0.696	0.807	0.919	1.030	1.142
0.35	0.138	0.252	0.365	0.479	0.592	0.706	0.819	0.933	1.046	1.160
0.36	0.138	0.254	0.369	0.485	0.600	0.716	0.831	0.947	1.063	1.178
0.37	0.138	0.256	0.373	0.491	0.608	0.726	0.843	0.961	1.079	1.196
0.38	0.138	0.258	0.377	0.497	0.616	0.736	0.855	0.975	1.094	1.214
0.39	0.138	0.260	0.381	0.503	0.624	0.746	0.867	0.989	1.110	1.232
0.40	0.138	0.262	0.385	0.508	0.632	0.755	0.879	1.002	1.126	1.249
0.41	0.138	0.263	0.389	0.514	0.640	0.765	0.890	1.016	1.141	1.267
0.42	0.138	0.265	0.393	0.520	0.647	0.775	0.902	1.029	1.156	1.284
0.43	0.138	0.267	0.396	0.526	0.655	0.784	0.913	1.042	1.172	1.301
0.44	0.138	0.269	0.400	0.531	0.662	0.793	0.924	1.055	1.187	1.318
0.45	0.138	0.271	0.404	0.537	0.670	0.803	0.936	1.068	1.201	1.334

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TABLE 1.3.33

FLEXURE - REINFORCEMNT PERCENTAGES FOR DOUBLY REINFORCED SECTIONS

fck = 15 N/mm²
fy = 415 N/mm²

Mu/bd²	d' / d = 0.05		d'/d =0.10		d'/d =0.15		d'/d = 0.20	
N/mm2	Pt	Pc	Pt	Pc	Pt	Pc	Pt	Pc
2.08	0.719	0.003	0.720	0.003	0.720	0.003	0.720	0.003
2.10	0.725	0.009	0.726	0.009	0.726	0.010	0.727	0.011
2.20	0.754	0.039	0.757	0.041	0.759	0.045	0.761	0.050
2.30	0.784	0.069	0.787	0.073	0.791	0.080	0.796	0.089
2.40	0.813	0.099	0.818	0.106	0.824	0.115	0.831	0.127
2.50	0.842	0.129	0.849	0.138	0.857	0.150	0.865	0.166
2.60	0.871	0.160	0.880	0.170	0.889	0.185	0.900	0.205
2.70	0.900	0.190	0.910	0.202	0.922	0.220	0.935	0.244
2.80	0.929	0.220	0.941	0.234	0.954	0.255	0.969	0.282
2.90	0.959	0.250	0.972	0.267	0.987	0.290	1.004	0.321
3.00	0.988	0.280	1.003	0.299	1.020	0.325	1.039	0.360
3.10	1.017	0.311	1.034	0.331	1.052	0.360	1.073	0.399
3.20	1.046	0.341	1.064	0.363	1.085	0.395	1.108	0.438
3.30	1.075	0.371	1.095	0.395	1.117	0.430	1.142	0.476
3.40	1.104	0.401	1.126	0.427	1.150	0.465	1.177	0.515
3.50	1.134	0.432	1.157	0.460	1.183	0.500	1.212	0.554
3.60	1.163	0.462	1.188	0.492	1.215	0.535	1.246	0.593
3.70	1.192	0.492	1.218	0.524	1.248	0.571	1.281	0.631
3.80	1.221	0.522	1.249	0.556	1.280	0.606	1.316	0.670
3.90	1.250	0.552	1.280	0.588	1.313	0.641	1.350	0.709
4.00	1.279	0.583	1.311	0.621	1.346	0.676	1.385	0.748
4.10	1.309	0.613	1.342	0.653	1.378	0.711	1.420	0.787
4.20	1.338	0.643	1.372	0.685	1.411	0.746	1.454	0.825
4.30	1.367	0.673	1.403	0.717	1.443	0.781	1.489	0.864
4.40	1.396	0.703	1.434	0.749	1.476	0.816	1.524	0.903

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FLEXURE - REINFORCEMNT PERCENTAGES FOR DOUBLY REINFORCED SECTIONS

fck = 15 N/mm²
fy = 415 N/mm²

Mu/bd²	d' / d = 0.05		d'/d =0.10		d'/d =0.15		d'/d = 0.20	
N/mm2	Pt	Pc	Pt	Pc	Pt	Pc	Pt	Pc
4.50	1.425	0.734	1.465	0.781	1.509	0.851	1.558	0.942
4.60	1.455	0.764	1.495	0.814	1.541	0.886	1.593	0.980
4.70	1.484	0.794	1526	0.846	1.574	0.921	1.627	1.019
4.80	1.513	0.824	1.557	0878	1.606	0.956	1.662	1.058
4.90	1.542	0.855	1.588	0.910	1.639	0.991	1.697	1.097
5.00	1.571	0.885	1.619	0.942	1.672	1.026	1.731	1.136
5.10	1.600	0.915	1.649	0.975	1.704	1.061	1.766	1.174
5.20	1.630	0.945	1.680	1.007	1.737	1.096	1.801	1.213
5.30	1.659	0.975	1.711	1.039	1.769	1.131	1.835	1.252
5.40	1.688	1.006	1.742	1.071	1.802	1.166	1.870	1.291
5.50	1.717	1.036	1.773	1.103	1.835	1.201	1.905	1.329
5.60	1.746	1.066	1.803	1.136	1.867	1.236	1.939	1.368
5.70	1.775	1.096	1.834	1.168	1.900	1.271	1.974	1.407
5.80	1.805	1.126	1.865	1.200	1.932	1.306	2.008	1.446
5.90	1.834	1.157	1.896	1.232	1.965	1.341	2.043	1.485
6.00	1.863	1.187	1.927	1.264	1.998	1.376	2.078	1.523
6.10	1.892	1.217	1.957	1.296	2.030	1.411	2.112	1.562
6.20	1.921	1.247	1.988	1.329	2.063	1.446	2.147	1.601
6.30	1.950	1.278	2.019	1.361	2.095	1.481	2.182	1.640
6.40	1.980	1.308	2.050	1.393	2.128	1.517	2.216	1.678

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TABLE 1.3.34
MINIMUM SHEAR REINFORCEMENT (STIRRUPS) IN BEAMS FOR DIFFERENT WIDTHS

Dia mm	Area of 2 legs mm²	230	250	300	350	380	400	450
Steel : Fe 250		Maximum spacing of stirrups in mm						
6	56	150	140	225	100			
8	100	270	250	200	175	165	155	135
10	157	425	390	325	280	255	245	215
12.	226	450	450	450	400	370	350	310
Steel : Fe 415								
6	56	250	230	190	165	150	145	125
8	100	450	415	345	205	270	260	230
10	157	450	450	450	450	425	405	360
12	226	450	450	450	450	450	450	450

TABLE 1.3.35
ULTIMATE SHEAR RESISTANCE OF MINIMUMVERTICAL STIRRUPS IN kN FOR DIFFERENT WIDTHS AND DEPTHS OF RECTANGULAR BEAMS - CONCRETE M 15

Depth in mm Total	Effec--tive	Width of Beam in mm 230	250	300	350	380	400	450
300	250	20.12	21.87	26.25	30.62	33.25	35.0	39.37
350	300	24.15	26.25	31.50	36.75	39.90	42.0	47.25
380	330	26.56	28.87	34.65	40.42	43.89	46.2	51.97
400	350	28.17	30.62	36.75	42.87	46.55	49.0	55.12
450	400	32.20	35.00	42.00	49.00	53.20	56.0	63.00
500	450	36.22	39.37	47.25	55.12	59.85	63.0	70.87
530	480	38.64	42.00	50.40	58.80	63.84	67.2	75.60
550	500	40.25	43.75	52:50	61.25	66.50	70.0	78.75

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600	550	44.27	48.12	57.75	67.37	73.15	77.0	86.62
650	600	48.30	52.50	63.00	73.50	79.80	84.0	94.50
680	630	50.71	55.12	66.15	77.17	83.79	88.2	99.22
700	650	52.32	56.87	68.25	79.62	86.45	91.0	102.37
750	700	56.35	61.25	73.50	85.75	93.10	98.0	110.25

Notes :

1. For obtaining Shear resistance for concrete grade M20 multiply the above values by 36/35.
2. For obtaining Shear resistance at working loads multiply the above values by 22/35.

TABLE 1.3.36
DESIGN FOR SHEAR - LIMIT STATE METHOD

a) Design shear strength Tcd in N/mm²						b)Shear strength of bentup Bars in kN				
Pt %	M15	M20	Pt %	M15	M20	Dia mm	Fe 250 0-45°	0-60°	Fe415 0-45°	0-60°
0.25	0.35	0.36	1.40	0.67	0.70	10	12..05	14.79	20.05	24.56
0.30	0.38	0.39	1.50	0.68	0.72	12	17.39	21.30	28.87	35.36
0.40	0.43	0.44	1.60	0.69	0.73	16	30.92	37.87	51.33	62.87
0.50	0.46	0.48	1.70	0.71	0.75	18	39.14	47.93	64.97	79.87
0.60	0.50	0.51	1.80	0.71	0.76	20	48.32	59.18	80.21	98.23
0.70	0.53	0.55	1.90	0.71	0.77	22	58.46	71.60	97.05	118.86
0.80	0.55	0.57	2.00	0.71	0.79	25	75.49	92.46	125.32	153.48
0.90	0.57	0.60	2.10	0.71	0.80	28	94.70	115.98	157.20	192.53
1.00	0.60	0.62	2.20	0.71	0.81	32	123.69	151.49	205.32	251.47
1.10	0.62	0.64	2.30	0.71	0.82					
1.20	0.63	0.66	2.40	0.71	0.82					

θ =angle between bar &axis of the member

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TABLE 1.3.37

SHEAR - VERTICAL STIRRUPS

Values of Vus/d for two legged stirrups, kN/cm.

Stirrup spacing cm	f_y = 250 N/mm² Diameter, mm				f_y = 415 N/mm² Diameter, mm			
	6	8	10	12	6	8	10	12
5	2.460	4.373	6.833	9.839	4.083	7.259	11.34	16.334
6	2.050	3.644	5.694	8.200	3.403	6.049	9.452	13.611
7	1.757	3.124	4.881	7.028	2.917	5.185	8.102	11.667
8	1.537	2.733	4.271	6.150	2.552	4.537	7.089	10.208
9	1.367	2.429	3.796	5.466	2.269	4.033	6.302	9.074
10	1.230	2.100	3.416	4.920	2.042	3.630	5.671	8.167
11	1.118	1.988	3.106	4.472	1.886	3.299	5.150	7.424
12	1.028	1.822	2.847	4.100	1.701	3.026	4.720	6.806
13	0.946	1.682	2.628	3.784	1.571	2.702	4.363	6.286
14	0.879	1.562	2.440	3.514	1.458	2.593	4.051	5.833
15	0.820	1.458	2.278	3.280	1.361	2.420	3.781	5.445
16	0.769	1.366	2.135	2.075	1.276	2.269	3.545	5.104
17	0.723	1.286	2.010	2.894	1.201	2.135	3.336	4.804
18	0.683	1.215	1.898	2.733	1.134	2.016	3.151	4.537
19	0.647	1.151	1.798	2.589	1.075	1.910	2.985	4.298
20	0.615	1.093	1.708	2.460	1.020	1.815	2.836	4.083
25	0.492	0.875	1.367	1.968	0.817	1.452	2.269	3.267
30	0.410	0.729	1.139	1.640	0.681	1.210	1.890	2.722
35	0.351	0.625	0.976	1.406	0.583	1.037	1.620	2.333
40	0.307	0.547	0.854	1.230	0.510	0.907	1.418	2.042
45	0.273	0.486	0.759	1.093	0.454	0.807	1.260	1.815

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TABLE 1.3.38 FLEXURE — MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 15 N/mm²
f_y = 415 N/mm²
Thickness = 10·0 cm

Bar Spacing, cm	Bar Diameter, mm 6	8	10	12
5	13·53	0·00	0·00	0·00
6	11·72	0·00	0·00	0·00
7	10·32	0·00	0·00	0·00
8	9·21	0·00	0·00	0·00
9	8·31	13·20	0·00	0·00
10	7·56	12·16	0·00	0·00
11	6·94	11·26	0·00	0·00
12	6·42	10·48	0·00	0·00
13	5·96	9·80	0·00	0·00
14	5·57	9·20	13·04	0·00
15	5·22	8·67	12·37	0·00
16	4·92	8·19	11·75	0·00
17	4·64	7·77	11·20	0·00
18	4·40	7·38	10·69	0·00
19	4·18	7·03	10·22	0·00
20	3·98	6·71	9·79	12·91
21	3·80	6·42	9·39	12·44
22	3·64	6·15	9·03	12·00
23	3·49	5·91	8·69	11·59
24	0·00	5·68	8·37	11·21
25	0·00	5·47	8·08	10·84
26	0·00	5·28	7·81	10·50
27	0·00	5·09	7·55	10·18
28	0·00	4·92	7·31	9·88
29	0·00	4·77	7·08	9·59
30	0·00	4·62	6·87	9·32
35	0·00	3·99	5·97	8·16
40	0·00	3·51	5·28	7·26

Note 1 — Zeros indicate inadmissible reinforcement percentage.
Note 2 — Bar spacings below the dividing line exceed 3*d*.

TABLE 1.3.39 FLEXURE — MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 15 N/mm²
f_y = 415 N/mm²
Thickness = 11·0 cm

Bar Spacing, cm	Bar Diameter, mm 6	8	10	12
5	15·57	0·00	0·00	0·00
6	13·42	0·00	0·00	0·00
7	11·77	0·00	0·00	0·00
8	10·48	16·67	0·00	0·00
9	9·44	15·21	0·00	0·00
10	8·59	13·97	0·00	0·00
11	7·87	12·91	0·00	0·00
12	7·27	12·00	0·00	0·00
13	6·75	11·20	15·96	0·00
14	6·30	10·50	15·06	0·00
15	5·90	9·88	14·26	0·00
16	5·55	9·33	13·53	0·00
17	5·24	8·83	12·86	0·00
18	4·97	8·39	12·26	16·22
19	4·72	7·99	11·71	15·56
20	4·49	7·62	11·21	14·95
21	4·29	7·29	10·74	14·39
22	0·00	6·98	10·32	13·86
23	0·00	6·70	9·92	13·37
24	0·00	6·44	9·55	12·91
25	0·00	6·20	9·21	12·48
26	0·00	5·97	8·90	12·07
27	0·00	5·77	8·60	11·69
28	0·00	5·57	8·32	11·34
29	0·00	5·39	8·06	11·00
30	0·00	5·22	7·82	10·68
35	0·00	4·51	6·78	9·33
40	0·00	0·00	5·99	8·28
45	0·00	0·00	5·36	7·44

Note 1 — Zeros indicate inadmissible reinforcement percentage.
Note 2 — Bar spacings below the dividing line exceed 3*d*.

TABLE 1.3.40 FLEXURE — MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 15 N/mm²
f_y = 415 N/mm²
Thickness = 12·0 cm

Bar Spacing, cm	Bar Diameter, mm 6	8	10	12
5	17·61	0·00	0·00	0·00
6	15·12	0·00	0·00	0·00
7	13·23	21·00	0·00	0·00
8	11·76	18·94	0·00	0·00
9	10·57	17·23	0·00	0·00
10	9·61	15·79	0·00	0·00
11	8·80	14·56	20·65	0·00
12	8·12	13·51	19·32	0·00
13	7·53	12·59	18·14	0·00
14	7·02	11·79	17·09	0·00
15	6·58	11·09	16·14	0·00
16	6·19	10·46	15·30	20·24
17	5·84	9·90	14·53	19·33
18	5·53	9·40	13·84	18·49
19	5·26	8·94	13·20	17·71
20	0·00	8·53	12·62	16·99
21	0·00	8·15	12·09	16·33
22	0·00	7·80	11·60	15·71
23	0·00	7·49	11·15	15·14
24	0·00	7·19	10·74	14·61
25	0·00	6·92	10·35	14·11
26	0·00	6·67	9·99	13·64
27	0·00	6·44	9·65	13·20
28	0·00	6·22	9·33	12·79
29	0·00	6·02	9·04	12·41
30	0·00	5·83	8·76	12·04
35	0·00	0·00	7·59	10·50
40	0·00	0·00	6·70	9·30
45	0·00	0·00	5·99	8·35

Note 1 — Zeros indicate inadmissible reinforcement percentage.
Note 2 — Bar spacings below the dividing line exceed 3*d*.

TABLE 1.3.41 FLEXURE — MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 15 N/mm²
f_y = 415 N/mm²
Thickness = 13·0 cm

Bar Spacing, cm	Bar Diameter, mm 6	8	10	12	16
5	19·65	0·00	0·00	0·00	0·00
6	16·82	0·00	0·00	0·00	0·00
7	14·69	23·59	0·00	0·00	0·00
8	13·03	21·21	0·00	0·00	0·00
9	11·71	19·24	0·00	0·00	0·00
10	10·63	17·60	24·99	0·00	0·00
11	9·73	16·21	23·23	0·00	0·00
12	8·97	15·02	21·68	0·00	0·00
13	8·32	13·99	20·32	0·00	0·00
14	7·75	13·09	19·11	0·00	0·00
15	7·26	12·30	18·03	23·95	0.00
16	6·83	11·59	17·07	22·79	0.00
17	6·44	10·97	16·20	21·73	0.00
18	6·10	10·40	15·41	20·75	0.00
19	0·00	9·90	14·69	19·86	0.00
20	0·00	9·43	14·04	19·04	0·00
21	0·00	9·01	13·44	18·27	0·00
22	0·00	8·63	12·89	17·57	0·00
23	0·00	8·28	12·39	16·92	0·00
24	0·00	7·95	11·92	16·31	0·00
25	0·00	7·65	11·48	15·74	0·00
26	0·00	7·37	11·08	15·21	0·00
27	0·00	7·11	10·70	14·72	22·92
28	0·00	6·87	10·35	14·25	22·30
29	0·00	6·64	10·01	13·81	21·70
30	0·00	6·43	9·70	13·40	21·13
35	0·00	0·00	8·40	11·66	18·66
40	0·00	0·00	7·41	10·32	16·69
45	0·00	0·00	6·62	9·25	15·09

Note 1 — Zeros indicate inadmissible reinforcement percentage.
Note 2 — Bar spacings below the dividing line exceed 3*d*.

TABLE 1.3.42 FLEXURE — MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 15 N/mm²
f_y = 415 N/mm²
Thickness = 14·0 cm

Bar Spacing, cm	Bar Diameter, mm 6	8	10	12	16
5	21·69	0·00	0·00	0·00	0·00
6	18·52	29·54	0·00	0·00	0·00
7	16·15	26·18	0·00	0·00	0·00
8	14·31	23·48	0·00	0·00	0·00
9	12·84	21·26	0·00	0·00	0·00
10	11·65	19·41	27·82	0·00	0·00
11	10·65	17·86	25·80	0·00	0·00
12	9·82	16·53	24·05	0·00	0·00
13	9·10	15·38	22·50	0·00	0·00
14	8·48	14·38	21·14	28·14	0·00
15	7·94	13·51	19·92	26·68	0·00
16	7·47	12·73	18·84	25·34	0·00
17	0·00	12·03	17·87	24·13	0·00
18	0·00	11·41	16·99	23·02	0·00
19	0·00	10·85	16·19	22·01	0·00
20	0·00	10·34	15·46	21·08	0·00
21	0·00	9·88	14·79	20·22	0·00
22	0·00	9·45	14·18	19·43	0·00
23	0·00	9·06	13·62	18·69	0·00
24	0·00	8·71	13·10	18·01	0·00
25	0·00	8·37	12·61	17·37	27·18
26	0·00	8·07	12·17	16·78	26·37
27	0·00	7·79	11·75	16·23	25·61
28	0·00	7·52	11·36	15·71	24·89
29	0·00	7·27	10·99	15·22	24·20
30	0·00	0·00	10·65	14·76	23·55
35	0·00	0·00	9·21	12·83	20·74
40	0·00	0·00	8·12	11·34	18·51
45	0·00	0·00	7·25	10·16	16·70

Note 1 — Zeros indicate inadmissible reinforcement percentage.
Note 2 — Bar spacings below the dividing line exceed 3*d*.

TABLE 1.3.43 FLEXURE — MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 15 N/mm²
f_y = 415 N/mm²
Thickness = 15·0 cm

Bar Spacing, cm	Bar Diameter, mm: 6	8	10	12	16	18
5	23·73	0·00	0·00	0·00	0·00	0·00
6	20·22	32·56	0·00	0·00	0·00	0·00
7	17·60	28·77	0·00	0·00	0·00	0·00
8	15·58	25·74	0·00	0·00	0·00	0·00
9	13·97	23·27	33·30	0·00	0·00	0·00
10	12·67	21·23	30·66	0·00	0·00	0·00
11	11·58	19·51	28·38	0·00	0·00	0·00
12	10·67	18·04	26·41	0·00	0·00	0·00
13	9·89	16·78	24·68	32·91	0·00	0·00
14	9·21	15·68	23·16	31·06	0·00	0·00
15	8·62	14·72	21·81	29·40	0·00	0·00
16	0·00	13·86	20·61	27·89	0·00	0·00
17	0·00	13·10	19·53	26·53	0·00	0·00
18	0·00	12·42	18·56	25·29	0·00	0·00
19	0·00	11·80	17·68	24·16	0·00	0·00
20	0·00	11·25	16·88	23·12	0·00	0·00
21	0·00	10·74	16·14	22·16	0·00	0·00
22	0·00	10·28	15·47	21·28	0·00	0·00
23	0·00	9·85	14·85	20·47	32·08	0·00
24	0·00	9·46	14·28	19·71	31·05	0·00
25	0·00	9·10	13·75	19·01	30·08	0·00
26	0·00	8·76	13·26	18·35	29·16	0·00
27	0·00	8·45	12·80	17·74	28·30	0·00
28	0·00	0·00	12·37	17·17	27·48	0·00
29	0·00	0·00	11·97	16·63	26·70	31·22
30	0·00	0·00	11·59	16·12	25·97	30·43
35	0·00	0·00	10·02	14·00	22·81	26·97
40	0·00	0·00	8·82	12·36	20·32	24·18
45	0·00	0·00	0·00	11·07	18·31	21·89

NOTE — Zeros indicate inadmissible reinforcement percentage.

TABLE 1.3.44 FLEXURE — MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 20 N/mm²
f_y = 415 N/mm²
Thickness = 10·0 cm

Bar Spacing, cm	Bar Diameter, mm: 6	8	10	12
5	14·33	0·00	0·00	0·00
6	12·27	0·00	0·00	0·00
7	10·72	17·11	0·00	0·00
8	9·52	15·40	0·00	0·00
9	8·55	13·98	0·00	0·00
10	7·77	12·79	0·00	0·00
11	7·11	11·79	16·78	0·00
12	6·55	10·92	15·67	0·00
13	6·08	10·18	14·70	0·00
14	5·67	9·52	13·83	0·00
15	5·31	8·95	13·05	17·32
16	4·99	8·44	12·36	16·39
17	4·71	7·99	11·73	15·64
18	4·46	7·58	11·16	14·94
19	4·24	7·21	10·65	14·30

Bar Spacing, cm	Bar Diameter, mm: 6	8	10	12
20	4·03	6·87	10·16	13·72
21	3·85	6·57	9·74	13·17
22	3·68	6·29	9·35	12·67
23	3·52	6·03	8·98	12·20
24	0·00	5·79	8·64	11·76
25	0·00	5·57	8·33	11·36
26	0·00	5·37	8·03	10·98
27	0·00	5·18	7·76	10·63
28	0·00	5·00	7·51	10·29
29	0·00	4·84	7·27	9·97
30	0·00	4·69	7·04	9·68
35	0·00	4·04	6·10	8·43
40	0·00	3·55	5·38	7·46

NOTE 1 — Zeros indicate inadmissible reinforcement percentage.
NOTE 2 — Bar spacings below the dividing line exceed 3d.

TABLE 1.3.45 FLEXURE — MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 20 N/mm²
f_y = 415 N/mm²
Thickness = 11·0 cm

Bar Spacing, cm	Bar Diameter, mm: 6	8	10	12
5	16·37	0·00	0·00	0·00
6	13·97	22·23	0·00	0·00
7	12·18	19·70	0·00	0·00
8	10·79	17·66	0·00	0·00
9	9·6[illegible]	15·99	0·00	0·00
10	8·79	14·61	20·37	0·00
11	8·04	13·44	19·35	0·00
12	7·40	12·44	18·03	0·00
13	6·86	11·57	16·88	0·00
14	6·40	10·82	15·85	21·04
15	5·99	10·16	14·94	19·94
16	5·63	9·57	14·13	18·94
17	5·31	9·05	13·40	18·04
18	5·03	8·5[illegible]	12·74	17·21
19	4·77	8·16	12·14	16·45

Bar Spacing, cm	Bar Diameter, mm: 6	8	10	12
20	4·54	7·78	11·59	15·76
21	4·33	7·43	11·09	15·11
22	0·00	7·11	10·64	14·52
33	0·00	6·82	10·21	13·97
24	0·00	6·55	9·82	13·46
25	0·00	6·30	9·46	12·99
26	0·00	6·07	9·12	12·55
27	0·00	5·85	8·81	12·13
28	0·00	5·65	8·52	11·75
29	0·00	5·47	8·24	11·38
30	0·00	5·29	7·99	11·04
35	0·00	4·56	6·91	9·59
40	0·00	0·00	6·09	8·48
45	0·00	0·00	5·44	7·60

NOTE 1 — Zeros indicate inadmissible reinforcement percentage.
NOTE 2 — Bar spacings below the dividing line exceed 3d.

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TABLE 1.3.46 FLEXURE — MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 20 N/mm²
f_y = 415 N/mm²
Thickness = 12·0 cm

Bar Spacing, cm	Bar Diameter, mm: 6	8	10	12
5	18·41	0·00	0·00	0·00
6	15·67	25·25	0·00	0·00
7	13·64	22·29	0·00	0·00
8	12·07	19·93	0·00	0·00
9	10·32	18·01	25·76	0·00
10	9·81	16·42	23·70	0·00
11	8·96	15·08	21·93	0·00
12	8·26	13·95	20·40	26·99
13	7·65	12·97	19·06	25·39
14	7·13	12·12	17·88	23·95
15	6·67	11·37	16·83	22·66
16	6·27	10·71	15·90	21·49
17	5·91	10·12	15·07	20·44
18	5·60	9·59	14·31	19·48
19	5·31	9·12	13·63	18·60

Bar Spacing, cm	Bar Diameter, mm: 6	8	10	12
20	0·00	8·69	13·01	17·80
21	0·00	8·29	12·44	17·06
22	0·00	7·93	11·92	16·38
23	0·00	7·61	11·45	15·75
24	0·00	7·30	11·00	15·16
25	0·00	7·02	10·59	14·62
26	0·00	6·77	10·21	14·12
27	0·00	6·52	9·86	13·64
28	0·00	6·30	9·53	13·20
29	0·00	6·09	9·22	12·79
30	0·00	5·90	8·93	12·40
35	0·00	0·00	7·72	10·76
40	0·00	0·00	6·80	9·50
45	0·00	0·00	6·07	8·50

NOTE 1 — Zeros indicate inadmissible reinforcement percentage.
NOTE 2 — Bar spacings below the dividing line exceed 3*d*.

TABLE 1.3.47 FLEXURE — MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 20 N/mm²
f_y = 415 N/mm²
Thickness = 13·0 cm

Bar Spacing, cm	Bar Diameter, mm: 6	8	10	12	16
5	20·45	32·66	0·00	0·00	0·00
6	17·38	28·28	0·00	0·00	0·00
7	15·10	24·88	0·00	0·00	0·00
8	13·34	22·20	31·72	0·00	0·00
9	11·95	20·02	28·91	0·00	0·00
10	10·83	18·23	26·54	0·00	0·00
11	9·89	16·73	24·51	32·49	0·00
12	9·11	15·46	22·76	30·39	0·00
13	8·43	14·36	21·24	28·53	0·00
14	7·86	13·41	19·90	26·87	0·00
15	7·35	12·58	18·72	25·38	0·00
16	6·91	11·84	17·67	24·04	0·00
17	6·51	11·19	16·73	22·84	0·00
18	6·16	10·60	15·89	21·74	0·00
19	0·00	10·07	15·12	20·75	0·00

Bar Spacing, cm	Bar Diameter, mm: 6	8	10	12	16
20	0·00	9·59	14·43	19·84	30·85
21	0·00	9·16	13·79	19·00	29·73
22	0·00	8·76	13·21	18·23	28·67
23	0·00	8·39	12·68	17·52	27·69
24	0·00	8·06	12·18	16·87	26·77
25	0·00	7·75	11·73	16·25	25·90
26	0·00	7·46	11·30	15·69	25·08
27	0·00	7·20	10·91	15·16	24·31
28	0·00	6·95	10·54	14·66	23·59
29	0·00	6·72	10·20	14·20	22·90
30	0·00	6·50	9·88	13·76	22·26
35	0·00	0·00	8·53	11·92	19·49
40	0·00	0·00	7·50	10·52	17·33
45	0·00	0·00	6·70	9·41	15·59

NOTE 1 — Zeros indicate inadmissible reinforcement percentage.
NOTE 2 — Bar spacings below the dividing line exceed 3*d*.

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TABLE 1.3.48 FLEXURE — MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 20 N/mm²
f_y = 415 N/mm²
Thickness = 14·0 cm

Bar Spacing, cm	Bar Diameter, mm: 6	8	10	12	16
5	22·49	36·29	0·00	0·00	0·00
6	19·08	31·20	0·00	0·00	0·00
7	16·56	27·48	39·12	0·00	0·00
8	14·62	24·47	35·26	0·00	0·00
9	13·09	22·04	32·06	0·00	0·00
10	11·85	20·05	29·37	38·94	0·00
11	10·82	18·38	27·08	36·20	0·00
12	9·96	16·97	25·12	33·79	0·00
13	9·22	15·76	23·42	31·67	0·00
14	8·58	14·71	21·93	29·78	0·00
15	8·03	13·79	20·61	28·10	0·00
16	7·55	12·98	19·44	26·60	0·00
17	0·00	12·25	18·40	25·24	0·00
18	0·00	11·61	17·46	24·01	0·00
19	0·00	11·03	16·61	22·90	35·87

Bar Spacing, cm	Bar Diameter, mm: 6	8	10	12	16
20	0·00	10·50	15·85	21·88	34·48
21	0·00	10·02	15·14	20·95	33·18
22	0·00	9·58	14·50	20·09	31·97
23	0·00	9·18	13·91	19·30	30·84
24	0·00	8·82	13·37	18·57	29·79
25	0·00	8·48	12·86	17·89	28·80
26	0·00	8·16	12·39	17·26	27·87
27	0·00	7·87	11·96	16·67	27·00
28	0·00	7·60	11·55	16·12	26·18
29	0·00	7·34	11·18	15·60	25·41
30	0·00	0·00	10·82	15·12	24·68
35	0·00	0·00	9·34	13·09	21·56
40	0·00	0·00	8·21	11·54	19·14
45	0·00	0·00	7·33	10·32	17·20

NOTE 1 — Zeros indicate inadmissible reinforcement percentage.
NOTE 2 — Bar spacings below the dividing line exceed 3*d*.

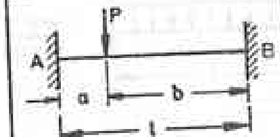
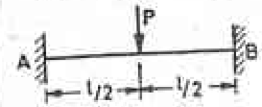
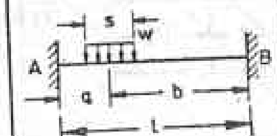
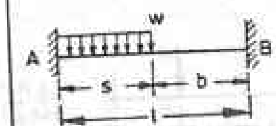
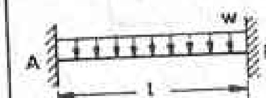
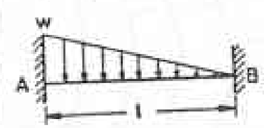
TABLE 1.3.49 FLEXURE—MOMENT OF RESISTANCE OF SLABS, kN.m PER METRE WIDTH

f_{ck} = 20 N/mm²
f_y = 415 N/mm²
Thickness = 15·0 cm

BAR SPACING, cm	BAR DIAMETER, mm 6	8	10	12	16	18
5	24·53	39·92	0·00	0·00	0·00	0·00
6	20·78	34·32	0·00	0·00	0·00	0·00
7	18·01	30·07	43·16	0·00	0·00	0·00
8	15·90	26·73	38·80	0·00	0·00	0·00
9	14·22	24·06	35·21	0·00	0·00	0·00
10	12·87	21·86	32·20	43·03	0·00	0·00
11	11·75	20·03	29·66	39·91	0·00	0·00
12	10·81	18·48	27·48	37·19	0·00	0·00
13	10·00	17·15	25·60	34·80	0·00	0·00
14	9·31	16·00	23·95	32·70	0·00	0·00
15	8·71	15·00	22·50	30·82	0·00	0·00
16	0·00	14·11	21·22	29·15	0·00	0·00
17	0·00	13·32	20·07	27·64	43·25	0·00
18	0·00	12·61	19·04	26·28	41·40	0·00
19	0·00	11·98	18·11	25·04	39·69	0·00
20	0·00	11·41	17·26	23·92	38·11	0·00
21	0·00	10·88	16·49	22·89	36·64	0·00
22	0·00	10·41	15·79	21·94	35·27	41·27
23	0·00	9·97	15·14	21·07	34·00	39·90
24	0·00	9·57	14·55	20·27	32·81	38·60
25	0·00	9·20	14·00	19·52	31·70	37·38
26	0·00	8·86	13·48	18·83	30·66	36·24
27	0·00	8·54	13·01	18·18	29·69	35·15
28	0·00	0·00	12·57	17·58	28·77	34·13
29	0·00	0·00	12·15	17·01	27·91	33·16
30	0·00	0·00	11·77	16·48	27·09	32·24
35	0·00	0·00	10·15	14·26	23·64	28·29
40	0·00	0·00	8·92	12·56	20·95	25·19
45	0·00	0·00	0·00	11·22	18·81	22·69

NOTE 1 — Zeros indicate inadmissible reinforcement percentage.
NOTE 2 — Bar spacings below the dividing line exceed $3d$.

TABLE 1.3.50 FIXED END MOMENTS FOR PRISMATIC BEAMS

LOAD TYPE	M_{FA}	M_{FB}
	$+\frac{Pab^2}{l^2}$	$-\frac{Pa^2b}{l^2}$
	$+\frac{Pl}{8}$	$-\frac{Pl}{8}$
	$+\frac{ws}{12l^2}[12ab^2+s^2(l-3b)]$	$-\frac{ws}{12l^2}[12a^2b+s^2(l-3a)]$
	$+\frac{ws}{12l^2}[2l(3l-4s)+3s^2]$	$-\frac{ws^3}{12l^2}(4l-3s)$
	$+\frac{wl^2}{12}$	$-\frac{wl^2}{12}$
	$+\frac{5wl^2}{96}$	$-\frac{5wl^2}{96}$
	$+\frac{wl^2}{20}$	$-\frac{wl^2}{30}$
	$-M\frac{b}{l}(2-\frac{3b}{l})$	$-M\frac{a}{l}(2-\frac{3a}{l})$

Note :- w is the load per unit length

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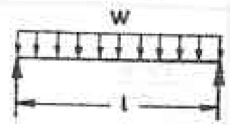
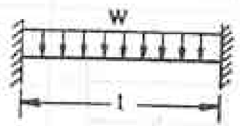
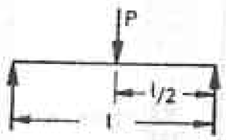
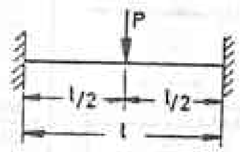
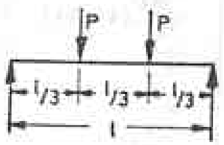
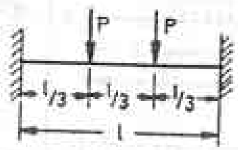
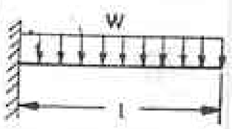
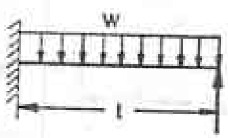
TABLE 1.3.51 DEFLECTION FORMULAE FOR PRISMATIC BEAMS

	$\frac{5}{384} \times \frac{Wl^3}{EI}$		$\frac{Wl^3}{384\,EI}$
	$\frac{Pl^3}{48\,EI}$		$\frac{Pl^3}{192\,EI}$
	$\frac{23\,Pl^3}{648\,EI}$		$\frac{5\,Pl^3}{648\,EI}$
	$\frac{Wl^3}{8\,EI}$		$\frac{Wl^3}{185\,EI}$
	$\frac{Pl^3}{3\,EI}$		$\frac{1}{16} \times \frac{Ml^2}{EI}$

Note:- W is total distributed load

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Chart 1 **COMPRESSION WITH BENDING** — Rectangular Section — Reinforcement Distributed Equally on Two Sides

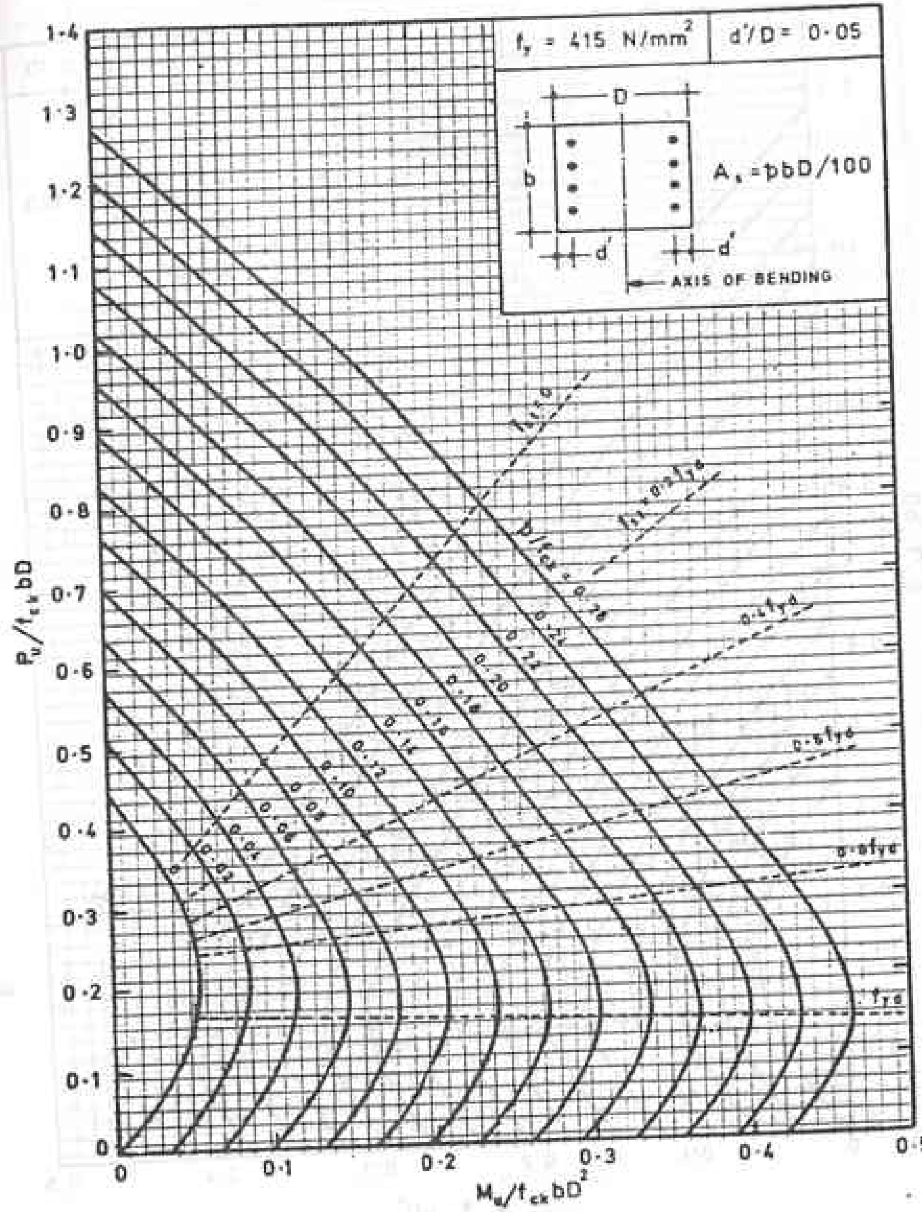

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Chart 2 **COMPRESSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Two Sides**

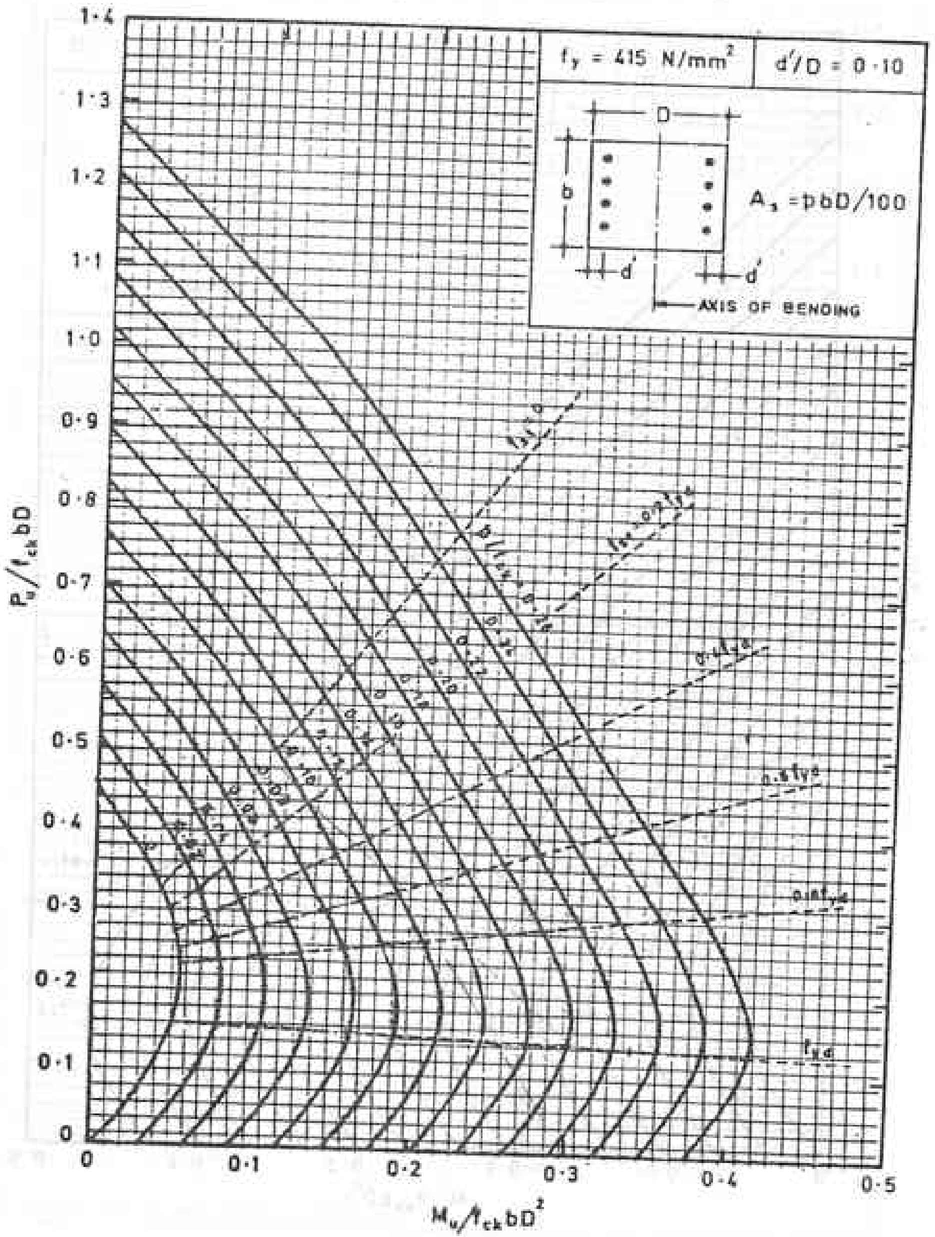

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Chart 3 **COMPRESSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Two Sides**

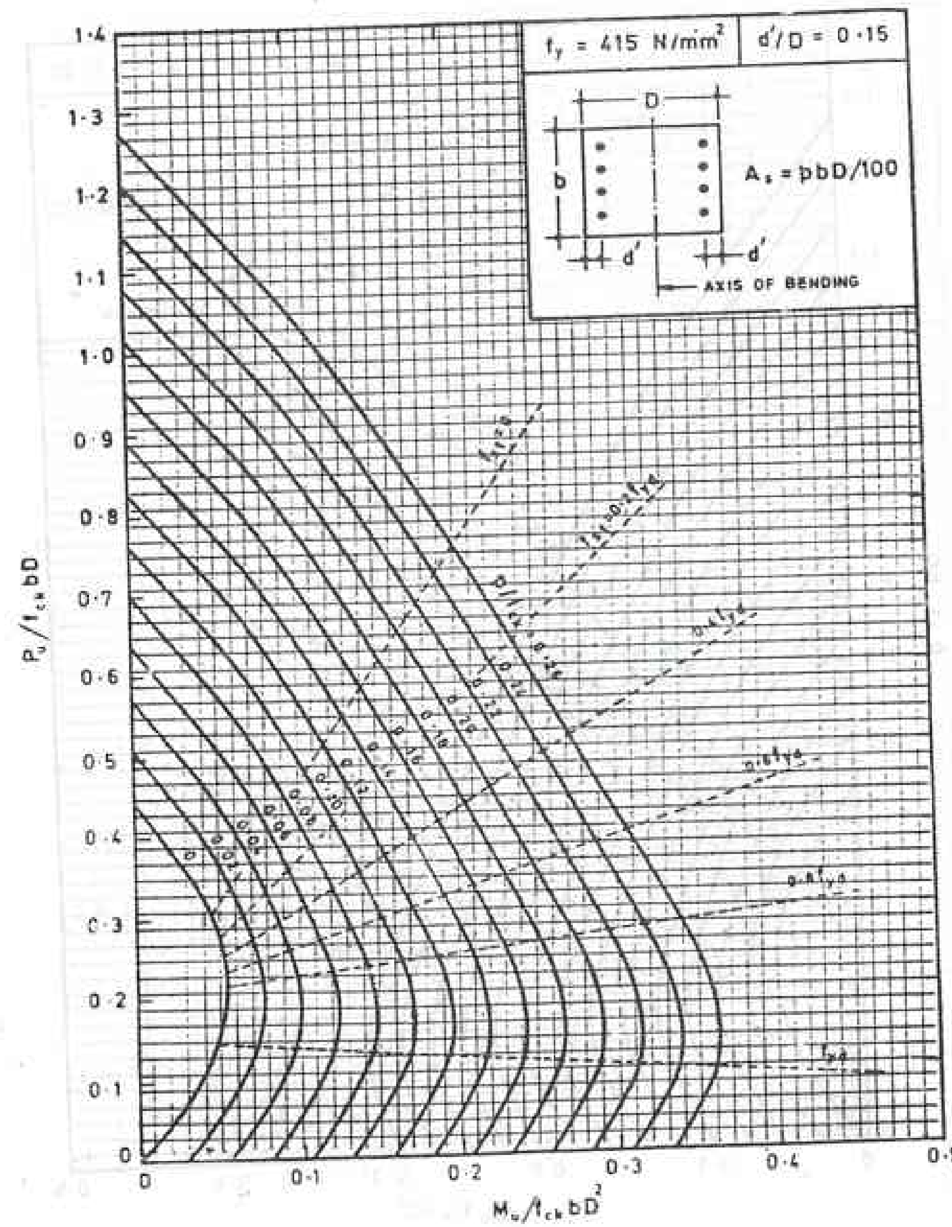

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Chart 4 **COMPRESSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Two Sides**

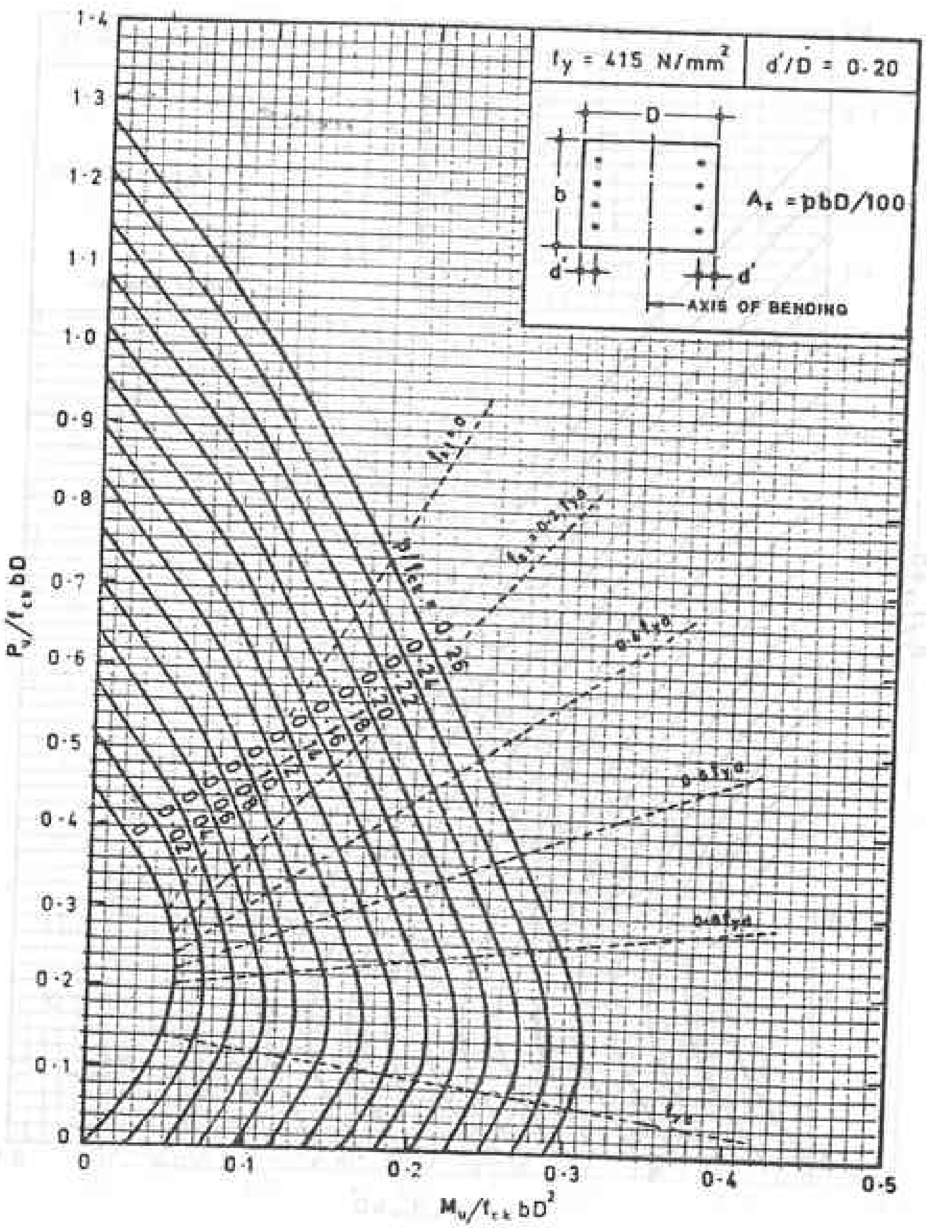

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Chart 5 **COMPRESSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Four Sides**

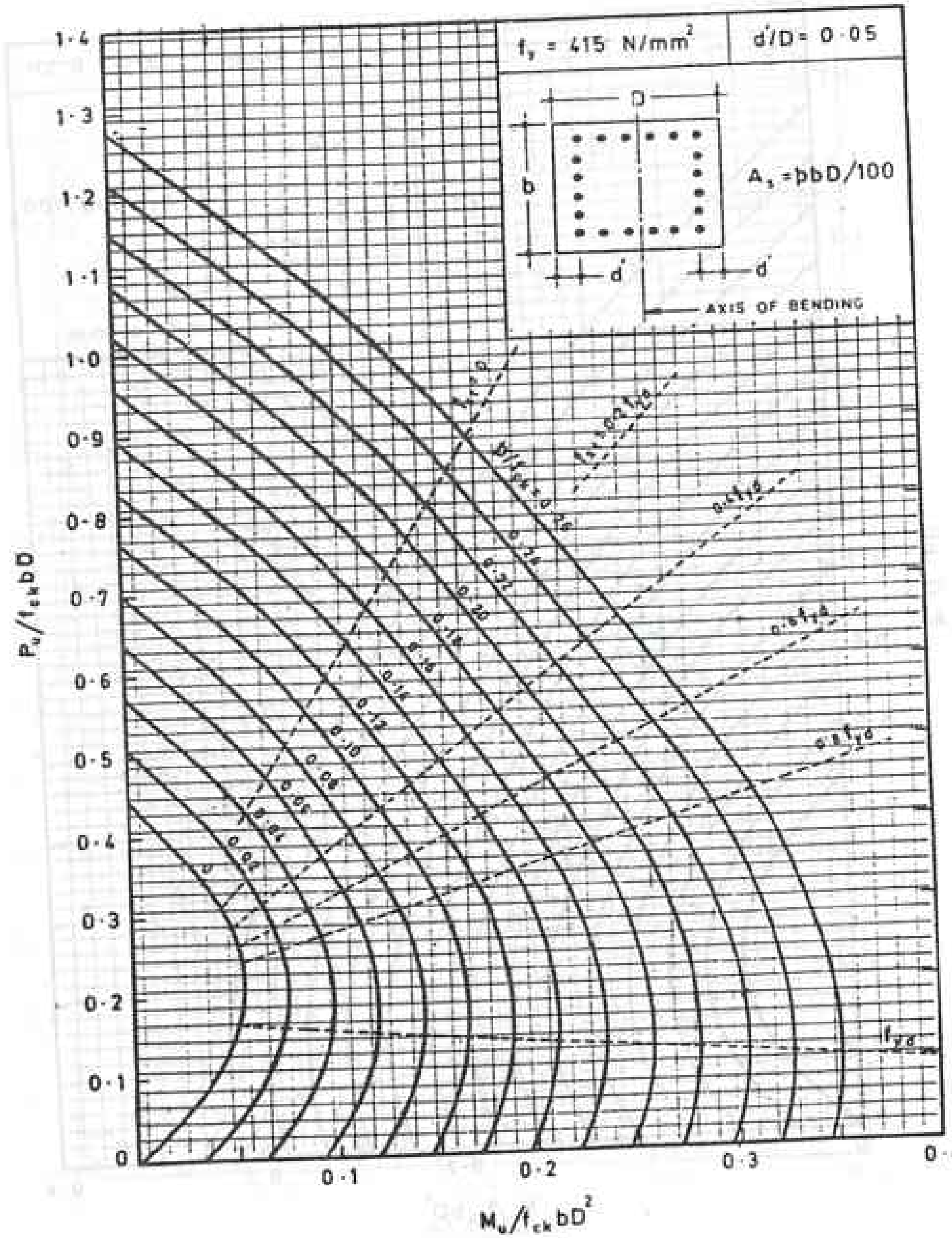

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Chart 6 **COMPRESSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Four Sides**

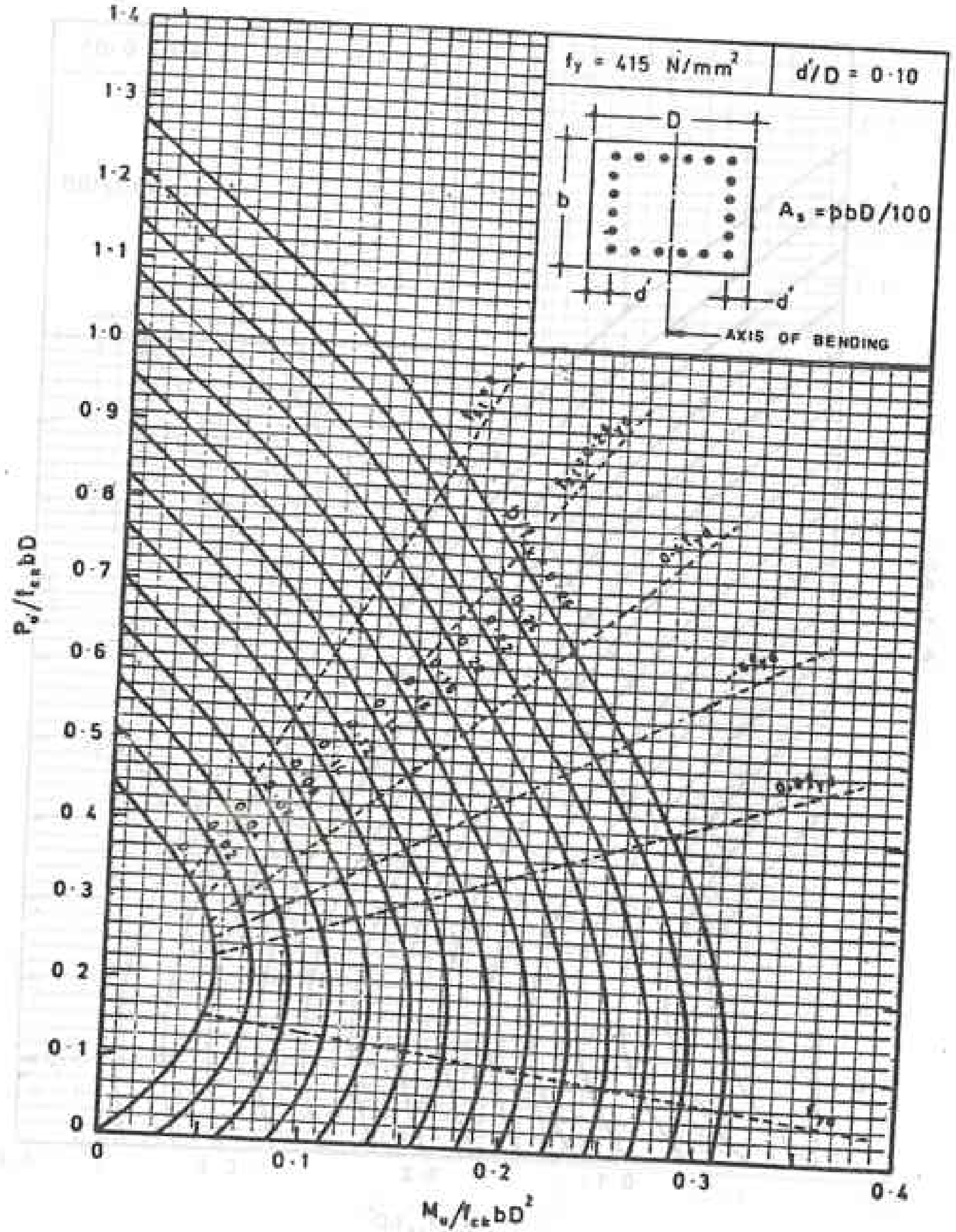

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Chart 7 **COMPRESSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Four Sides**

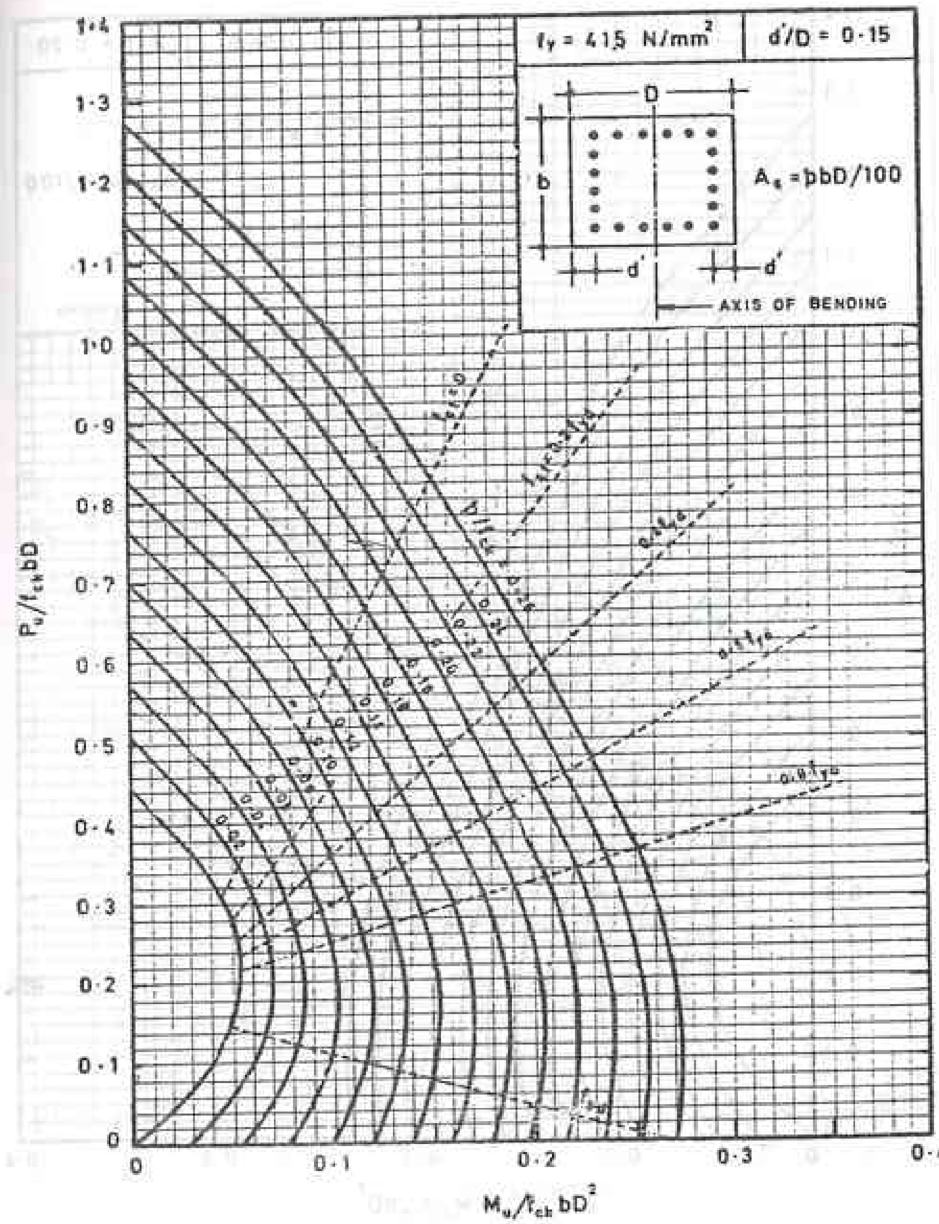

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Chart 8 **COMPRESSION WITH BENDING — Rectangular Section — Reinforcement Distributed Equally on Four Sides**

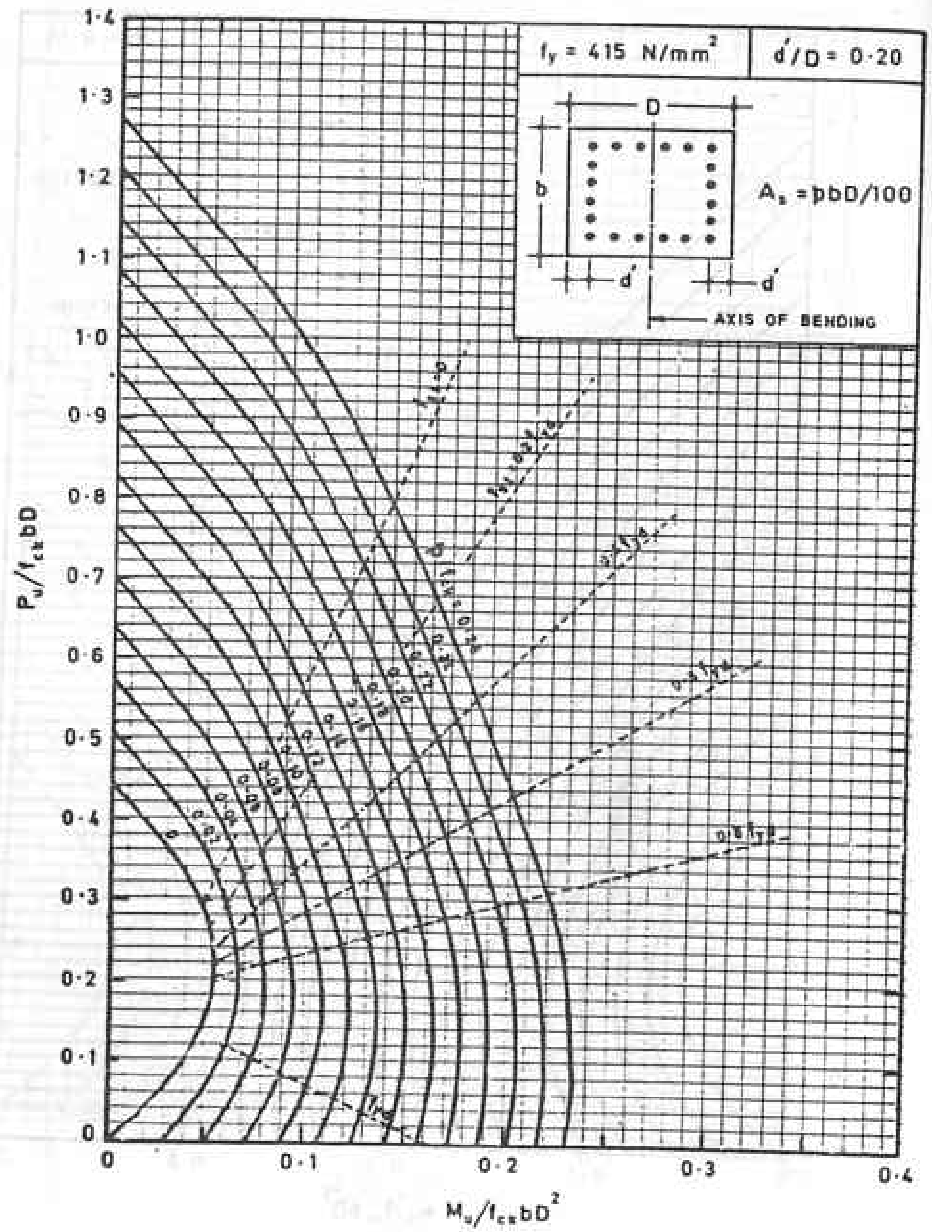

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Chart 9 **COMPRESSION WITH BENDING — Circular Section**

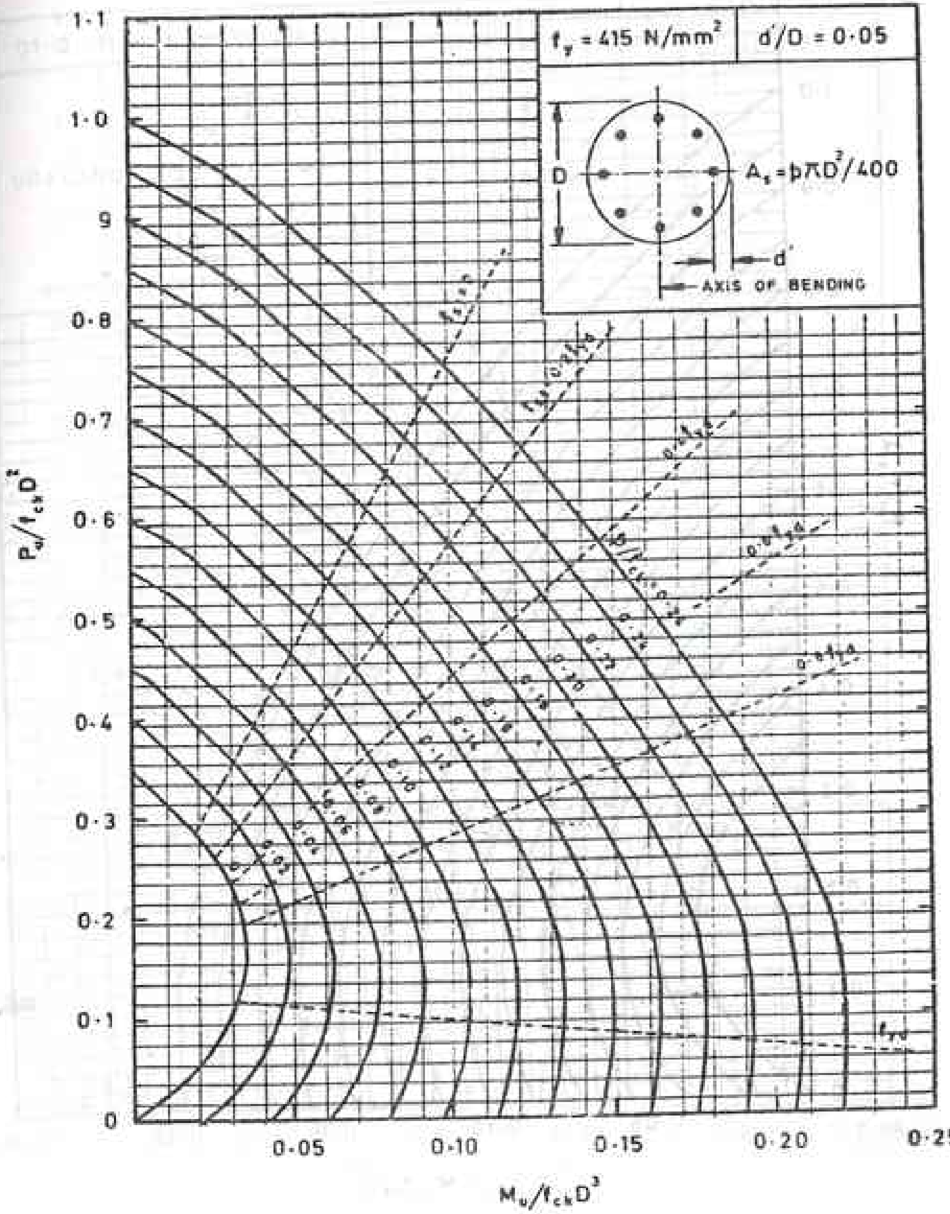

Chart 10 **COMPRESSION WITH BENDING – Circular Section**

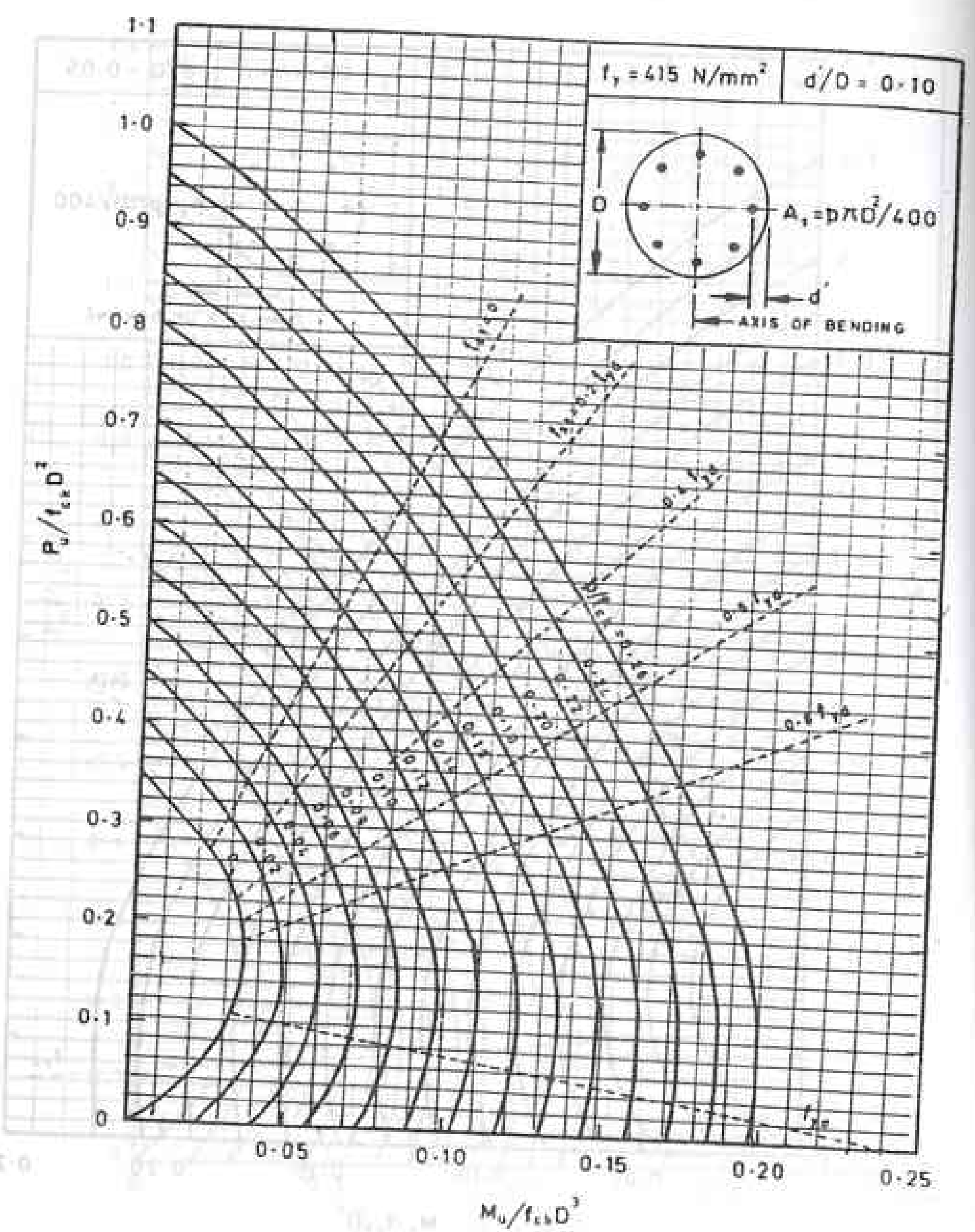

Chart 11 **COMPRESSION WITH BENDING – Circular Section**

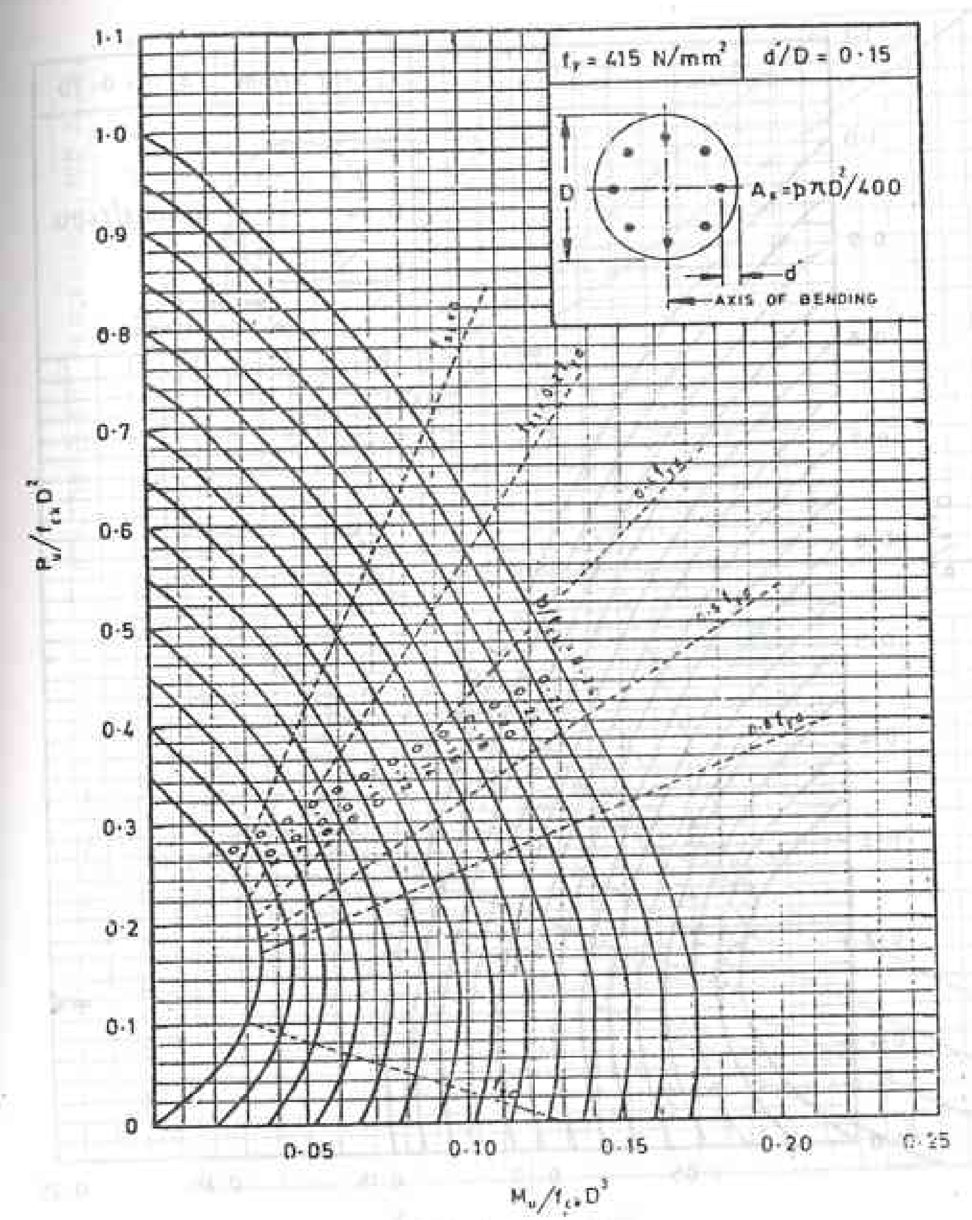

Chart 12 COMPRESSION WITH BENDING — Circular Section

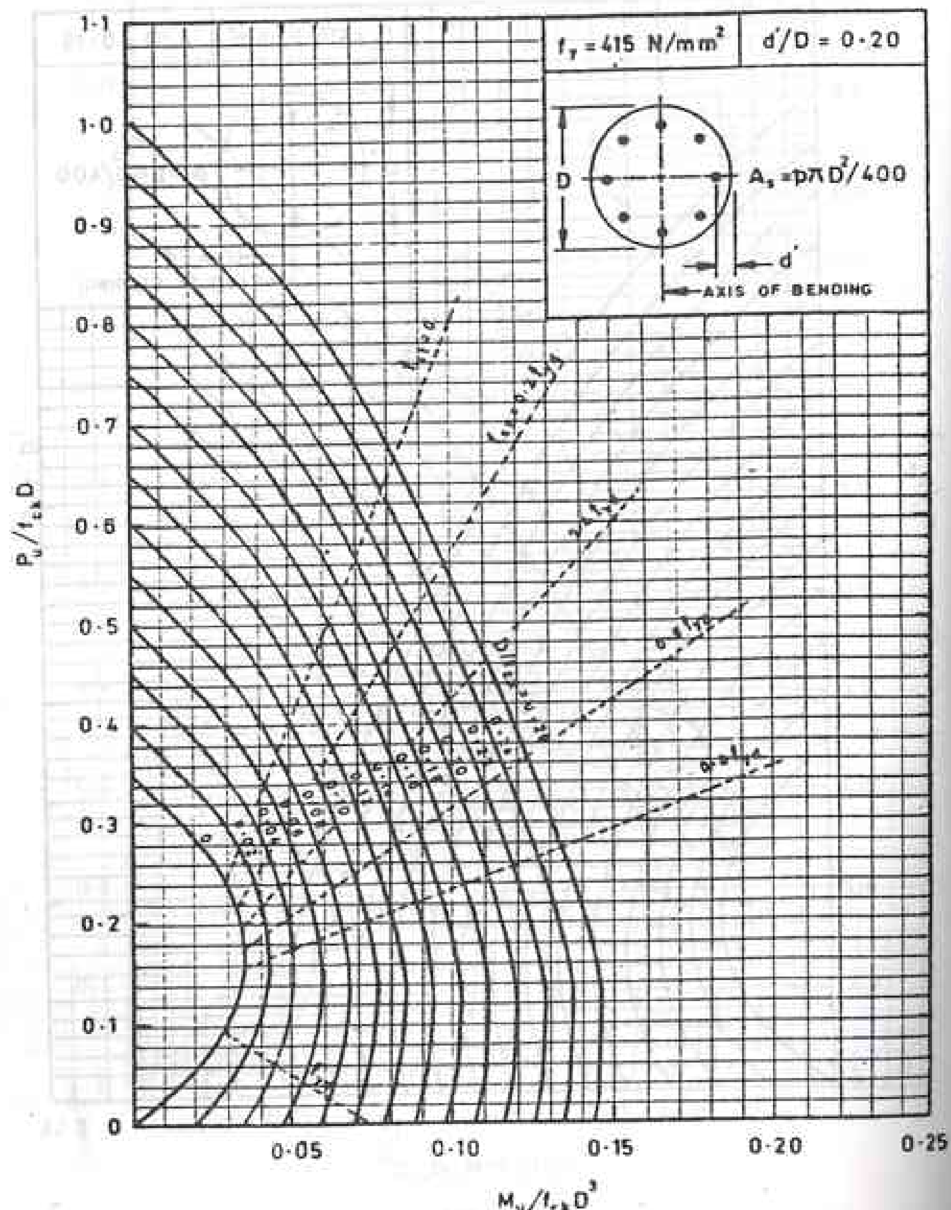

Chart 13 VALUES OF P_{uz} for COMPRESSION MEMBERS

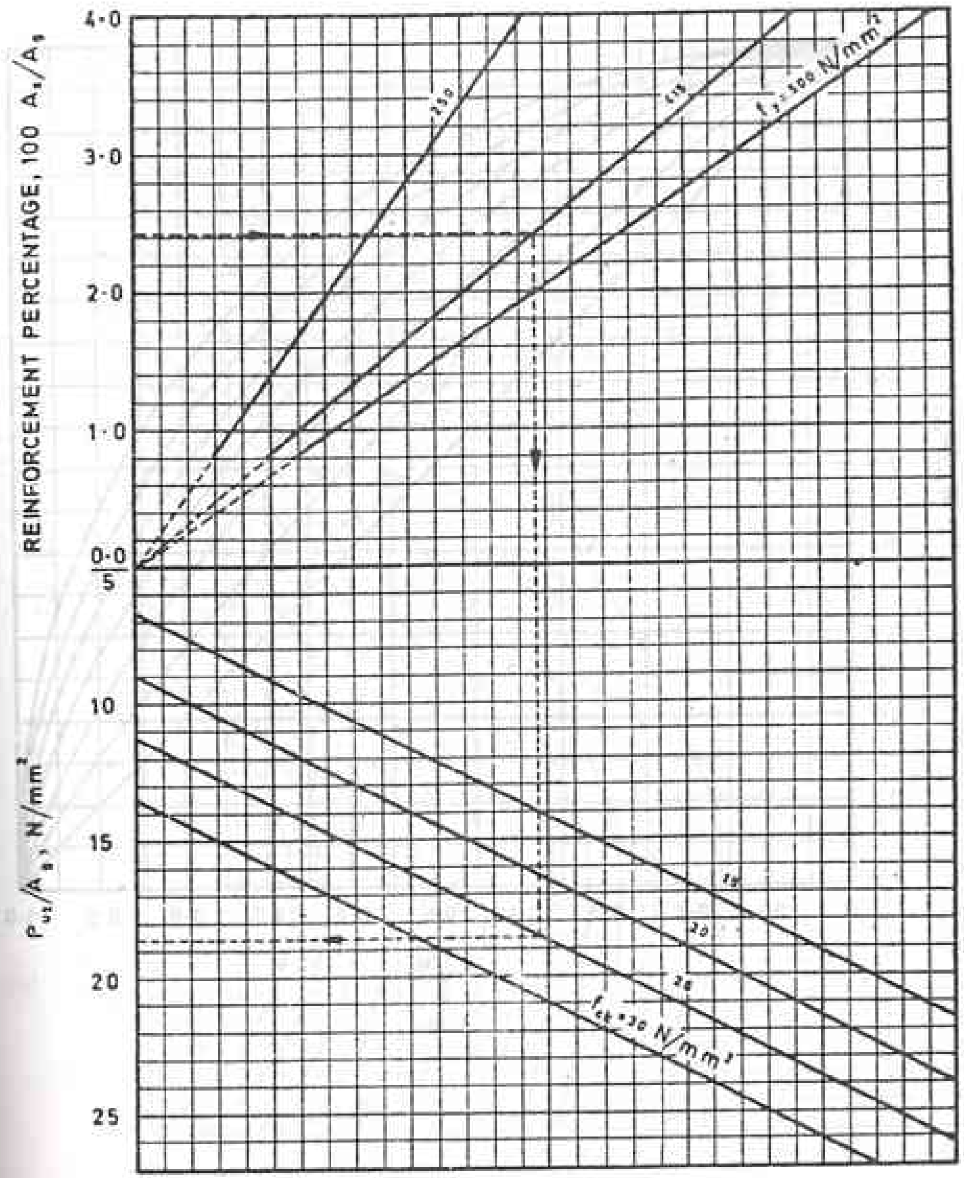

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Chart 14 BIAXIAL BENDING IN COMPRESSION MEMBERS

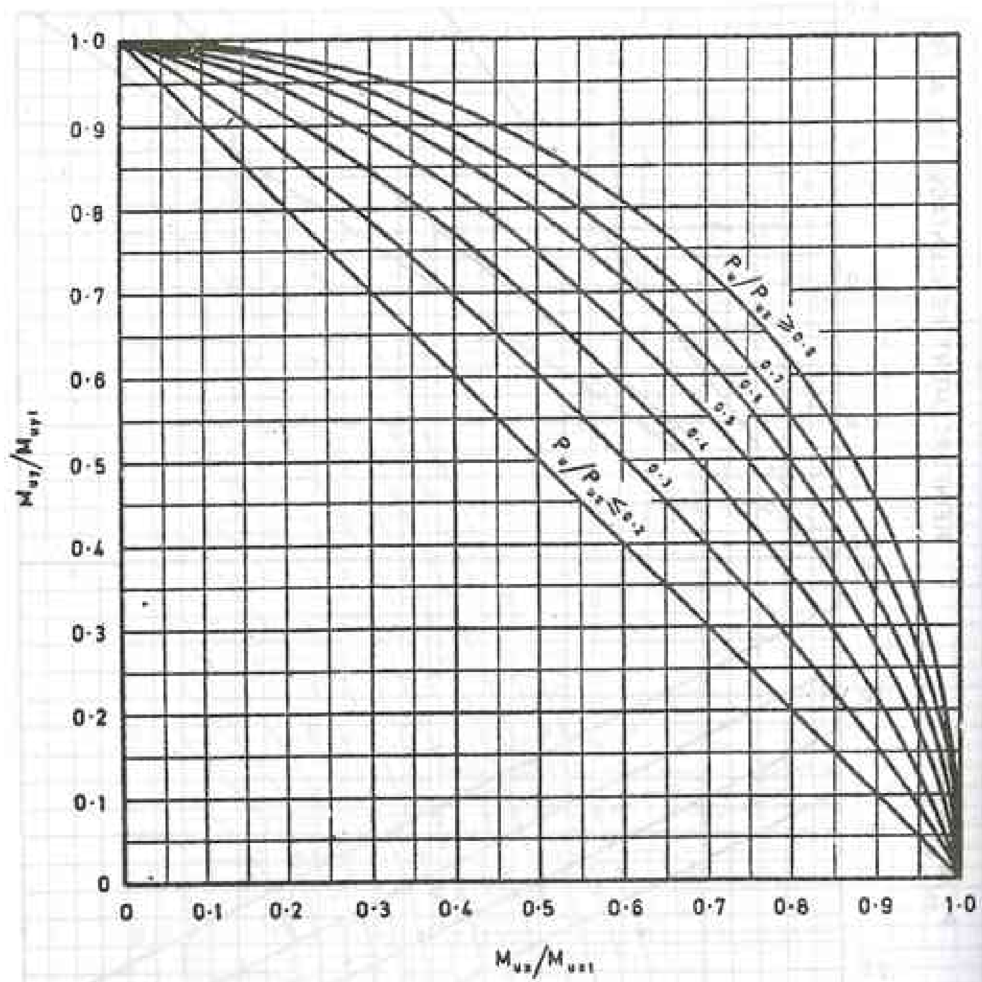

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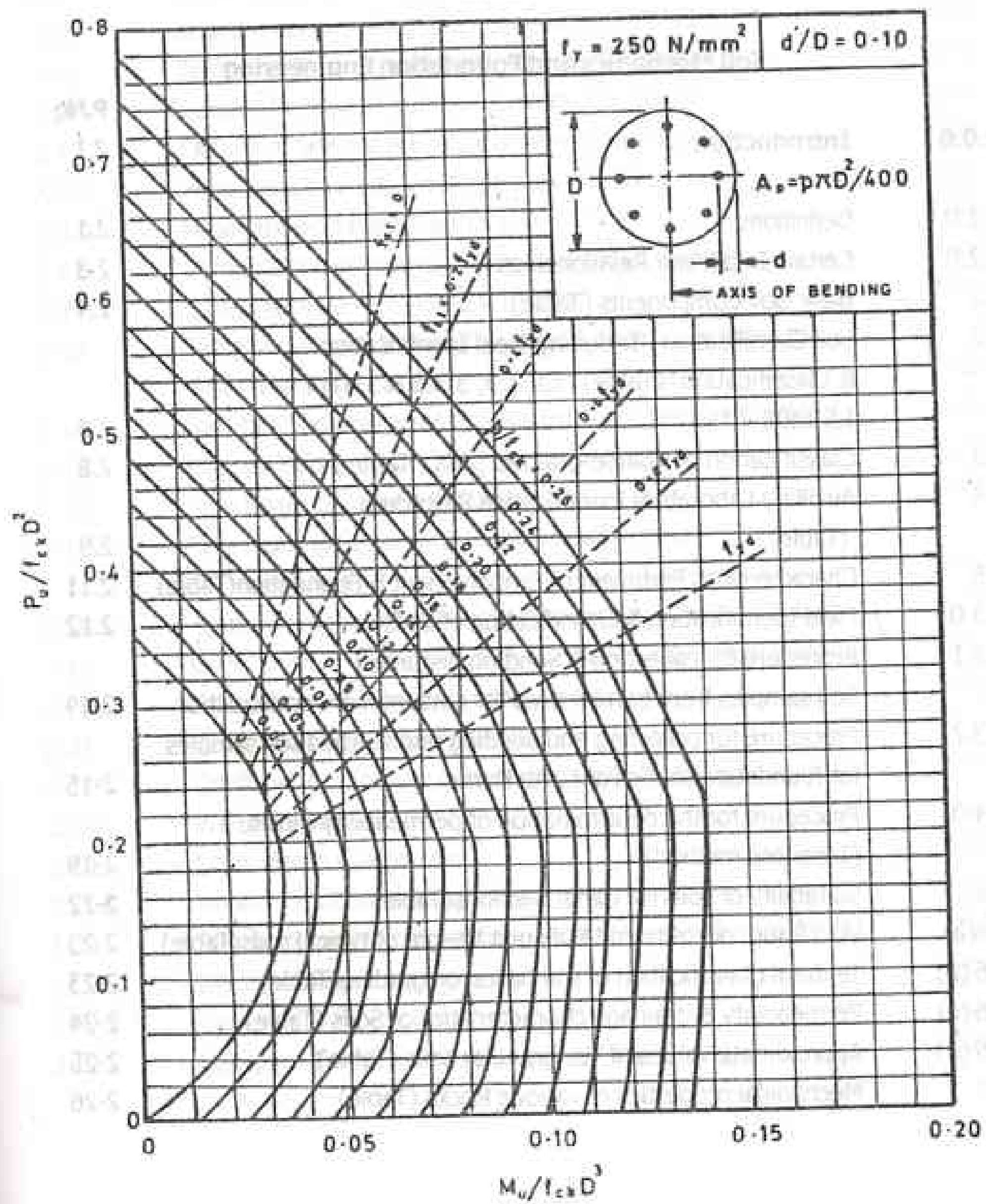
Chart 15 COMPRESSION WITH BENDING — Circular Section



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Chapter II

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Soil Mechanics and Foundation Engineering

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SECTION II

SOIL MECHANICS AND FOUNDATION ENGINEERING

2.0.0 Introduction : Soil is a complex material formed of the weathering process in solid rocks containing solid particles, water, air and other gases including organic matter. Its properties cannot be predicted with the same accuracy as other building materials due to its heterogeneous nature and varying characteristics. Besides theoretical concepts, it is essential to explore the soils, test them well before adequate design and construction controls are exercised for best results.

Boulders, gravels, sands clays and silts extending upto 75 microns in size are termed soils. Formation of soils is the result of geologic cycle continuously taking place, the cycle being weathering or denudation, transportation, deposition and upheaval etc., Weathering by physical agencies such as periodical temperature changes, impact and splitting action of flowing water or ice, wind, plants and animals etc., results in formation of cohesionless soils and residual soils of shallow depth formed at the parent rock, chemical weathering may be due to oxidation, hydrotion,carbonation and leaching by organic acids and water. Transported soils may be homogeneous or heterogeneous depending on the manner of transportation and deposition and will be of considerable depth; agencies of transport being water, ice, wind and gravity etc.,

Alluvial, marine or locustrine soils are of water transport. Glacial drift or simply drift is due to flow of molten ice and the components being of various kinds. Glacial streams contribute to stratified depths. Loess or aeoline is due to wind blown dunned sands or silty clays with no stratifications. Soils transported by gravitation forces are termed colluvial soils such as Talus (below pick up weirs).

2.1.0 Definitions: Soil mass consists of solid particles, water and air, the voids being occupied partly both by water and air in a saturated soil. In a fully saturated soil only water occupies the voids. In a dry soil mass the voids are filled with air only. Moisture content shall be expressed as w generally as a percentage and equal to

$$w = \frac{w - w_d}{w_d} \times 100$$

Where w = total weight of soil sample in grams
w_d = Dry weight of soil solids in the sample (oven dry)

<u>Bulk Density</u> γ is the total weight ω per unit of its total volume including water (and air).

Then $\gamma = w / V$.

<u>Dry Density</u> γ_d is the unit dry weight of soil solids per unit volume before drying.

$\gamma_d = w_d / V$.

<u>Saturated Density</u> γ_{sat} is the bulk Density of soil in its moist state.

<u>Submerged Density</u> γ buoyant (γ') = $\gamma_{Sat} - \gamma_w$

<u>Specific gravity</u>: G is the ratio of the weight of a given volume of soil solids in air at a given temperature (27° C) to the weight of an equivalent volume of water (distilled) and

$G = \gamma_s / \gamma_\omega$

<u>Voids ratio</u>:- e is the ratio of volume of voids to the volume of soil solids in a sample and
$e = V_v / V_s$.

<u>Porosity</u> n is the ratio of volume of voids to the total volume of a given soil mass and
$n = V_v / V$ generally expressed as a percentage.

Also $n = V_v / V = e / 1+e$ where $e = V_v / V_s = n/1-n$
$n = e / (1+e) = e (1-n)$ or $(1-n) = 1/1+e$.

<u>Degree of saturation</u> S_r , is the ratio of the volume of water present in a soil mass to the total volume of voids (including air) in it.

$S_r = V_w / V_v$ and is usually expressed as a percentage.

For a fully (100%) saturated sample, $V_w = V_v$ and $S_r = 1$,
and in a perfectly dry sample, $V_w = 0$ and $S_r = 0$
(V_w and V_v can be represented by e_w and e also when $S_r = e_\omega / e$)

<u>Density Index</u> I_D = the relative density or degree of density usually indicates the relative

compactness of the natural soil Deposit and is given by

$I_D = (e_{max} - e) / (e_{max} - e_{min})$

Where e_{max} = void ratio in the loosest state of the soil
e_{min} = void ratio in the densest state of the soil
e = natural void ratio of the deposit.

It is generally used for cohesionless soils only and it varies between 0 and 1

2.2.0 Certain important relationships:

a) e W = e S_r , (for a fully saturated sample, ea = O, e W = e and $S_r = 1$)

$w = Ww / Wd = ew\text{-} \gamma_w / \gamma_s$

but $G = \gamma_s / \gamma_w$ or $\gamma_s = G \gamma_w$

Therefore,

$$w = \frac{ew \gamma_w}{G \gamma_w} = ew/G$$ or ew = water void ratio = Gw = e. Sn, or e = Gw / S_r

b) $\gamma_d = Wd / v = \gamma_s . Vs / V$ but Vs = 1, V = (i+e)

$$\text{or } e = \frac{G.\gamma_w}{\gamma_d} - 1 \text{ ;}$$ Also Vs = (1-u) and V = 1

$Yd = G. .\gamma_w$ (i-u) /1 = (1-u) $G.\gamma_w$

c)

$$\gamma_{sat} = \frac{\text{Total weight of saturated soil}}{\text{Total Volume of soil}} = \frac{W\ sat}{V} = \frac{Wd + Ww}{V}$$

$$= \frac{\gamma_s . V_s + \gamma_w . V_w}{V}\text{; but } Vs = 1,\ Vw = e,\ V = 1+e$$

$$\gamma sat = \frac{\gamma_s (1 + \gamma_w . e)}{(1+e)} = \frac{G.\gamma_w + \gamma_w . e}{(1+e)} = \frac{\gamma_w (W + e)}{1+e}$$

d) W = Ww / Wd,

$$1+w = \frac{Ww + Wd}{Wd} = W/Wd$$

Wd = W / (1+w) ; $\gamma = Wd / V = W / (1+w)v$ or $\gamma_d = \gamma / (1+w)$

$\gamma = \gamma_d (1+W)$

Table 2-1

BASIC SOIL COMPONENTS (Clause 3.3.2 of 1498-1970)

Sl. No.	Soil	Soil Component	Symbol	Particle-Size Range and Description
(1)	(2)	(3)	(4)	(5)
i)	Coarse-grained Component	Boulder	None	Rounded to angular, bulky, hard, rock particle; average diameter more than 300 mm
		Cobble	None	Rounded to angular, bulky, hard, rock particle; average diameter smaller than 300 mm but retained on 80-mm IS Sieve
		Gravel	G	Rounded to angular, bulky, hard, rock particle; passing 80-mm IS Sieve but

			retained on 4.75 mm IS Sieve Coarse : 80 mm to 20-mm IS Sieve Fine : 20-mm to 4.75-mm IS Sieve
	Sand	S	Rounded to angular, bulky, hard, rock particle; passing 4.75-mm IS Sieve but retained on 75-micron IS Sieve Coarse: 4.75-mm to 2.0-mm IS Sieve Medium:2.0-mm to 425-micron IS Sieve Fine:425-micron to 75-micron IS Sieve
ii) Fine-grained	Silt	M	Particles smaller than 75-micron IS Sieve; identified by behaviour, that is, slightly plastic or non plastic regardless of moisture and exhibits little or no strength when air dried.
	Clay	C	Particles smaller than 75-micron IS Sieve; identified by behaviour, that is, it can be made to exhibit plastic properties within a certain range of moisture and exhibits considerable strength when air dried
	Organic matter	O	Organic matter in various sizes and stages of decomposition.

Note—A comparison between the size classifications of IS:1498-1959 'Classification and identification of soils for general engineering purposes' and the present revision is shown in Appendix A.

Table 2-2 SOIL CLASSIFICATION (INCLUDING FIELD IDENTIFICATION AND DESCRIPTION)
(Clause 3.3, 3.3.3 and 3.4) IS : 1498-1970

Division	Sub-Division		Group Letter Symbol	Hatching	Mapping Colour	Typical Names	Field Identification Procedures (Excluding particles larger than 80 mm and basing fractions on estimated weights)	Information Required for Describing Soils
1	2		3	4	5	6		8
COARSE-GRAINED SOILS More than half of material is larger than 75-micron IS Sieve size The smallest particle visible to the naked eye	*Gravels* More than half of coarse fraction is larger than 4·75-mm IS Sieve size	Clean gravels (Little or no fines)	GW		Red	Well graded gravels, gravel-sand mixtures; little or no fines	Wide range in grain sizes and substantial amounts of all intermediate particle sizes	For undisturbed soils add information on stratification; degree of compactness, cementation, moisture conditions and drainage characteristics Give typical name; indicate approximate percentages of sand and gravel; maximum size, angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbol in parentheses *Example*: Silty sand, gravelly; about 20 percent hard angular gravel particles, 10 mm maximum size; rounded and subangular sand grains; about 15 percent non-plastic fines with low dry strength; well compacted and moist; inplace; alluvial sand (SM)
			GP		Red	Poorly graded gravels or gravel-sand mixtures; little or no fines	Predominantly one size or a range of sizes with some intermediate sizes missing	
		Gravels with fines (Appreciable amount of fines)	GM		Yellow	Silty gravels, poorly graded gravel-sand-silt mixtures	Non-plastic fines or fines with low plasticity (for identification procedures, see ML and MI below)	
			GC		Yellow	Clayey gravels, poorly graded gravel-sand-clay mixtures	Plastic fines (for identification procedures, see CL and CI below)	
	Sands More than half of coarse fraction is smaller than 4·75-mm IS Sieve size	Clean sands (Little or no fines)	SW		Red	Well graded sands, gravelly sands; little or no fines	Wide range in grain size and substantial amounts of all intermediate particle sizes	
			SP		Red	Poorly graded sands or gravelly sands; little or no fines	Predominantly one size or a range of sizes with some intermediate sizes missing	
		Sands with fines (Appreciable amount of fines)	SM		Yellow	Silty sands, poorly graded sand-silt mixtures	Non-plastic fines or fines with low plasticity (for intermediate procedures, see ML and MI below)	
			SC		Yellow	Clayey sands, poorly graded sand-clay mixtures	Plastic fines (for identification procedures, see CL and CI below)	

(For visual classification the 5-mm size may be used as equivalent to the 4·75-mm IS Sieve size)

Table 2-2

FINE-GRAINED SOILS

More than half of material is smaller than 75-micron IS Sieve size

The 75-micron IS Sieve size is about the smallest particle visible to the naked eye

	Group Symbol	Pattern	Colour	Typical Names	Identification Procedures (on Fraction Smaller than 425-Micron IS Sieve Size): Dry Strength	Dilatancy	Toughness	
Silts and clays with low compressibility and liquid limit less than 35	ML		Blue	Inorganic silts and very fine sands rock flour, silty or clayey fine sands or clayey silts with none to low plasticity	None to low	Quick	None	For undisturbed soils add information on structure, stratification, consistency in undisturbed and remoulded states, moisture and drainage conditions
	CL		Green	Inorganic clays, gravelly clays, sandy clays, silty clays, lean clays of low plasticity	Medium	None to very slow	Medium	Give typical name, indicate degree and character of plasticity, amount and maximum size of coarse grains; colour in wet condition; odour, if any, local or geologic name and other pertinent descriptive information and symbol in parenthesis
	OL		Brown	Organic silts and organic silty clays of low plasticity	Low	Slow	Low	
Silts and clays with medium compressibility and liquid limit greater than 35 and less than 50	MI		Blue	Inorganic silts, silty or clayey fine sands or clayey silts of medium plasticity	Low	Quick to slow	None	*Example:* Clayey silt, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loose (ML)
	CI		Green	Inorganic clays, gravelly clays, sandy clays, silty clays, lean clays of medium plasticity	Medium to high	None	Medium	
	OI		Brown	Organic silts and organic silty clays of medium plasticity	Low to medium	Slow	Low	
Silts and clays with high compressibility and liquid limit greater than 50	MH		Blue	Inorganic silts of high compressibility, micaceous or diatomaceous fine sandy or silty soils, elastic silts	Low to medium	Slow to none	Low to medium	
	CH		Green	Inorganic clays of high plasticity, fat clays	High to very high	None	High	
	OH		Brown	Organic clays of medium to high plasticity	Medium to high	None to very slow	Low to medium	
Highly Organic Soils	Pt		Orange	Peat and other highly organic soils with very high compressibility	Readily identified by colour, odour, spongy feel and frequently by fibrous texture			

NOTE — *Boundary classification:* Soil possessing characteristics of two groups are designated by combinations of group symbols, for example, GW-GC, Well-graded, gravel-sand mixture with clay binder.

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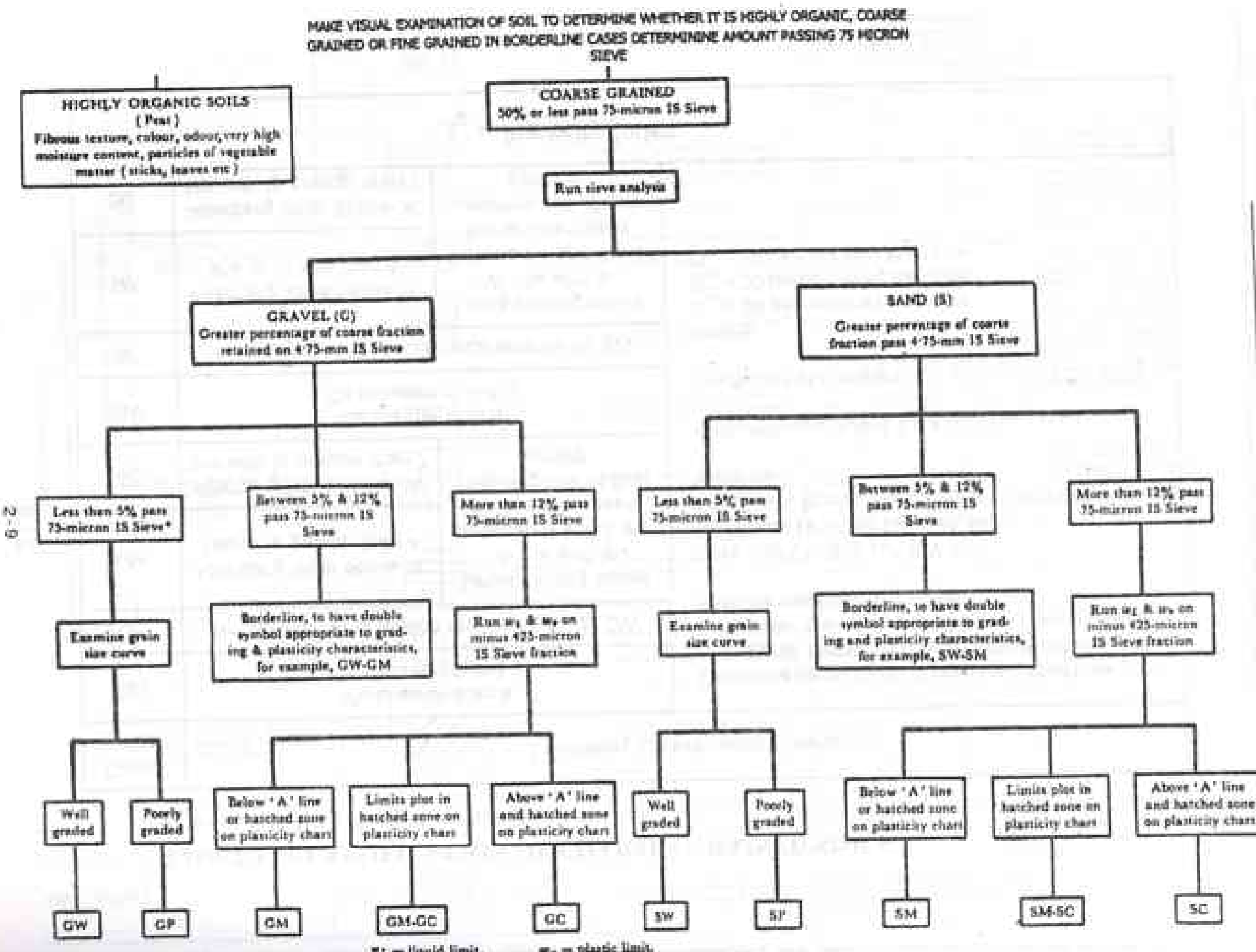
Table 2.3 : CLASSIFICATION OF COARSE - GRAINED SOILS

(clause 3.5.1 of IS 1498-1970)

Group Symbols	Laboratory Classification Criteria		
GW	Cu Greater than 4 Cc Between 1 and 3		Determine percentages of gravel and sand from grain - size curve. Depending on percentage of fines (fraction smaller than 75 micron IS sieve) coarse - grained soils are classified as follows: Less than 5% GW, GP, SW, Sp More than 12% GM, GC, SM, SC 5% to 12% Border-line cases requiring use of dual symbols Uniformity coefficient, $Cu = D_{60} / D_{10}$ Coefficient of curvature, $Cc = (D_{30})^2 / (D_{10} \times D_{60})$ Where, D_{60} = 60 percent finer than size D_{30} = 30 percent finer than size D_{10} = 10 percent finer than size
GP	Not meeting all gradation requirements for GW		
GM	Atterberg limits below 'A' line or Ip greater than 4	Limits plotting above 'A' line with Ip between 4 and 7 are border-line cases requiring use of dual symbol	
GC	Atterberg limits above 'A' line with Ip greater than 7		
SW	Cu greater than 6 Cc between 1 and 3		
SP	Not meeting all gradation requirements for SW		
SM	Atterberg limits below 'A' line or Ip less than 4	Limits plotting above 'A' line with Ip between 4 and 7 are border-line cases requiring use of dual symbols	
SC	Atterberg limits above 'A' line with Ip greater than 7		

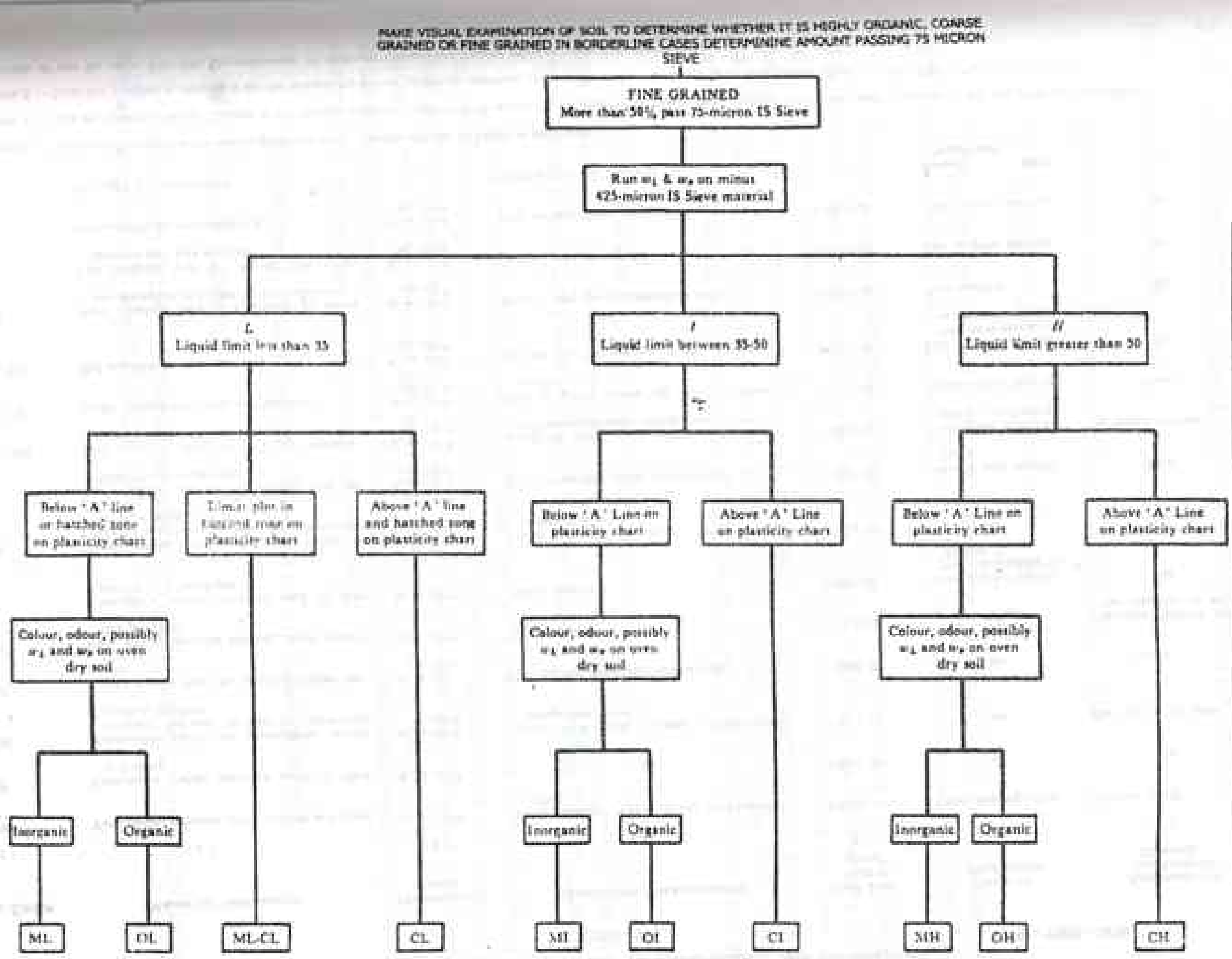
I_p = Plasticity Index



w_L = liquid limit. w_P = plastic limit.

*If fines interfere with free drainage properties use double symbol, such as GW-GM.

Table 2-4 AUXILIARY LABORATORY IDENTIFICATION PROCEDURE [CLAUSES 3.5.1 and 3.5.3]



AUXILIARY LABORATORY IDENTIFICATION PROCEDURE [CLAUSES 3.5.1 and 3.5.3]

TABLE 6 CHARACTERISTICS PERTINENT TO EMBANKMENTS AND FOUNDATIONS

(*Clause* 3.5)

IS : 1498 - 1970

Soil Group (1)	Value of Embankment (2)	Permeability cm/s (3)	Compaction Characteristics (4)	Unit Dry Weight g/cm³ (5)	Value of Foundation (6)	Requirements for Seepage Control (7)
GW	Very stable; pervious shells of dikes and dams	$K > 10^{-2}$	Good; tractor, rubber tyred, steel-wheeled roller	2·00-2·16	Good bearing value	Positive cutoff
GP	Reasonably stable, pervious shells of dikes and dams	$K > 10^{-2}$	do	1·84-2·00	do	do
GM	Reasonably stable; not particularly suited to shells, but may be used for impervious cores or blankets	$K = 10^{-3}$ to 10^{-6}	Good; with close control, rubber-tyred, sheeps-foot roller	1·92-2·16	do	Toe trench to none
GC	Fairly stable; may be used for impervious core	$K = 10^{-6}$ to 10^{-8}	Fair; rubber-tyred, sheeps-foot roller	1·84-2·08	do	None
SW	Very stable; pervious sections, slope protection required	$K > 10^{-3}$	Good; tractor	1·76-2·08	do	Upstream blanket and toe drainage or wells
SP	Reasonably stable; may be used in dike section with flat slopes	$K > 10^{-3}$	Good; tractor	1·60-1·92	Good to poor bearing value depending on density	do
SM	Fairly stable; not particularly suited to shells, but may be used for impervious cores or dikes	$K = 10^{-3}$ to 10^{-6}	Good; with close control, rubber-tyred, sheeps-foot roller	1·76-2·00	do	do
SC	Fairly stable; use for impervious core for flood control structures	$K = 10^{-6}$ to 10^{-8}	Fair; sheeps-foot roller, rubber tyred	1·68-2·00	Good to poor bearing value	None
ML, MI	Poor stability; may be used for embankments with proper control	$K = 10^{-3}$ to 10^{-6}	Good to poor, close control essential; rubber-tyred roller, sheeps-foot roller	1·52-1·92	Very poor, susceptible to liquefaction	Toe trench to none
CL, CI	Stable; impervious cores and blankets	$K = 10^{-6}$ to 10^{-8}	Fair to good; sheeps-foot roller, rubber-tyred	1·52-1·92	Good to poor bearing	None
OL, OI	Not suitable for embankments	$K = 10^{-4}$ to 10^{-6}	Fair to poor; sheeps-foot roller	1·28-1·60	Fair to poor bearing, may have excessive settlements	do
MH	Poor stability; core of hydraulic fill dams not desirable in rolled fill construction	$K = 10^{-4}$ to 10^{-6}	Poor to very poor; sheeps-foot roller	1·12-1·52	Poor bearing	do
CH	Fair stability with flat slopes; thin cores, blankets and dike sections	$K = 10^{-6}$ to 10^{-8}	Fair to poor; sheeps-foot roller	1·20-1·68	Fair to poor bearing	do
OH	Not suitable for embankments	$K = 10^{-6}$ to 10^{-8}	Poor to very poor; sheeps-foot roller	1·04-1·60	Very poor bearing	do
Pt	Not used for construction	—	Compaction not practical	—	Remove from foundation	—

NOTE 1 — Values in Column 2 and 6 are for guidance only. Design should be based on test results.

NOTE 2 — The equipment listed in Column 4 will usually produce densities with a reasonable number of passes when moisture conditions and thickness of lift are properly controlled.

NOTE 3 — Unit dry weights in column 5 are for compacted soil at optimum moisture content for Indian Standard light compaction effort [*see* IS : 2720 (Part VII)-1965*].

*Methods of test for soils: Part VII Determination of moisture content-dry density relation using light compaction.

2.3.0 FIELD IDENTIFICATION AND CLASSIFICATION OF SOILS

Field classification is based on:

(a) Particle size distribution discerned by the eye, and

(b) Plastic properties of the fine material

The general procedure is to make accurate visual observation and a few simple tests as described below and compare the results with description set out in the table.

Classify a representative sample as coarse or fine grained by observing whether 50 per cent or more of the sample consists of particles that can be individually seen by the naked eye; if the sample is coarse grained, observe if 50 per cent or more of the coarse grains are larger than 1/4 inch in diameter, in which case the soil belongs to the gravel group; and if otherwise, to the send groups; classify this further into one of the many gravel or sand groups described in the table by tests on the fraction smaller than 1/8 inch.

When the sample consists of particles more than 50 per cent which cannot be seen individually, the soil is fine grained Classify the sample by performing the shaking, breaking and thread tests on the minus 1/8 inch material and noting the colour and odour.

If the characteristics are 50-50 give the two identification symbols, the sample may be classified as under. In addition to mentioning the group symbols, always furnish other pertinent data like colour, maximum grain size shape and hardness of the grains, surface condition, friability, quality and quantity of extraneous material, etc. Do not attempt to classify on the result of any one test; complete all the tests and then compare with the description in the table to determine the grouping.

The following describes the procedure for the different tests:

1. Visual inspection:

(a) Grain shape:- Observe the sand and gravel particles and describe whether they are angular, sub-angular, rounded or sub-rounded.

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(b) Grading:- The naked eye can identify only particles retained on 200 mesh sieve. Spread a representative sample on a flat surface and observe the distribution or uniformity of grain size, see if all sizes are proportionately seen, say well graded. If one size predominates, poorly graded.

2. Shaking test:- This helps identification of a fine grained soil as fine sand, Silt or clay. Wet or saturate a small sample. Form into a small pat and shake it horizontally in the palm of the hand; observe whether there is a watery film on the surface, then squeeze the pat between fingers and see whether water seen disappears leaving a dull surface. Rapid reaction (Water appears and disappears, rapidly) shows plastic silt or silty clay. No reaction shows clay or organic matter with high plasticity.

3. Breaking Test:- The breaking test is a measure of the cohesion of the soil. Wet the soil and form it into a pat. Dry the pat and break it between the fingers and see if the resistance is slight, medium or high. Slight dry strength (very easily broken by finger) shows inorganic silt, very fine sand or rock flour or combination of these with traces of clay. Medium strength show low to medium plastic inorganic clay, silty-clay or sandy clay; if the powdered soil feels gritty, the material is sandy. If it is impossible to break the pat with the fingers, the soil is highly plastic clay or contains cementing material like Calcium Carbonate or Iron Oxide.

4. Plasticity or Thread Test:- The test indicates the degree of plasticity of a soil. Remove the coarse fraction from the sample, wet the fines and mix thoroughly be kneading. Roll the wet soil between the palms of the hands or on smooth surface to form threads 1/8 inch in diameter. The moisture content at this stage is called the plastic limit of the soil. Note the toughness of the thread and see if it can be lumped again. High plasticity is indicated by a tough thread which can be lumped again: Medium plasticity, by a thread of medium toughness which cannot be lumped again without getting crumbled; and low plasticity by a weak thread breaking easily and not letting itself to be lumped.

5. Odour and Colour Test:- Wet organic soils have a bad odour felt more when heated. Organic matter is generally dark in colour, clays smell 'earthy' when slightly moistened.

6. Acid Test:- This is mainly intended for showing up Calcium Carbonate. Treatment with a dilute solution of Hydro Chloric Acid will cause the material to react visibly.

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7. Shine Test:- This enables quick detection of the presence of clay in a soil. Cleave or cut through a lump of the sample and see if the surface is shining. A shiny surface indicates clay while a dull surface shows silt, sandy clay or clay of low plasticity.

8. Sedimentation Test:- This test helps approximate gradation of fine-grained soil. Shake a small quantity of the sample in a jar of water and let it settle. Grading is indicated by the materials in the different layers of the suspension. Sand settles in less than a minute, silts from one minute to sixty minutes; clays take comparatively a long time, sinking approximately about six inches in one hour. Observe settlements in a period of ten minutes.

2.3.1 Procedure for collecting and sending disturbed soil samples from borrow areas for embankment construction.

(1) Exploration for borrow areas is performed to locate comparatively large quantities of soils in which accessibility, uniformity and workability are very important. Generally it is preferable on the upstream side of the proposed earth dam and within the allowable leads as for as possible. These areas so selected should not be within 100m from the toe of the embankment proposed. This type of borrow areas is first located on the basis of surface indication.

(2) In each area, three or four pits of dimensions 1 x 1 x 1.5m should be excavated at well distributed location so as to fix the stretch of each type of the soil available.

(3) Different layers of soil exposed in each pit should be examined excluding the top 15cm. layer. Thickness of each layer of soil should be noted so as to establish the volume of soil available.

(4) About 25 to 30 Kgs. of soil of each type should be scraped and securely packed in gunny bags.

(5) With each soil sample sent, a slip containing following information should be enclosed and the same details should also be written on the outside of the gunny bags.

i) Name of the scheme or project.
ii) Pit No.
iii) Sample No.
iv) Depth at which sample is taken.
v) Survey No. and Chainage No.

(6) The soil sample should be sent to Director, A.P.Engineering Research Labs., Himayatsagar, Hyderabad - 500 030 AP. If the Soil samples are to be sent by train, they should be sent by passenger train and freight paid. Samples can be sent, by lorry transport by paying freight charges.

(7) Alongwith the samples the following information and drawings should be sent.

i) A short note regarding the location and history of the Project.
ii) A contoured site plan showing the borrow areas and borrow pits and the alignment of the proposed embankment.
iii) The L.S. and Cross Section of the proposed embankment showing the position of the sluices, weir etc.
iv) Profile of each borrow pit.

2.3.2 Procedure for collecting and Sending Undisturbed Soil Samples for Foundation Studies of Earth Dams.

The main purpose of the Foundation Investigations is to study the strength and permeability characteristics of the Sub-Soil Strata along the axis line by testing the undisturbed soil samples in the laboratory. The depth to which these studies are confined is governed by the height of water column at every chainage under M.W.L.conditions. Normally the trial pits are opened at intervals of four or five chains (closer if justified) to the maximum depth of water, which is going to be stored. As open trial pits may not be practicable beyond. 4.5 to 6 m feet depth, hand boring set or mechanical rotary type of boring machines may be used for obtaining undisturbed soil samples. For taking undisturbed soil samples from open pits following procedure is recommended:

1. Soil samples to be made out of thin walled seamless tube either 38mm dia. and 20 to 25cm. long or 10cm. dia. and 45cm. long. Each sampler should be provided with a cutting edge at one end bevelled on the outside. The other end be provided with holes to pit the adopter of the sampling rod shown in the drawing. 10cm dia. samplers are preferred.

2. After scraping the exposed layer of the soil 5cm. to 8cm. the sampler well oiled inside, should be driven in vertically or horizontally by giving light blows on the head of the rod with 5 kg. hammer. Care should however be taken not to over drive the sampler and disturb the ~~density~~ of the soil. To remove the sampler, the soil core should be sheared by rotating the sampler and then screwed out without giving a jerk.

3. The sampler is detached from the adopter and is sealed by melted paraffin about 12.5cm thick at both the ends. The object of the paraffin plugs is to conserve the moisture content of the sample during transit.

4. Undisturbed samples have to be taken for each distinctive layer of soil exposed or at intervals of 3 to 5 feet if the profile shows uniform stratum. At least three samples from every sampling point should be taken to give an average idea of density of the soil layer, if 38mm dia. samples are used. Otherwise one 10cm. dia. sampler is sufficient. At least 25 Kgs. of disturbed soil samples of each type should also be collected and sent to the laboratory in the gunny bags. Every sample should bear the following particulars written legibly:

i) Name of Scheme or Project.
ii) Pit No.
iii) Sample No.
iv) Depth of sampling and reduced level.

5. Along with the samples following drawings be sent:

i) Site plan showing the trial pits along the axis of the dam.
ii) L.S. of the embankment showing the profile of the trial pits and other relevant details like ground water table, and sheet rock if met during the investigations.

NOTE:
3) Sampling Equipment: Sampling tubes, adopter, Auger rods, turning rods, extension rods are required. Sampling tubes shall be made of thin walled seamless, M.S. or G.I. tubes of 10cm dia. and 45 cm. in length. They shall be provided with a cutting edge on one side and bevelled out side. The other end shall be provided with holes to fit the adopter of the rods. The wall thickness of the sampling tube shall be such that the area ratio (Ar) given by the following does not exceed 10 to 15 percent.

$$A_r = \frac{D^2 - d^2}{d^2} \times 100$$

Where,
D = external dia. of sampling tube, d = internal dia. of sampling tube
Generally the internal dia. = 10 cm and external dia = 10.6 cm.

RECORD OF BORING

Name of Boring Organization (clause 6.5.1 OF IS 1892-1979)

Bored for:.......................................
Ground surface level:.............................
Type of boring: Wash boring
Diameter of boring:...............................
Inclination: Vertical
Boring:...

Location: Site
Boring No:..
Soil sampler used:................................
Date started:.....................................
Date completed:...................................
Recorded:...

Description of Strata	Soil Classification	Thickness of Stratum	Depth from Ground Surface	R.L of Lower Contact	Samples: Type	Samples: No.	Samples: Depth and Thickness of Sample	Ground-Water Level	Remarks
Fine to medium sands with practically no binder	SP		1m, 2m, 2.5m		Undisturbed	1	1m, 1.4m, 1.7m, 2m		
Silty clays of medium plasticity, no coarse or medium sands	CI		3m, 4m, 5m		Undisturbed	2	3m, 4m, 4.3m, 5m	Not struck up to 5m depth	

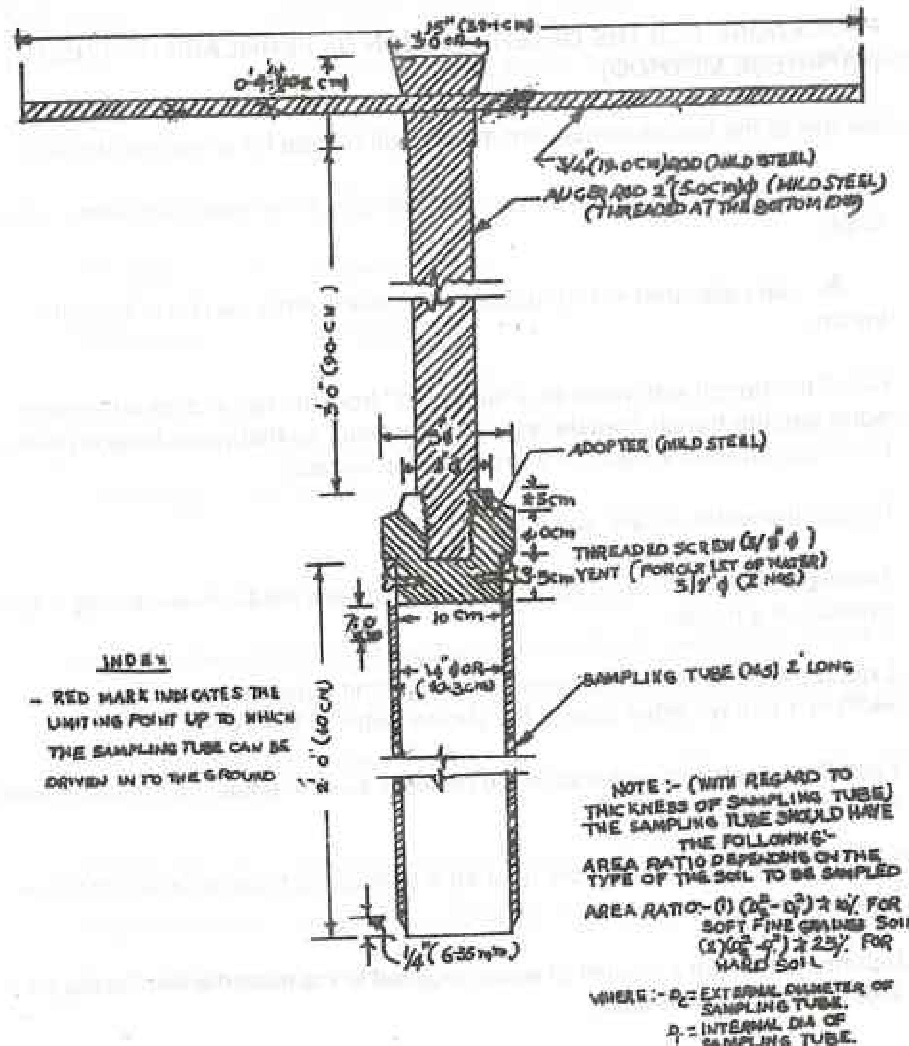


FIG:-2.1 10-30 CM Ø SAMPLING TUBE WITH ADOPTER USED FOR SAMPLING FROM OPEN TRIAL PITS

2.4.0 PROCEDURE FOR THE DETERMINATION OF PERMEABILITY (INSITU) (JAPANESE METHOD)

1. Scrape all the top loose material at the desired area for an extent of 6' X 6'.
2. Make a trench of bottom dimensions 4' X 2' and 1' - 9" deep with sides. 1/4:1 slope.
3. A can calibrated in 1/10 gallons from which water can be fed into the trench.
4. Fill up the trench with water to a depth of 2" from the top and go on feeding water into the trench from the top calibrated can, so that water level is main tained constant at a depth of 3" from the top surface.
5. Record the water in-take every one hour.
6. The experiment should be continued till the in-take readings are steady say for a period of 4 hours.
7. Extend the pit to the dimensions of 4' X 4' at the bottom by cutting an extra width of 1 foot on either side of the shorter sides of the trench.
8. Feed the trench with water as in the previous case maintaining the water level in the pit 3" from top.
9. Measure the in-tak · every one hour for a period of 2 hours or more till the in-take is constant.
10. Determine the extra amount of water required to maintain the level in the pit after it has been extended.

If the extra amount of water is 1/100 gallon per hour permeability is 1 ft/year.

If the extra amount of water is 1/10 gallon per hour, the permeability is 10 ft/year.

Suppose : = 1/100 G.P.H. Then K = 1 ft/year.

= 1/100 G.P.H. to 1/10 G.P.H. is in the limits of 1-10 ft/year.

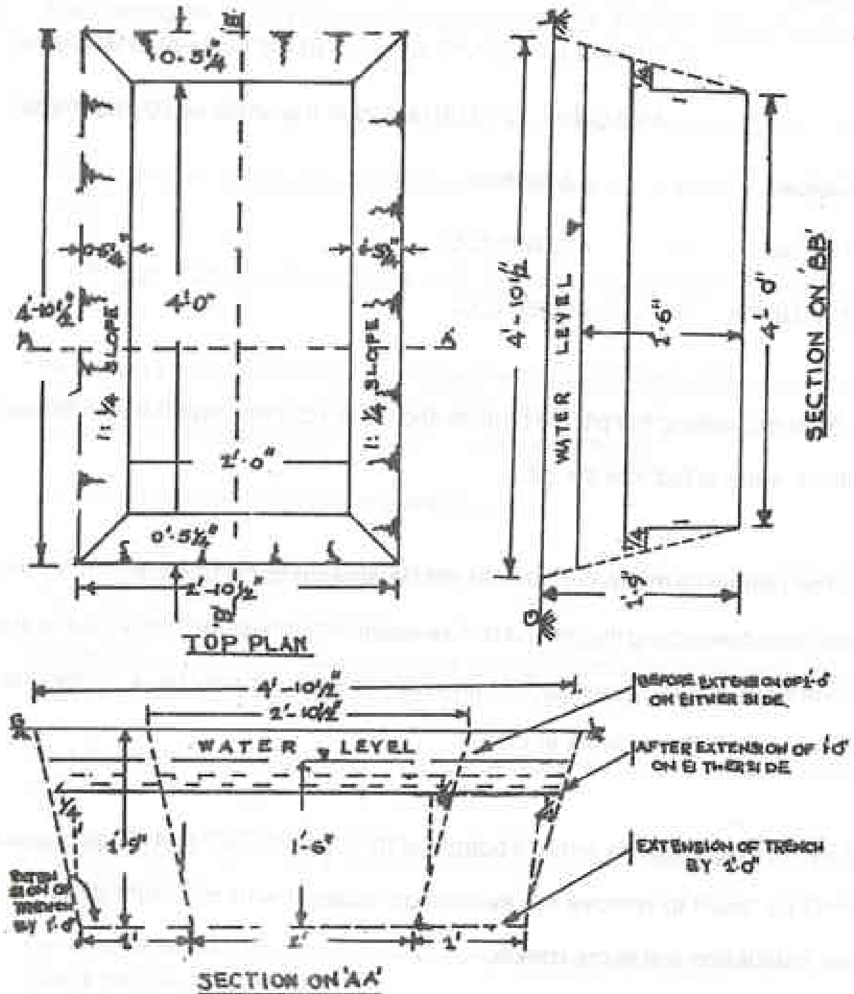
= 1/10 G.P.H. to 1 G.P.H. K is in the limits of 10-100 ft/year.

1 Gallon	=	4.536 litres
1/10 Gallon	=	453.600 C.C.
1/100 Gallon	=	45.360 C.C.

Note: 1. After excavating the pit, the bottom and sides may be searified with an iron nail before water is fed into the pit.

2. The bottom of the trench should not be allowed to be trampled under the feet and after dewatering the pit and before extending the width from 2' to 4' it shall be covered by polyethylene cloth or tarpaulin or empty gunny bags. Only one man should be allotted to work in the pit.

3. While extending the width of bottom of trench from 2'-0" to 4'-0" all precautions shall be taken to remove the excavation material with minimum disturbance to the foundation soil in the trench.



NOTE:-
SIDE SLOPES OF THE TR...H DEPEND ON THE SOIL BUT WITH BUT ARE TENTATIVELY PROPOSED AS 1:¼ TRENCH MAY BE EXCAVATED WITH VERTICAL SIDES IF THE SOIL PERMITS.

FIG-2.2 IN-SITU PERMEABILITY TEST(TRENCH METHOD)
(JAPANESE METHOD)

SCALE:-1"=1'-0"

TABLE 2-6 SUITABILITY FOR CANAL SECTIONS, COMPRESSIBILITY, WORKABILITY AS A CONSTRUCTION MATERIAL AND SHEAR STRENGTH (Clause 3.8 of IS 1498-1970)

Soil Group	Relative suitability for canal sections		Compressibility when compacted and saturated	Workability as a construction Material	Shearing strength when compacted and saturated
	Erosion Resistance	Compacted earth lining			
GW	1	-	Negligible	Excellent	Excellent
GP	2	-	Negligible	Good	Good
GM	4	4	Negligible	Good	Good
GC	3	1	Very low	Good	Good to fair
SW	6	-	Negligible	Excellent	Excellent
SP	7,if gravelly	-	Very low	Fair	Good
SM	8,if gravelly	5(Erosion critical)	Low	Fair	Good
SC	5	2	Low	Good	Good to fair
ML,MI	-	6(Erosion critical)	Medium	Fair	Fair
CL,CI	9	3	Medium	Good to Fair	Fair
OL,OI	-	7(Erosion critical)	Medium	Fair	Poor
MH	-	-	High	Poor	Fair or Poor
CH	10	8(Volume change (critical)	High	Poor	Poor
OH	-	-	High	Poor	Poor
Pt	-	-	-	-	-

* Number 1 is th best.

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TABLE 2-6(a)

Void Ratio, Porosity and Bulk Unit Weight of Typical Soils (Kezdi, 1974)

Soil Type	State of Soil	Porosity	Void Ratio	Bulk unit weight, g/cc Dry	natural	saturated	buoyant
Sandy gravel	loose	38-42	0.61-0.72	1.42-1.73	1.81-2.03	1.93-2.13	0.91-1.12
	dense	18-25	0.22-0.33	1.93-2.13	2.03-2.34	2.13-2.42	1.12-1.42
Coarse sand	loose	40-45	0.67-0.82	1.31-1.51	1.61-1.93	1.81-1.93	0.81-0.90
Medium sand	dense	25-32	0.33-0.47	1.73-1.81	1.81-2.13	2.03-2.13	1.01-1.12
Uniform fine sand	loose	45-48	0.82-0.82	1.42-1.51	1.51-1.93	1.81-1.93	0.81-0.91
	dense	33-36	0.49-0.56	1.73-1.81	1.81-2.13	2.03-2.13	1.01-1.12
Coarse silt	loose	45-55	0.82-1.22	1.31-1.51	1.51-1.93	1.81-1.93	0.81-0.91
	dense	35-40	0.54-0.67	1.61-1.73	1.73-2.13	2.03-2.13	1.01-1.12
Silt	Soft	45-50	0.82-1.00	1.31-1.51	1.61-2.03	1.81-2.03	0.81-1.01
	slightly plastic	35-40	0.54-0.67	1.61-1.73	1.73-2.13	2.03-2.13	1.01-1.12
	hard	30-35	0.43-0.49	1.81-1.93	1.81-1.93	1.81-2.23	1.12-1.21
Lean clay	soft	50-55	1.00-1.22	1.31-1.42	1.51-1.81	1.81-2.03	0.81-1.01
	slightly plastic	35-45	0.54-0.82	1.51-1.81	1.73-2.13	1.93-2.13	0.91-1.12
	hard	30-35	0.43-0.54	1.81-1.93	1.81-2.23	2.13-2.23	1.12-1.21
Fat clay	Soft	60-70	1.50-2.30	9.05-1.51	1.21-1.81	1.42-1.81	0.40-0.81
	slightly plastic	40-55	0.67-1.22	1.51-1.81	1.51-2.03	1.73-2.13	0.70-1.12
	hard	30-40	0.43-0.67	1.81-2.03	1.73-2.23	1.93-2.32	0.91-1.31

TABLE 2-6(b) TEXTURAL CLASSIFICATION OF SOIL BASED ON GRADING

Textutal class	Composition in per cent Sand	Silt-size	Clay-size
Sand	80-100	0-20	0-20
Sandy loam	50-80	0-50	0-20
Loam	30-50	30-50	0-20
Silt 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	20-30
Sandy clay	55-70	0-15	30-45
Silty clay	0-15	55-70	30-45
Clay	0-55	0-55	30-100

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Table 2-6(C)

Permeability and drainage characteristics of soils (Casagrande and Fadum)

	10^{2}	10^{1}	10^{0}	10^{-1}	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-6}	10^{-7}	10^{-8}	10^{-9}
Drainage	Good						Poor		Practically impervoius			
Soil types	Clean gravel		Clean sands, clean sand and gravel mixtures			Very fine sand, organic and inorganic silts, mixtures of sand silt and clay, glacial till, stratified clay deposits etc.				Impervious soils, e.g. homogeneous clays below zone of weathering		
					Impervious soils modified by effects of vegetation and weathering							
Direct determination of K	Direct testing of soil in its original position : pumping tests - reliable if properly conducted; considerable experience required											
	Constant -head permeameter : little experience required											
	Falling head permeameter Reliable Little experience required					Falling head permeameter, unreliable Much experience required			Falling head permeameter fairly reliable considerable experience required			
Indirect determination of K	Computation from grain size distribution Applicable only to clean cohesionless sands and gravels					Computation based on results of consolidation tests, reliable - considerable experience required						

Table 2-6(D)

Approximate Values of ϕ' for granular Soils as affected by Stte of compaction, Size, Gradation and Angularity of Grains (Leonards, 1962)

SN	Size of grains	State of compaction	Values of ϕ' deg Rounded grains	Angular grain
1	Medium Sand	Very loose	28-30	32-34
		Moderately dense	32-34	36-40
		Very dense	35-38	44-46
2	* Sand and Gravel			
	65%G - 35%S	Loose	-	39
	65%G - 35%S	Moderately dense	37	41
	80%G - 20%S	Dense	-	45
	80%G - 20%S	Loose	34	-
3.	Blastererd Rock fragments		40 -45	

** Values interpolated from Holtz and Gibbs (1956)*

Relationship between Relative Density, Penetration Resistance and Angle of Internal Friction of Cohesionless Soils (Winterkorn and Fang, 1975)

Type of Soil	Penetration resistance N	Relative density Dr	Angle of Internal friction Peck(1974)	Meyerhof(1956)
Very loose Sand	< 4	< 0.2	< 29	< 30
Loose	4-10	0.2 - 0.4	29-30	30-35
Medium Sand	10-30	0.4-0.6	30-36	35-40
Dense Sand	30-50	0.6-0.8	36-41	40-45
Very dense Sand	>50	>0.8	>41	> 45

Table 2-7

Mechanical Properties of various Rocks (Obert and Duvall 1967; and Winterkorn and Fang 1975)

	Rock Type	Bulk Qty g / cc	Porosity %	Compressive strength range Kg/cm² x 10² Min	Max	Tensile strength Kg/sqcm	Modulus of Elasticity range kg/cm² x 10² (10-3 cm/cm) Min	Max	Breaking range Ratio Min	Max	Poisson's
1	Marble	2.6-2.7	0.5-2	4.71	24.25	70-200	5.06	8.36	930	4300	0.27-3.0
2.	Lime stone	2.2-2.6	5-20	0.49	20.95	52-250	0.28	8.43	1630	8000	0.27-3.0
3.	Granite	2.6-2.7	0.5-1.5	16.17	29.95	70-250	2.18	7.03	4030	8660	0.23-0.27
4.	Diorite	.	.	18.42	27.98	150-300	5.62	10.26	2070	3370	0.26-0.27
5.	Gneiss	2.9-3.0	0.5-1.5	15.60	18.56	50-200	2.46	6.81	2300	7790	.
6.	Sand stone	2.0-2.6	5.0-25.0	3.37	23.97	40-250	0.63	5.13	3150	12000	.
7.	Dolomite	2.5-2.6	1.0-5.0	7.73	37.26	150-250	1.97	7.94	2170	4649	0.30
8.	Marl Stone	.	.	5.69	19.82	.	1.26	4.92	1800	6610	.
9.	Diabase	.	.	15.88	32.76	.	6.25	9.77	2540	4010	0.27-0.30
10.	Basalt	2.8-2.9	0.1-1.0	12.44	36.70	100-300	5.83	8.71	2540	5980	.
11.	Green stone	.	.	12.44	31.99	.	4.85	10.68	1540	3800	.
12.	Shale	2.0-2.4	10-30	7.6	23.55	20-100	1.12	6.96	1720	9250	.
13.	Silt stone	.	.	3.51	32.20	.	3.16	7.59	690	4910	.

TABLE 2-8
Coefficient of Friction of Rocks and Minerals
(Laeger & Cook, 1969)

Minerals	Coefficient of friction	(West)	Rocks	Coefficient of friction
Nacl	0.70	-	Sand Stone	0.51-0.68
Pbs	0.6	-	Granite	0.60-0.64
S	0.50	-	Quartzite	0.48-0.67
Al2O3	0.40	-	Dolorite	0.64-0.95
			Dolomite	0.40
Glass	0.70	-	Tracyte	0.56-0.68
Diamond	0.10-0.30	-	Marble	0.62-0.75
Quartz	0.11-0.10	0.42-0.65	Porphyry	0.86
Feldspar	0.1	0.46	Gneiss	0.61-0.71
Calcite	0.14	0.68	Gabbro	0.18-0.66
Muscovite	0.43	0.23		
Biotite	0.31	0.13		
Serpentine	0.62	0.29		
Talc	0.36	0.16		

Classification of Explosives

Class 1 Gun Powder
Class 2 Nitrate mixtures(All slurry explosives manufactured by IDLChemical Limited)
Class 3 Nitro Compounds
Division 1 : Nitro - Glycerine based Explosives
Division 2 : PETN, TNT and Pentolite Boosters
Class 4 Chlorate Mixtures
Class 5 Initiators (Lead Azide, Fulminate etc.)
Class 6 Ammunition
Division 1 : Safety Fuse, Ignitor Cord, Connectors
Division 2 : Plastic Ignitor cord, Detonating Fuse, (D-Cord I-Cord & S-Cord)
Division 3 : Detonators, Detonating Relays
Class 7 Fire Works
Class 8 Lox

Table 2-9
Cubic metre of Rock broken per metre of bore hole

Burden	Spacing between Charges (m): 1.0	1.2	1.8	2.4	3.0	3.6	4.2	4.8	5.4	6.0	6.6	7.2	7.8	8.4	9.0
	Yield of muck in cubic metres														
1.0	1.0	1.2	1.8	2.4	3.0	3.6	4.2	4.8	5.4	6.0	6.6	7.2	7.8	8.4	9.0
1.2	1.2	1.4	2.2	2.9	3.6	4.3	5.0	5.8	6.5	7.0	7.9	8.6	9.4	10.0	11.8
1.8	1.8	2.2	3.2	4.3	5.4	6.5	7.6	8.6	9.7	10.9	11.9	13.0	14.0	15.1	16.2
2.4	2.4	2.9	4.3	5.8	7.2	8.6	10.1	11.5	13.0	14.4	15.8	18.7	18.7	20.2	21.6
3.0	3.0	3.6	5.4	7.2	9.0	10.8	12.6	14.4	16.2	18.0	19.8	21.8	23.4	25.2	27.0
3.6	3.6	4.3	6.5	8.6	10.8	13.1	15.1	17.3	19.4	21.6	23.8	25.9	28.1	30.2	32.4
4.2	4.2	5.0	7.6	10.1	12.6	15.0	17.6	20.0	22.7	25.2	27.7	30.2	32.8	35.4	37.8
4.8	4.8	5.8	8.6	11.5	14.4	17.3	20.0	23.0	26.0	28.8	31.7	34.6	37.4	40.3	43.2
5.4	5.4	6.5	9.7	13.0	16.2	19.3	22.6	26.0	29.2	32.2	35.6	38.9	42.1	45.4	48.6
6.0	6.0	7.2	10.8	14.4	18.0	21.6	25.2	28.8	32.5	36.0	39.6	43.2	46.8	50.4	54.0
6.6	6.6	7.9	11.9	15.8	19.8	23.8	27.7	31.7	35.6	39.6	43.6	47.5	57.5	55.4	59.4
7.2	7.2	8.6	13.0	17.3	21.6	25.9	30.2	34.6	38.9	43.2	47.5	51.9	56.1	60.5	64.8
7.8	7.8	9.4	14.0	18.7	23.4	28.1	32.8	37.4	42.1	46.8	51.5	56.1	60.8	65.1	70.2
8.4	8.4	10.0	15.1	20.0	25.2	30.2	35.3	40.3	45.4	50.4	55.4	60.5	65.5	70.6	75.6
9.0	9.0	10.8	16.2	21.6	27.0	32.4	37.8	43.2	40.6	54.0	59.4	64.8	70.2	75.6	81.0

Table 2-10

Loading Densities in Kgs/ Metre of Bore Hole

Bore Hole dia in mm	Density of Explosives in gm/cc										
	0.9	0.95	1.00	1.05	1.10	1.15	1.20	1.25	1.30	1.35	1.40
32	0.72	0.76	0.80	0.84	0.88	0.92	0.96	1.00	1.04	1.08	1.12
40	1.13	1.19	1.26	1.33	1.39	1.45	1.50	1.56	1.62	1.68	1.75
50	1.76	1.86	1.96	2.05	2.15	2.25	2.35	2.45	2.54	2.64	2.74
75	3.96	4.18	4.41	4.63	4.85	5.07	5.29	5.51	5.73	5.95	6.17
100	7.06	7.45	7.25	8.24	8.63	9.02	9.42	9.81	10.2	10.6	11.0
125	11.04	11.65	12.27	12.88	13.50	14.11	14.72	15.34	15.95	16.56	17.17
150	16.86	17.76	18.65	19.55	20.59	21.49	22.38	23.28	24.32	25.22	26.11
175	21.64	22.84	24.05	25.25	26.45	27.65	28.86	30.36	31.26	32.46	33.67
200	28.27	29.85	31.42	32.99	34.56	36.13	37.70	39.27	40.84	42.42	43.99
250	44.18	46.63	49.09	51.54	54.00	56.45	58.91	61.36	63.82	63.27	68.72

TABLE - 2-11
TESTS FOR DIFFERENT PHASES OF EXPLORATION
(Clause 8.1 of IS 1892-79)

Phase of Exploration	Tests Necessary on a Sample	
	Type of Test	Detailed Tests
i) Reconnaissance exploration		Visual classification (see IS:1498-1970)
ii) Detailed exploration	Physical Tests	Liquid and plastic limits (see IS:2720 (Part V) -1970) Grain size analysis (see IS:2720 (Part IV) - 1975) Specific Gravity (see IS:2720 (Part III) - 1980) Natural moisture content (see IS:2720 (Part II) - 1973) Unit weight (see IS:2720 (Part III) - 1980) Consolidation test (including pre-consolidation pressure) (see IS:2720 (Part XV) - 1973) Shear strength: unconfined compression (see IS:2720 (Part X) - 1973) Triaxial Compression (see IS:2720 (Part XI) - 1971) Direct Shear Test (see IS:2720 (Part XIII) - 1986) Permeability Test (see IS:2720 (Part XVII) - 1986)

Chemical tests	Soluble salt content: Chlorides and sulphates (see IS:2720 (Part XXVII) - 1977)
	Calcium Carbonate content (if warranted) (see IS:2720 (Part XIII) - 1976)
	Organic matter content (if warranted) (see IS:2720 (Part XXVI) - 1972)
Ground water	Chemical analysis including P^H determination (see IS:2720 (Part XXVI) - 1973)
	Bacteriological analysis (if necessary)
Rock drilling	(a) Visual examination unit weight Water absorption porosity.
	(b) Petro-graphic analysis compressive strength. shear strength.

TABLE 2-12
(Clause 3.1.2 of IS:1892-1979)
CURRENT METHODS OF SUBSOIL EXPLORATION

A. RECONNAISSANCE METHODS
I) Geophysical

METHOD	MODE OF OPERATION	TYPE OF FORMATION
1) Electrical resistivity method (ac or dc)	Measurement of variations in the apparent resistivity as measured on the ground	Alluvial deposits weathered and fissured rock, buried channels and ground water
2) Seismic refraction method	Measurement of velocities of compressional waves from the travel time curves of seismic waves	do
	II) Sounding	
3) a) Standard penetration test (see IS:2131-1963*)	Variations in the stratification is correlated with the number of blows required for unit penetration of standard penetrometer by a drive hammer.	Non-cohesive soils without boulders
b) Static cone penetrometer test (see IS:4968 (Part III) - 1976)	The cone penetrometer is advanced by pushing and the static force required for unit penetration is correlated to the engineering properties like density, bearing capacity, settlement, stratification, etc.	Primarily used in cohesive soils
c) Dynamic cone penetrometer test (see IS:4968 (Part II) - 1976)	The cone is driven by a standard hammer and the rest is as in (b) above	Primarily used in cohesive soils

B. EXPLORATORY METHODS

I) Drilling

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4) Shell and auger	Using auger for soft clays and shell for firm to stiff clays; in sand to be used with casing for lining and with bentonite; for boring at depths if more than 25m power operator winches are used	All types of soils specially soils of mixed type
5) Hand auger	The auger is power or hand operated with periodic removal of the cuttings	All soils except sands and gravels above water table
6) Simplified mud boring	Manual rotation of cutter fixed with drill rods by means of pipe wrench; simultaneously pumping of bentonite slurry by manually operating a double piston pump. Chisel and gravel trap used for hard bed, gravel and kankars	Silts and sands or mixed soils specially below water table
7) Wash boring	Light chopping, strong jetting and removal of cuttings by circulating water. Change of stratification could be guessed from the rate of progress and colour of the wash water	Soft to stiff cohesive soils and fine sand except gravel and boulders
8)Percussion drilling	Power chopping, hammering and periodic removal of the slurry with bailers. The strata could be identified from the slurry	Rocks and soils with boulders, except clay or loose sand
9) Rotary drilling	Power rotation of the coring bit which may vary from metal bits to tungsten carbide or diamond bits depending upon the hardness of formation (see IS:6926-1973 and IS:5313-1980)	Rocks, fissured rock and all soils except cobbles and boulders

10) Open tube sampler and split tube sampler	Driving standard sampler by a hammer weighing 65.0 kg through a drop of 750 mm (see IS:2131-1963)	Cohesive soils and silts
11) Double tube core barrels	Used with a rotary machine; non rotating inner barrel of swivel type slips over the sample and retains it as the outer bit advances (see IS:6926-1973)	Coarse sand and gravels; most suitable for soft rocks like shale and any weathered rock formation

C. DETAILED INVESTIGATIONS

i) Undisturbed Sampling

12) Thin walled tubes 50 to 125 mm	The tubes are jacked into a cleaned hole under a static force (see IS:2132-1972)	Soils of medium strength
13) Piston type sampler	The tubes are jacked into a cleaned hole under a static force	Clays and silts
14) Samlers with special core retainers	do	do
15) Sand sampler	The tubes are jacked into a cleaned hole under a static force (see IS:8763-1978)	Sand without boulders
16) Solidification methods	Solidification at the bottom of the sampler after jacking the sampler into soils	do
17) Open cuts and trenches	The sample is cut from the sides and bottom of a trench and sealed in a wooden box	All types of formations

	ii) Bearing Capacity Tests	
18) Plate load test (soils)	Loading a steel plate at desired elevation and measuring the settlement under each load, until a desired settlement takes place or failure occurs (see IS:1888-1971)	Clay and sandy formations
19) Load test (rocks)	Loading two dises placed diametrically opposite each other on two sides of a trench, by means of a jack and measuring the deflection near the sides	Rocks
20) Vane shear test	Advancing a four-winged vane into a fresh soil at desired elevation and measuring the torque developed in rotating the vane (see IS:4434-1978)	Soft and sensitive clays
	iii) Logging of Bore Holes by Geophysical Methods	
21) Electrical Logging	Measuring the potential and resistances of formation by an electrode system at various elevations	-
22) Neutron Logging	Measuring the intensity of scattered radiation from a system at desired elevation	-
23) Gamma ray Logging.	Measuring the intensity of scattered gamma radiation from a system at desired elevation.	-

2.4.1 OUTLINE OF SEISMIC AND ELECTRICAL RESISTIVITY METHODS

The various velocities for different materials is given below as a guide: for Seismic Method.

Materials	Velocity, m/s
Sand and top soil	180 to 365
Sandy clay	365 to 580
Gravel	490 to 790
Glacial fill	550 to 2135
Rock talus	400 to 760
Water in loose materials	1400 to 1830
Shale	790 to 3350
Sandstone	915 to 2740
Granite	3050 to 6100
Limestone	1830 to 6100

2.5.0 FIELD TESTS TO MEASURE PROPERTIES OF SOIL

VERTICAL LOADING TESTS

Loading tests may be used to determine whether the proposed loadings on foundations and subgrades are within safe limits, and subject to certain limitations, to assess the likely settlement of a structure. The greater, the uniformity of the strata tested, the more reliance may be placed on the results obtained.

Table gives guidance regarding the methods of estimating bearing capacity and settlement of structures for various types of soils.

METHODS OF ESTIMATION OF BEARING CAPACITY AND SETTLEMENT

S.N	Type of Strata	Methods of Estimation: Ultimate bearing Capacity	Methods of Estimation: Settlement of Structures
1)	a) Hard rocks	L	L
	b) Soft rocks, such as shales, weak limestones and sand stones	FL	L
	c) Non-cohesive soils	FL	F
	d) Soft compressible soils	LF	LF
	e) Stiff, fissured clays	LF	LF
2)	Soft, compressible stratum overlying hard stratum	LF	L
3)	Hard stratum overlying compressible stratum	LF*	L
4)	Very variable strata varying in type, thickness and arrangement	Each case to be dealt with on its merits.	

* Tests should be made on each stratum.

Note : Methods are given in order of preference :

F = Field Load Test

L = Laboratory te sts : Compression and shear tests on undisturbed samples. Consolidation test on undisturbed samples. Elastic modules tests on undisturbed samples.

2.6.0 FOUNDATIONS ON BLACK COTTON SOIL

1. The exact depth where cracks in Black Cotton Soil disappear can be ascertained only by proper observation in driest months by excavating trail pits. In other period trail pits should be excavated and left for some time to sun to obtain the dry weather conditons and the depth of cracks carefully observed. The water table during driest months should also be observed. The bottom of concrete should be taken to 2 ft. O" below this point where cracks in soil disappear. The concrete should rest on a sand cushion of atleast 2'-O" depth.

2. The sides of the foundation trench should be sloped of consistent with the capacity of the soil to retain slopes. Refilling foundations should never be done by the excavated Black Cotton soil but with sand right to the ground level outside. No part of the foundation concrete or masonry should be allowed to be exposed to Black Cotton Soil to avoid lateral pressures on the masonry due to shrinkage and expansion of side earth.

3. A sloping ramp 10'-0" wide all-round the building with the impervious pavement should be laid and surface finished with asphalt so that this forms an effective seal against rain water soaking into the foundations and thus eliminating a possible cause for differential shrinkage of soils under foundations.

4. Inside the building all filling should be done with sand or other non-active material (whichever is cheaper) upto the cracking depth and not with the excavated soil, so that flooring is laid on non-active soil filling and not on Black Cotton Soil base.

All these operations should be done in dry weather well ahead of monsoons and the aim should be that the foundations are fully loaded before the onset of monsoons.

1. PENETROMETER TEST : It consists of driving a sampling spoon of 2 in, diameter with a 140 lb weight falling through 30 in and noting the number of blows required for 1 ft penetration. Generally if the number of blows is less than ten, the soil is considered to be in a loose state.

2. THE LOAD TEST: For this a test pit is dug 5 to 6 ft. square to the desired elevation, and at the bottom of the pit a bearing plate 1 ft square is placed and loaded in increments of 200 lb till 1 1/2 times the estimated allowable soil pressure is reached. An average value of six or more tests is taken. The allowable bearing pressure if the soil is sandy is taken as half the load corresponding to half an inch settlement. These values are halved if the water-table is near the base of the footing.

On clayey soils, the settlement is found to vary in direct proportion to the width of the footing, and the ultimate bearing capacity is dependent on the compressive strength of the soil. For rectangular footing the ultimate bearing capacity is equal to.

$$q = 2.85\ C\ (1+0.3\ B/L) + YD$$

Where q is the unconfined compressive strength of the soil and B and L are the breadth and length of the footing. An excellent treatment on the allowable pressures on soil is found in soil Mechanics in Engineering Practice, by Karl Terzaghi and Ralph B.Peck (1948).

2.7.0 COMPUTATION OF BEARING CAPACITY OF SOILS - TERZAGHI

1. a) Continuous Footings:

$$q_c = [\, C.N_c + \gamma D_f (N_q - 1) + 0.5\ \gamma B N_\gamma \,] / F$$

Where q = safe bearing capacity.
C = cohesion;
γ = unit weight of the soil;
D_f = the distance from the level of the ground surface to the base of the footing (depth of foundation)
B = width of footing;
F = factor of safety; and
$N_c\ N_q\ N_\gamma$ = non-dimensional bearing capacity factors depending on the angle of internal friction

Values of $N_c\ N_q\ N_\gamma$ may be obtained from curves in the annexed Fig.

b) Square Footing:

Safe bearing capacity

$$q_q = [\, 1/F 1.3\ .C.N_q + D_f (N_q - 1) + 0.4\ \gamma B N_R \,] + \gamma_d$$

Where

B = Side of square

c) Circular Footing:

Safe bearing capacity $q_\gamma = [\, (1/F)*1.3\ .C.N_q + p(N_q - 1) + 0.6\, \gamma B N_\gamma \,] + \gamma_d$

B = Radius of Footing

2. Fairly Loose or Soft Soil (Local Shear Failure) — In the case of fairly loose or soft soil

the values of C' and ϕ' be obtained arbitrarily as follows:

$$C' = 2/3\ C$$

$$\tan \phi' = 2/3 \tan \phi$$

The $N_c\ N_q\ N_\gamma$ values corresponding to C' and ϕ' can be obtained from firm curves of Fig or alternatively C and ϕ values can be directly used to get $N_c'\ N_q'\ N_\gamma'$ from the dotted curves of Fig. In either case, the bearing capacity can be obtained by using the equations given in 1.

3. Coarse Grained Cohesionless Soils, Sands and Gravels

In this case C = 0

therefore,

a) for strip footing $q_c = (\gamma / F)[\, D_f (N_q - 1) + 0.5\ \ B N_\gamma \,] + \gamma_d$

b) for square footing $q_s = (\gamma / F)[\, D_f (N_q - 1) + 0.4\ \ B N_\gamma \,] + \gamma_d$

c) for circular footing $q_\gamma = (\gamma / F)[\, D_f (N_q - 1) + 0.6\ \ B N_\gamma \,] + \gamma_d$

4. Fine Grained Cohesive Soils

In this case ϕ is nearly equal to zero and therefore.

$$N_c = 5.7 \text{ and } N_q = 1$$

a) for strip footing = qc $5.7C/F + \gamma_d$

b) for circular or square footing q_r or $q_s = 7.4C/F + \gamma_d$

c) for rectangular footing $q = 5.7C/F\, [1 + 0.3\, B/L] + \gamma_d$

where,

L is length of footing.

2.8.0 EARTH PRESSURE

COULOMB'S THEORY

According to Coulomb's Theory, earth Pressure Pa is given by the formula:

$$Pa = \frac{\sin^2(\alpha-\phi)}{\sin^2\alpha \sin(\alpha+\delta) \quad [1 - \sqrt{(\sin(\phi+\delta)\sin(\phi-\beta)/\sin(\alpha+\delta)\sin(\alpha+\beta))}]^2} \; wh^2/2 = kwh^2/2$$

Where, α = Angle of the back wall,

ϕ = Angle of friction of the back fill,

δ= Angle of wall friction between back fill and wall,

β = Angle of surcharge or the angle of the stoping fill,

h = Height of back fill,

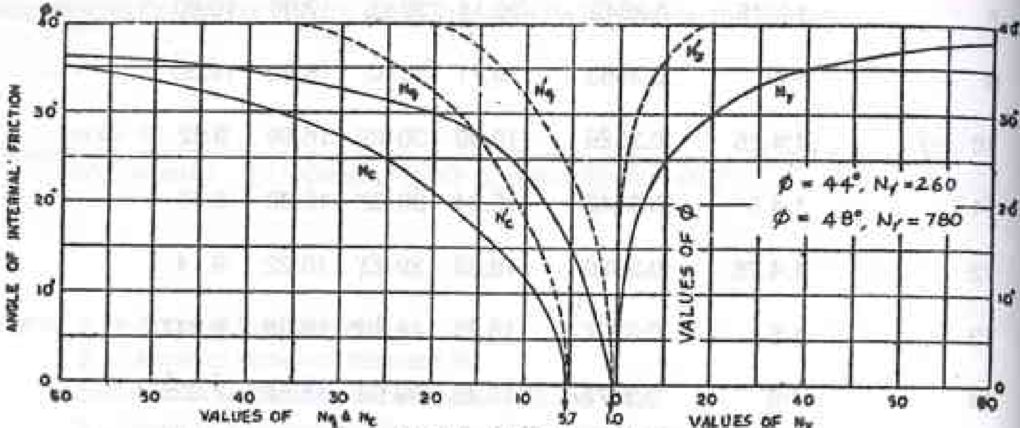
W = Unit weight of back fill

Note: For Values "K", see the table below:

COULOMB'S THEORY - PRESSURE CONSTANTS

S.No.	Rear batter	k	P/h^2	a	PH/h^2	PV/h^2	Remarks
1	1:2	0.5019	27.61	44 4'	19.82	19.21	Assumed values ϕ = 35°, δ = ϕ/2 = 17°30' β = 0 w = 110 lbs / cft Notations : δ = inclination to the horizontal of the resultant P (if inclined Ph = PCosa Pv = Psina
2	1:2.25	0.4674	25.71	41 28'	19.26	17.02	
3	1:2.5	0.4402	24.21	39 8'	18.74	15.34	
4	1:2.75	0.4183	23.02	37 24'	18.28	13.98	
5	1:3	0.4024	22.13	35 56'	17.92	12.99	
6	1:3.25	0.3884	21.36	34 36'	17.58	12.13	
7	1:3.5	0.3770	20.74	33 27'	17.30	11.43	
8	1:3.75	0.3642	20.14	32 26'	17.00	10.80	
9	1:4	0.3583	19.71	31 32'	16.80	10.30	
10	1:4.25	0.3526	19.39	30 45'	16.69	9.92	
11	1:4.5	0.3446	18.94	30 02'	16.36	9.46	
12	1:4.75	0.3386	18.62	29 23'	16.22	9.14	
13	1:5	0.3337	18.35	28 49'	16.08	8.847	
14	1:6	0.3177	17.48	26 58'	15.58	7.925	
15	1:10	0.2875	15.81	23 13'	14.53	6.233	
16	1:12	0.2803	15.42	22 16'	14.27	5.842	
17	Zero(Vertical)	0.2460	13.53	17 30'	12.91	4.069	
18	1:12(Reverse)	0.2124	11.69	12 48'	11.40	2.58	
19	1:10(Reverse)	0.2088	11.49	11 45'	11.25	2.34	

Structures designed to retain earth fills shall be proportioned to withstand pressure calculated in accordance with any rational theory. Coulomb's theory shall be acceptable subject to the modification that the centre of pressure exerted by the back fill when considered dry is located at an elevation of 0.42 of the height of the wall above the base instead of 0.33 of the height. No structure shall, however, be designed to withstand a pressure less than that exerted by a fluid weighing 30 lb/cft.



BEARING CAPACITY FACTORS

Bearing capacity Factors as given by Terzaghi
Fig. 12.16,

1). Circular footings : $q_u = 1.3CN_c + 0.6\gamma DN_\gamma + \gamma D_f(N_q - 1)$

2). Square footings : $q_u = 1.3CN_c + 0.4\gamma B.N_\gamma + D_f(N_q - 1)$

3). Strip footings : $q_u = CN + \frac{1}{2}\gamma.B.N_\gamma + D_f(N_q - 1)$

Table 2-13

SAFE LOAD FOR VERTICAL UNDER - REAMED PILES IN SANDY AND CLAYEY SOILS INCLUDING BLACK COTTON SOILS
[Clauses 6.3, 6.3.1, 6.6 and 6.3.2.2 IS 2911 (Part III) - 1973]

REINFORCEMENT					SAFE LOADING									
					Bearing Resistance				Up lift Resistance				Lateral Resistance	
Diameter of Pile	Under reamed Diameter	Longitudinal Reinforcement		Ring spacing of 6mm Diameter Rings	Single under Reamed	Double under Reamed	Increase per 30 cm. Length	Decrease per 30 cm. Length	Single under Reamed	Double under Reamed	Increase per 30 cm. Length	Decrease per 30 cm. Length	Single under Reamed	Double under Reamed
		No. of bars	Dia											
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
cm	cm	No.	mm	cm	t	t	t	t	t	t	t	t	t	t
20	50	3	10	18	8	12	0.9	0.7	4	6	0.65	0.55	1.0	1.2
25	62.5	4	10	22	12	18	1.15	0.9	6	9	0.85	0.70	1.5	1.8
30	75	4	12	25	16	24	1.4	1.1	8	12	1.05	0.85	2.0	2.4
37.5	94	5	12	30	24	36	1.8	1.4	12	18	1.35	1.10	3.0	3.6
40	100	6	12	30	28	42	1.9	1.5	14	21	1.45	1.15	3.4	4.0
45	112.5	7	12	30	35	52.5	2.15	1.7	17.5	25.75	1.50	1.30	4.0	4.5
50	125	9	12	30	42	63	2.4	1.9	21	31.5	1.80	1.45	4.5	5.4

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Note: 1. The value of bearing resistance, uplift resistance and lateral thrust given in the table are for a minimum pile length of 3.5 m. except in double under-reamed piles. In double under-reamed piles the minimum recommended lengths for 37.5m, 40 cm, 45 cm, and 50 cm, piles will normally be 3.75 m, 4.0 m, 4.5 m, and 5.0 m respectively so as to suitably accommodate the bulbs at specified distance.

2. Langitudinal bars should normally be provided with a clear cover of 4 cm and may be curtailed or eliminated towards the toe depending upon the stresses in pile section.

3. For under-reamed piles subjected to a pull and/or lateral thrust, the requisite amount of steel should be provided.

4. Values given in column 14 & 15 for lateral thrusts may not be reduced for changes in pile lengths and are fairly conservative. Higher values may be adopted after conducting lateral load tests on single or group of piles.

5. In 25 and 30 cm dia. normal under-reamed piles when concreting is done by a tremie, equivalent reinforcement in shape of single iron piece placed centrally may be used.

6. When a pile designed for a certain safe load is found to be just short of the load required to be carried by it, an overload of 10 percent should be allowed on it.

7. For working out the safe load for a group of piles the safe load on individual piles is multiplied with the number of piles in the group.

8. Only 75 pecent of the above safe loads should be taken for piles in which the bore holes are full of subsoil water during concreting. When water is confined to the bottom portion only, no such reduction need be made. For boerd compaction piles safe loads upto 85 percent of the values given in the table may be taken before applying the multiplying factor specified in 6.6

9. In sandy soils when boring and under-reaming under water, minimum size of pile recommended is 25 cm.

10. In multi under-reamed piles the depth of the centre of upper bulb below ground level shall be kept at a minimum of two times the diameter of the under-ream bulb.

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11. The values given should be increased by 50 percent for broken wire condition in the design of transmission line tower fottings.

12. Safe loads for multi under-reamed piles may be worked out from the table by allowing 50 percent of the loads as per col.6 for each additional bulb. Increase in capacity due to increase in **length** will be as per col.8.

13. For taking very high loads, the pile shaft above the top most under-ream should be either increased in diameter and/or additional reinforcement provided as in short column.

DIFFERENTIAL FREE SWELL TEST

Two samples of the dried soil weighing 10 g each and passing through 425 micron sieve should be taken. One sample should be put in a 50 ml graduated glass cylinder having kerosene oil (a nonpolar liquid). The other sample should be similarly put in the cylinder having water (preferably distilled water). Both the samples should be left undistribed for 24 hours and then their volume noted.

The differential free swell, DFS, shall be expressed as,

DFS = Soil volume in water
Soil volume in Kerosene /
Soil Volume in

The degree for expansiveness and consequent damage to the structures with light loading may be qualitatively judged by the following table.

Degree of expansiveness	**DFS percent**
Low	< 20
Moderate	20 to 35
High	35 to 50
Very High	> 50

In areas of soil showing high or very high DFS, concentional shallow foundations will not be adequate.

Reference: Hand book on under-reamed and Bored Compaction Pile Foundations CBRI, Roorkee.

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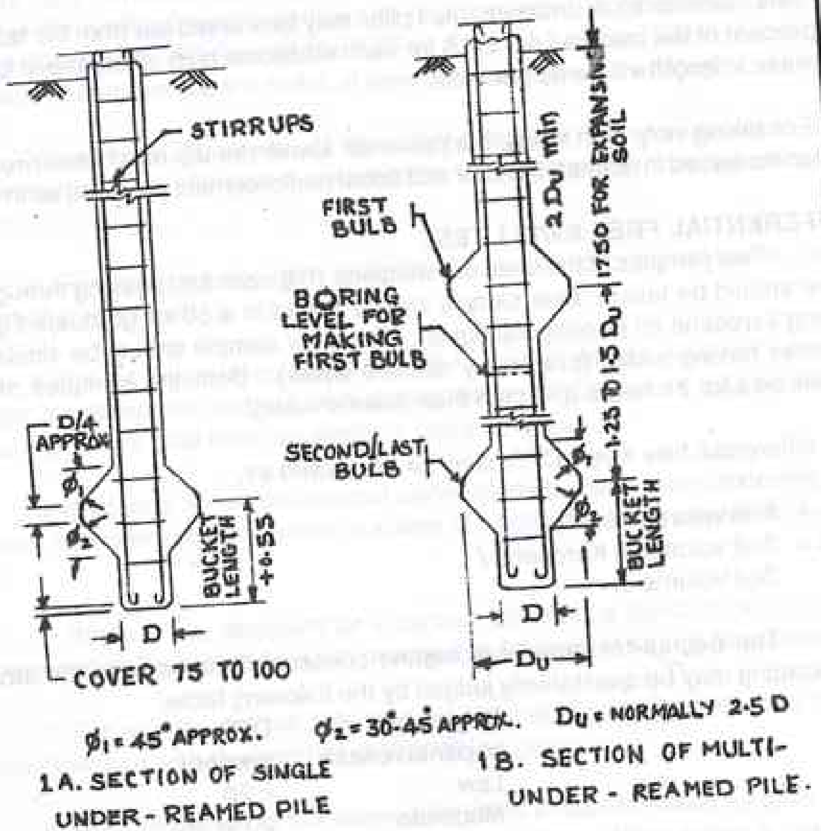

ALL DIMENSIONS ARE IN MILLIMETRES

FIG. 2.4: TYPICAL DETAILS OF BORED CAST IN SITU UNDER-REAMED PILE FOUNDATIONS.

Table 2-14

SAFE BEARING PRESSURES ON CONSTRUCTIONAL MATERIALS

Constructional material				Max. bearing pressure under uniform load		Max. bearing pressure under eccentric load =1.25x that under uniform load Max. bearing pressure under concentrated load = 1.5x that under uniform load
Description		28-day cube N/mm²	strength lb/in²	MN/m²	lb/in²	
Plain concrete	1:4:8	8.6	1,250	1.7	250	If slenderness ration K≤6 use tabulated values of max. bearing pressure directly. If 6≤k≤27, multiply tabulated value by a=1:165-0.275k+0.06e (6-k) where e=eccentricity of vertical loading as proportion of thickness. (Expression valid only when a ≮0.2) For member having a cross-sectional area A of less than 0.3m² or 500in², the max. bearing pressure calculated must be multiplied by the reduction factor (9+10A) 12 (metric units) or (1,500+A)/2,000 (imperial units)
	1:3:6	11.5	1,650	2.4	350	
	1:2:4	21.0	3,000	5.3	760	
	1:1½:3	25.5	3,750	6.5	950	
	1:1:2	30.0	4,500	7.6	1,140	
Masonry brickwork (unreinforced) with 1:3 cement mortar		20.5	3,000	1.65	240	If slenderness ratio k < 15, use tabulated value of max bearing pressure directly If 15≤k≤24, multiply tabulated value by (45-k)/30
		27.5	4,000	2.05	300	
		34.5	5,000	2.50	360	
		52.0	7,500	3.50	510	
		69.0	10,000	4.55	660	
		96.5 or more	14,000 or more	5.85	850	

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TABLE - 2.15

PILE CAP DEPTH:

Pile caps provide the medium through which the load is transmitted from the column to the piles. They are constructed deep, usually not less than 630mm and with general recommendation that the depth be related to the pile size used, as detailed below:

Pile Size (mm)	300	350	400	450	500	550	600	750
Pile Cap Depth (mm)	700	800	900	1000	1100	1200	1400	1800

Ref : Reinforced Concrete foundation by Bell *P 256*

TABLE 2.16

SAFE BEARING CAPACITY

Sl. No.	Type of Rocks/Soils	Safe Bearing capacity KN/M (kgf/fms)	Tonnes	Remarks
1.	2.	3.		4.
	a) Rocks			
i)	Rocks-hard without lamination and defects, for example, granite, trap and diorite.	3,240 (33)	330	—
ii)	Laminated rocks for example, sandstone and lime-stone in sound conditon	1,620(16.5)	165	—
iii)	Residual deposits of shattered and broken bed rock and hard shale, cemented material	880 (9.0)	90	—
iv)	Soft Rock	440 (4.5)	45	—
	(b) Non-cohesive Soils			
v)	Gravel, sand and gravel compect and offering high resistance to penetration when excavated by tools.	440 (4.5)	45	(See Note 2)
vi)	Coarse sand, compact and dry	440 (4.5)	45	Dry means that the ground water level is at a depth not less than the width of foundation below the base of the foundation.
vii)	Medium sand, compact and dry	245 (2.5)	25	—
viii)	Fine sand, silt (dry lumps easily pulverised by the fingers)	150 (1.5)	15	—
ix)	Loose gravel or sand gravel mixture loose coarse to medium sand, dry	245 (2.5)	25	(See Note 2)
x)	Fine sand, loose and dry	100 (1.0)	10	—
	(c) Cohisive Soils			
xi)	Soft shale, hard or stiff clay in deep bed, dry	440 (4.5)	45	This group is susceptible to long term consolidation settlement.
xii)	Medium clay, readily indented with a thumb nail	245 (2.5)	25	—

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xiii)	Moist clay and sand clay, mixture which can be indented with strong thumb pressure	150 (1.5)	15	—
xiv)	Sift clay indented with moderate thumb pressure	100 (1.0)	10	—
xv)	Very soft clay which can be penetrated several inches with the thumb	50 (0.5)	5	—
xvi)	Black cotton soil or other shrinkable or expansive clay in dry condition (50 percent saturation) d) Made-up Ground			See Note 3. To be determined after investigation.
xvii)	Fills or made-up ground		-	See Note 2 and Note 4. To be determined after investigation

Note:-

1. Values listed in the table are from shear consideration only.
2. Values are very much rough due to the following reasons:
   a) Effect of characteistics of foundations (that is, effect of depth, width, shape, roughness, etc.) has not been considered
   b) Effect of range of soil properties (that is, angle of frictional resistance, cohesion, water table, density, etc) has not been considered.
   c) Effect of eccenticity and inclination of loads has not been considered.
3. For non-cohesive soils, the values listed in the table shall be reduced by 50 percent if the water table is above on near the base of footing.
4. Compactness or looseness of non-cohesive soils may be determined by driving the cone of 65 mm dia and 60 apex angle by a hammer of 65 kg falling from 75 cm. If corrected number of blows (N) (see IS:6403-1971*) for 30 cm penetration are less than 10, the soil is called loose, if N lies between 10 and 30, it is medium, if more than 30, the soils is called as dense.

* Code of practice for determination of allowable bearing pressure on shallow foundations.

TABLE 2.17

SAFE BEARING CAPACITY AND FOUNDATION BASED ON N-VALUE

1. Penetration Resistances and Emprical Correlation for Non-cohesive Soil

Penetration Resistance N (Blows)	Approx. (degree)	Density Index (%)	Description	Approximate moist density (t/m)
—	25.30	0	Very Loose	1.12-1.60
4	27.32	15	Loose	1.44-1.84
10	30.35	35	Medium	1.76-2.08
30	35.40	65	Dense	1.76-2.24
50	38.43	100	Very dense	2.08-2.40

2. Penetration Resistance and Empirical Correlation for Cohesive Soil.

Penetration resistance (Blows)	Unconfined compressive strength (t/m)	Saturated density	Consistency
0	0	—	Very soft
2	2.5	1.60-1.92	Soft
4	5		Medium
8	10	1.76-2.08	Stiff
16	20		Very stiff
32	40	1.92-2.24	Hard

3. Type of Soil and Proper Soil Test to be done

Type of Test Class	Lab. Test for C-φ Mechanical Analysis	Standard Penetration Test	Load Test	Static Class Resistance Test
Type of soil for which the test is recommended for good results	C-φ Soil when undisturbed sample can be taken	Sandy soils	Gravelly Soil and Sandy Soils	Fine sand with varying density
Type of soil for which the test is not recom-mended for better results	Gravelly Soil	Gravelly soil and fine sand	Clayey soils	Gravelly soils and Clayey soils
Type of soil **particularly** suited for the test	C-φ Soil	Sandy soil	Gravelly and sandy soil	Fine sand with varying density

TABLE - 2.18

Approximate weights of soils and Angles of Repose

Material		Angle of Repose Degrees	Weight of soil in lbs/cft	Kgs/m
1. Clay	Wet	15-20	140-160	2240-2560
	Damp, Well drained	30-45	125-160	2000-2560
	Dry	25-30	110-130	1760-2080

2. Sand	Wet	15-30	110-125	1760-2000
	Moist	30-45	100	1600
	Dry	25-35	90-100	1440-1600
3. Earth Loose		30-45	100-110	1600-1760
	Rivermud	50-65	120	1920
	Dry	30-40	110	1760
4. Sandy Clay	Wet	18-20	125	2000
5. Gravel	Dry	40-45	90	1440
	Wet	27	125	2000
	Gravelly clay wet	18	127	2000
	Gravelly sand and clay wet	19	130	2080
	Gravelly sand	25-30	100-110	1600-1760
6. Shingle	Dry	30-40	90	1440
	Shingle or earth moist	40	140	2240
7. Organic soils	Dry	30	90-100	1440-1600
	Moist	45-50	100-110	1600-1760
	Very wet	17	110-120	1760-1920

8. Silt	Wet	10-20	110	1760
	Dry	20	100	1600

Minimum Depth of foundations: $d = \dfrac{p(1-\sin\phi)^2}{w(1+\sin\phi)}$

d = Depth in feet or meter: P=intensity of pressure in lbs/sft or kg/m^2
W = Weight of soil in lbs/cft or Kgs/m^3 0= Angle of repose in degrees.

TABLE 2.19
Coefficients used in Rankines Earth pressure calculations

ϕ in degrees	$\frac{1-\sin\phi}{1+\sin\phi}$	$\frac{(1-\sin\phi)^2}{(1+\sin\phi)}$	$\frac{1+\sin\phi}{1-\sin\phi}$	$\frac{(1+\sin\phi)^2}{(1-\sin\phi)}$
15	0.59	0.35	1.70	2.88
17	0.55	0.30	1.82	3.33
18	0.53	0.28	1.89	3.59
20	0.49	0.24	2.04	4.16
22.5	0.45	0.20	2.24	5.02
25	0.41	0.16	2.46	6.07
26	0.39	0.15	2.56	6.56
28	0.36	0.13	2.77	7.67
29	0.35	0.12	2.88	8.30
30	0.33	0.11	3.00	9.00
31	0.32	0.10	3.12	9.76
32	0.31	0.09	3.25	10.59
33	0.29	0.09	3.40	11.50
34	0.28	0.68	3.83	12.57
35	0.27	0.07	3.69	13.62
40	0.22	0.05	4.60	21.16
45	0.17	0.03	5.83	33.94
50	0.13	0.02	7.54	56.85

Table 2-20

Rankines Coefficients

Slope	Angle (Degrees) ϕ	$\frac{1-\sin\phi}{1+\sin\phi}$ (Active state)	$\frac{1+\sin\phi}{1-\sin\phi}$ (Passive state)
1:6	8°	0.84	1.19
	10°	0.704	1.42
1:5	11° 20'	0.672	1.49
1:4	14°	0.610	1.64
	15°	0.589	1.70
1:3	18° 30'	0.518	1.93
	20°	0.490	2.04
1:2	26° 30'	0.383	2.61
	30°	0.333	3.00
1:1½	33° 40'	0.287	3.49
	40°	0.217	4.60
1:1	45°	0.172	5.83

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Table 2-21

Soil Particle Fractions

Designation of soil fractions	Diameter of Soil particle in mm: U.S.Dept of Agriculture Bureau of soils.	American Society for testing materials	International society of soil science
	1951	1958	
1. Fine gravel	2-1	2.0	
2. Coarse sand	1-0.5	2-0.42	2 - 0.2
3. Medium sand	0.5-0.25	--	--
4. Fine sand	0.25-0.10	0.42-0.74	0.2 - 0.2
5. Very fine sand	0.10-0.05	--	--
6. Silt	0.05-0.002	0.074.0-0.005	0.024 - 0.002
7. Clay	0.002	0.005	0.002
8. Colloids	--	0.0001	--

Wall Friction and Adhesion

The angle of friction between the back of a wall and the backfill is often assumed to $2/3\phi_\lambda$ or between $1/2\phi_\lambda$ and $3/4\phi_\lambda$. In general it is suggested that the angle be assumed to be equal to $2/3\phi_\lambda$ and that wall adhesion be assumed to be equal to 2/3 C for retaining wall design where ϕ_λ = angle if internal friction of the backfill soil and c = unit cohesive strength.

Ref : earth Pressures and Retaining walls- Whitnfy Clark Huntington

Table 2-22

Co-efficient of friction of concrete on soils (average values)

Type of soil		Co-efficient of friction
Dry Clay Gravel Loam or Sand	--- --- --	0.5 to 0.6
Moist Clay or Moist Sand	--- --- --	0.33
Wet Clay	--- --- --	0.6 to 0.7
Dry rock	--- --- --	0.6 to 0.7
Wet rock	-- -- --	0.5

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2.9.0 Well Foundations

Thickness of staining ≮ 450 mm (18 in) or less than that given by the following equation

i) For circular shaped wells of cement concrete :

1. Thickness for wells in sandy strata = 1.0 [(H/100) + (D/10)]
2. Thickness for wells in soft clay strata = 1.1 [(H/100) + (D/10)]
3. Thickness for wells in Hard clay strata = 1.25 [(H/100) + (D/10)]
4. Thickness for wells in strata where boulders, kankar, shale or laterite or such hard materials are met with = 1.25 [(H/100) + (D/8)]

where D = external dia of the well.

H = full depth to which the well is designed to be sunk below the bed.

Reinforcement in concrete well stening :

Vertical bond rods in the steining ≮ 9 kg per m³ (56 ib/100 cft.). This should be provided on both faces of the well adequately tied up with hoop steel not less than 3.25 kg/m³ (20 ib/100 cft.). The cover over the bond rods shall not be less than 75 mm (3 in.) well curb : Minimum reinforcement shall be 72 kg/m³ 4.5 ib/cft.).

DETERMINATION OF THE MAXIMUM DEPTH OF SCOUR

(as per clause 110.1.2 of I.R.C. code-section. i)

The following theoretical method may be used when dealing with natural channels in regime in alluvial beds and where the effective linear waterway provided is not less than the regime width $d = 1.34\,(Q^2/f)^{1/3}$.

Where d = normal depth of scour in metres below the flood level corresponding to the value of Q adopted.

Q = The discharge adopted for the design, in cubic metre per second; **per meter width**

f = The silt factor for a representative sample of the bed material;

$= 1.76\sqrt{m}$ where, m is the mean dia in millimetre.

The values of 'f' normally recommended for various grades are as under :

Type of bed material to which applicable		Mean dia of particle in mm.	Value of 'f'
Fine silt	..	0.081	0.500
Fine silt	..	0.120	0.600
Fine silt	..	0.158	0.700
Medium silt	..	0.233	0.850
Standard silt	..	0.323	1.000
Medium sand	..	0.505	1.250
Coarse sand	..	0.725	1.500
Fine bajri and sand	..	0.988	1.750
Heavy sand	..	1.290	2.000

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Maximum scour depth below H.F.L shall be taken as follows :

1.	In a straight reach	=	1.27 X	[Normal depth of scour]
2.	At a moderate bend	=	1.50 X	[do]
3.	At a serve bend	=	1.75 X	[do]
4.	At a right angled bend	=	2.00 X	[do]

When the bridge causes contraction i.e. $L < W$

The normal scour depth $D' = D\,(W/L)^{0.6}$ (metric)

Where L = Linear waterway of the bridge and W is the regime width in the case of alluvial streams and unobstructed natural width in the case of quasialluvial streams.

RULES FOR FINDING MAXIMUM SCOUR DEPTH

Rule (1) : For average conditions on a straight reach of stream and when the bridge is a single span structure, i.e, it has no piers obstructing the flow, the maximum scour depth should be taken as 1.5 times the normal scour depth, modified for the effect of contraction where necessary. ($L = 0.65\,W$)

Rule (2) : For bad sites on curves or where diagonal currents exist or the bridge is a multi-span structure, the maximum scour depth should be taken as 2 times, the normal scour depth modified for the effect of contraction where necessary. ($L\_ 0.47\,W < 0.67\,W$)

Rule (3) : For bridges causing contraction, scour depth obtained by rule (1) or (2), should be compared with that given by the following equation and the greater of the two values adopted.

$D_m = D\,(W/L)^{1.56}$ (Fps)

Where D_m = the maximum scour depth and D,L,W being as defined above.

2.10.0 Scour Depth Calculations

Lacey's water way L : 4.75 (Q) 1/2 (Metric)

1) <u>When Actual water way is less than the Legace's water way L, as above the maximum scour depth.</u>

$R = \; : \; 1.346\,(q^2/f)^{1/3}$ (metric)

R : Probable normal scour depth in metres

q : Intensity of discharge/in cumecs per metric

f : Lacey's slit factor = $1.76\sqrt{m}$

: md : mean dia of spheres : oblate spherical particles of bed in mm particles of bed in mm.

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<u>Factor of safety or scour factors generally adopted</u>

U/s of pucca of piers 1.50 R

D/s of pucca floor of piers 2.0 R

Guide Bunds

Noses	2.25 R
Transition from nose to straight	1.50 R
Straight reaches of guide banks	1.25 R

Type of materials		Grain size in mm	f.silt factor
Silt	Fine	0.12	0.60
	Medium	0.233	0.85
	Standard	0.323	1.00
Sand	Medium	0.505	1.25
	Coarse	0.725	1.50
Bajra	Fine	0.988	1.75
	Medium	1.29	2.00
	Coarse	2.422	2.75
Gravel	Medium	7.28	4.7
	Heavy	26.1	9.0
Boulders	Small	50.1	12.00
	Medium	72.1	15.00
	Large	188.8	24.00

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Depth of foundations : Df :

Rule 1 : Erodible beds : HFL $1^{1/3}$ calculated maximum scour depth subject to a minimum of 1.2 cm below scour fine (like sand)

Rule 2 :Hard beds : (Solid rocks etc.,) : The foundation shall be securely anchored into that material ie., 0.30m or so into rock and 0.6 m or so into other hard material.

Rule 3 : All beds : The pressure on foundations must be well within the safe bearing capacity of the materials. In case of small culverts (Structures) keep the top of floor at about 0.30m below bed level; take abutment foundations 1.2m deep below the top of floor level. U/s and D/s curtain walls 0.9m to 1.5 m and 1.5m to 2.4m deep from top of floor depending on the velocity of flow, through structure and erodibility of bed material.

(in the absence of scour depth. data)

ii.When actual water way is not equal to or more than the Legacey's linear water, L

$$R = 0.4774\ [Q/f]^{1/3}\ (\text{Metric})$$

Where Q = Total discharge in cumecs.
R : H.M.D or normal scour depth in meters.

Earth canal : Approximate HMD 0.75 Q; 0.25 (F.P.S.)

Transmission losses in canals in cusecs per millionsquare feet

Decayed rock Gravel	10
Alluvial or red soil	8
Black cotton soil	5
Rock	3

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2.11.0 IMPERVIOUS ZONE AND CUT OFF TRENCH

Whenever the depth of water upto F.T.L. in a particular cross section of the bund exceeds 3m then impervious zone should be invariably proposed. The top of the impervious zone should be 0.3m above the maximum water level and the top width of that will be 2/3 of the top width of the bund.

The side slopes will be 1/2 to 1 (1/2 horizontal to 1 vertical) and hence the bottom width of the impervious zone will be the height of Impervious zone plus the top width of the impervious zone plus th top width of the impervious zone. The depth of the cut off trench will be half the height between the average ground level and the F.T.L. The bottom width of cut off trench will be 3m and its outer edges will be connected with that of impervious zone.

KEY TRENCHES:

Key trenches will be provided in the upstream half of the bund at 4.5m centre to centre from the upstream outer edge of the impervious zone. In the cross sections where there is no IPZ key trenches should invariably be provided throughout the bottom width of the bund to have proper bond of the bund with the earth below. The size of teh trenches will invariably be a trapezium in shape having a bottom width of 1.5m with side slopes 1 to 1 and a depth of 0.3m from the average ground level.

HYDRAULIC GRADIENT:

The Hydraulic Gradient line will be drawn in the cross section of a bund from a point at which the FTL meets the upstream face of the bund and its slope will be and 1 in 4 in SPZ and 1 in 3 in IPZ. When the Hydraulic Gradient line cuts the bottom surface of the bund beyond 3/4th of the width of bund from the upstream edge, filters should be invariably be provided at the downstream edge of the bund.

FILTERS:

As stated in the above para graded filters of size 2.7m x 0.9m will be provided in the down stream edge of the bund, graded filters alone are provided upto 6m depth of graded filter should be provided.

TOE DRAIN:

Toe drain should invariably be provided in all cross sections of the bund in the rear side, to discharge the rain water falling over the bund to the adjacent drainage course. The bottom width will be 0.9m and the side slopes will be 1 to 1. The bottom level will be at the same level of the bottom of the filter.

TOE WALL:

Whenever revetment with gravel backing is proposed in the cross sections of the bunds, then a square toe wall of size 0.9m x 0.9m as shown in the sketch at page 277 of Ellis Manual of Irrigation (1963 Edition) should be provided at the bottom of the revetment.

Table 2-23 Predicting Performance of Embankment on Soft Soil

PREDICTED	METHOD	PARAMETERS	SELECTION OF PARAMETERS
I STABILITY DURING CONSTRUCTION (Undrained)	A TOTAL STRESS METHOD	S_u	FIELD TESTS field vane, pressure meter, static cone penetrometer UU LAB TESTS-triaxial compression, incl. unconf. compr. CU LAB TESTS-triaxial compression -tests duplicating other-stress systems e.g., plane strain, simple shear etc.
	B EFFECTIVE STRESS METHOD	$\bar{C}, \bar{\phi}$	$\overline{CU}$ & CD LAB TESTS-e.g., triaxial compression, plane strain, direct shear
		u	Requires field measurements
II INITIAL SETTLEMENT (Undrained)	A ELASTIC	E_u =0.5	EMPIRICAL CORRELATION--e.g., E_u/S_u=constant UU LAB TESTS --not recommended CU LAB TESTS--triaxial compression
	B FINITE ELEMENT	$E_u \mu$=0.5, $S_u(V), S_u(H)$	CU LAB TESTS--requires tests duplicating in situ stress systems
		K_o	FIELD TESTS--pressure meter, hydraulic fracturing LAB TESTS--K_o test
III FINAL CONSOLIDATION SETTLEMENT	A ONE-DIMENSIONAL	in situ stress history m_v	LAB OEDOMETER TESTS
	B SKEMPTON-BJERRUM (1957)	in situ stress history m_v	LAB OEDOMETER TESTS
		A	$\overline{CU}$ LAB TESTS--triaxial compression
	C LAMBE STRESS PATH (1964, 1967)	Measured vert. strain	CD LAB TESTS following typical in situ stress paths
	D FINITE ELEMENT & FINITE DIFFERENCE	$\bar{E}, \bar{\mu}$	CD LAB TESTS that should follow typical in sity stress paths
IV RATE OF CONSOLIDATION SETTLEMENT	A TERZAGHI 1-D	C_v	LAB OEDOMETER TESTS
	B FINITE ELEMENT & FINITC DIFFERENCE	$C_v, \bar{\mu}$	LAB OEDOMETER and CD stress path tests
		k_h/k_v	LAB PERMEABILITY TESTS on vertical and horizontal specimens
V LATERAL DEFORMATION (Undrained)	A ELASTIC	E_u, μ=0.5	SEE II A
	B FINITE ELEMENT	$E_u \mu$=0.5 k_o $S_u(V), S_u(H)$	SEE II B
VI LATERAL DEFORMATION DURING CONSOLIDATION	A FINITE ELEMENT	$\bar{E}, \bar{\mu}$	SEE III D
		$C_v, k_h/k_v$	SEE IV B

2.12.0 I. Design of retaining Walls :-

2.12.1 Masonry retaining walls :-

a) Termed as (i) low upto 10m (30') height, top width (2') 0.6m
(ii) High, exceeding 10m (30') height, top width 0.9m (3')
Note : For walls up to 6m (20') height-top width 0.45m.

b) Back fill on earth face adds to the weight of the wall. By keeping the entire batter on earth face stresses in the section will increase, so it is not economical proposal.

c) Front-outer face (exposed face) batter 1/6 to 1/12 or 1/4- 1/6 base width 1/3 H

d) Rear face (earth face) batter as per Rankines theory :
1/3 - 1/6 or 1/12 or 1/6 - 1/4

e) The earth pressure is horizontal and acts as 1/3 H from base

II Retaining wall - TVA procedure (based on Coloumbs) :
Dr. Kerl Terzaghi's classified experiments on " high retaining walls " detailed in Engineering News Record (1934) have indicated that the pressure behind the retaining walls, closely approximates to that given by Coloumb's wedge theory and that it acts at 0.4 H from the base and in a direction inclined to the back of the wall.

For no tension at foundations, a back batter of 1 in 3 (earthern force) may be admissible. For greater batter the over turning ratio should not exceed 1.75. Back batter as minimum as possible (earth face) say 1 in 12 (a = 4° 46')

Hydraulic structures :- Saturated fills :- i) Hydraulic pressures below the point of saturation is determined as though no fill were present.
ii) The earth pressure resulting from the buoyant weight of the fill (65 lbs/cft) about 1.08 t/m 3 = is calculated using the " same values of and "as would be used if the fill were dry (no reduction in value due to saturation is allowed.

These two forces are combined in quantity to find the moment to produce a resultant earth pressure on the walls (abutments of U.T.S, Aqueducts, syphons etc.,)

Note : Water pressure of the saturated fill acts at 0.33 h

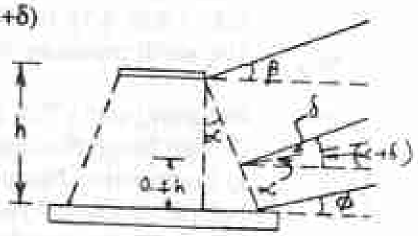
$$P = \frac{wh^2}{2} \text{Cos}\left\{\frac{(\phi - \alpha)}{(1+n)\cos\alpha}\right\}^2 \times \left\{\frac{1}{\cos(\alpha+\delta)}\right\}$$

The resolved components or $P = Kwh^2$, where are Ph and Pv which act at 0.4 h from base.

alpha = α
delta = δ
beeta = β

$$\text{Where K :} \frac{1}{2}\,\frac{\text{Cos}(\phi - \alpha)}{(1+\nu)\text{Cos}\,\alpha} \times \frac{1}{\text{Cos}(\alpha+\delta)}$$

$$\text{and } n = \frac{\text{Sin}(\phi+\delta)\,\text{Sin}(\phi-\beta)}{\text{Cos}(\alpha + d)\,\text{Cos}(\alpha-\beta)}$$



w = (105 ib/cft) for sand or gravel 1680 kg/m³
1920 kg/m³ (120 ib/cft) for finer materials like loams
2240 kg/m³ (140 lbs/cft) for clays
$PH = KH\,wh^2, = K\cos(\alpha+\delta)\,wh^2$, to 2.100 T/m³ are adopted.
$Pv = kv\,wh^2, = K\,\text{Sin}(\alpha+\delta)\,wh^2$ in designs.

Where P = pressure in kg/metre
h = height of wall in metres
w= weight of fill in kg/m³

Note : The general practice is to give stepped back for better grip on earth face and assume $\alpha = 0$ for design purposes.

Types of back fills for retaining walls :

1. Coarse grained soil without admixture of fine soil particles - permeable (clear sand and gravel)

2. Coarse grained soil of low permeability due to a mixture of particles of silt size

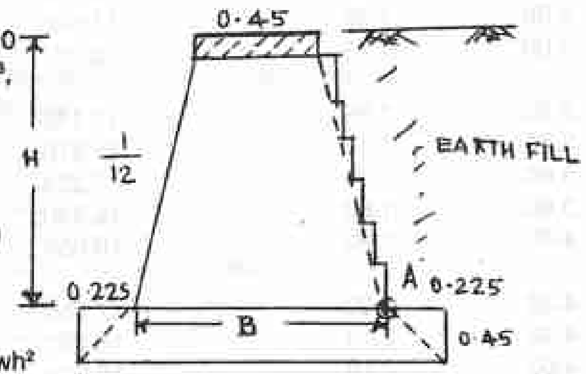
3. Residual soil with soil stones, fine silty sand and granular material with conspicuous clay content.

4. Very soft clay organic silts or silty clays (3:1 Max. Slopes)

5. Medium or stiff clay deposited in hunks and protected in such a way that a negligible amount of water enters if the conditions cannot be satisfied, the clay should not be used as back fill material as with increasing stiffness of clays, danger to the wall due to filtration of water increases rapidly (2:1 max. slopes) ..

Design Parameters (As adopted Nagarjuna Sagar Canal)

a) $\phi = 32°$, $\delta = 32/2 = 16°$, $\alpha = 0$
W : Unit weight of earth 2100 in kg/m³,
masonry concrete 2240/m³
vertical components PV.
Horizontal components PH.



i) Level surcharge (No fill above wall)

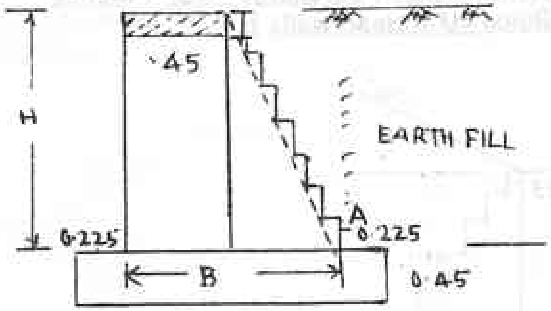
	$0.0384\ wh^2$	$0.1338\ wh^2$
ii) Surcharge Slope		
1H : 1V	$0.1031\ wh^2$	$0.3596\ wh^2$
1½H : 1V	$0.1031\ wh^2$	$0.3596\ wh^2$
2H : 1V	$0.0623\ wh^2$	$0.2174\ wh^2$
2½ H : 1V	$0.05375\ wh^2$	$0.1875\ wh^2$

Note : Allowable tension in CRS Masonry in CM (1:5) 0.70 kg/cm2 (10 psi) -
No Tension in RR Masonry is allowed -

Table **Retaining wall with 1 in 12 face better :- Level surcharge :- No Tension in Masonry/foundations - (Sagar Canals Practice)**

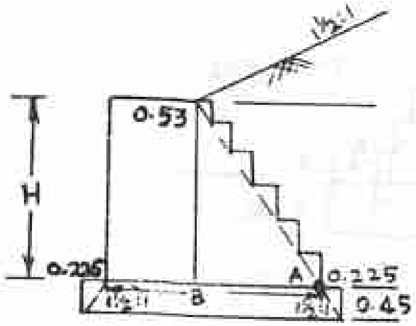
H in meters	B in meters	Comprehensive stress in t/m² Masonry	Soil
1.00	0.52	3.323	3.339
1.20	0.55	5.257	3.982
1.40	0.65	6.110	4.617
1.60	0.75	6.899	5.292
1.80	0.85	7.690	5.977
2.00	0.95	8.452	6.696
2.20	1.05	9.240	7.375
2.40	1.15	10.010	8.084
2.60	1.25	10.792	8.800
2.80	1.35	11.600	9.520
3.00	1.45	12.377	10.274
3.20	1.55	13.148	10.978
3.40	1.60	14.576	12.168
3.60	1.70	15.233	12.797
3.80	1.80	15.990	13.539
4.00	1.90	16.856	14.369
4.20	2.00	17.566	15.064
4.40	2.10	18.351	15.825
4.60	2.15	19.633	16.936
4.80	2.25	20.475	17.713
5.00	2.35	21.367	18.576
5.20	2.45	22.023	19.248
5.40	2.55	22.810	20.028
5.60	2.65	23.596	20.807
5.80	2.75	24.388	21.592
6.00	2.80	25.762	22.769

Table Retaining walls with front face vertical - (Sagar Canal Practice) level surcharge



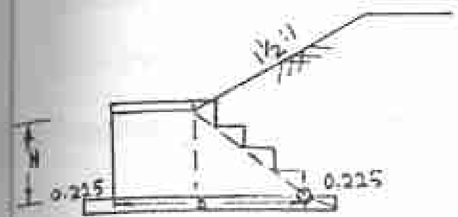
H in meters	B in meters	Comprehensive stress in t/m² Masonry	Soil
1.00	0.50	4.60	3.46
1.20	0.60	5.58	4.14
1.40	0.70	6.52	4.85
1.60	0.80	7.433	5.64
1.80	0.90	5.380	6.38
2.00	1.00	9.280	7.16
2.20	1.10	10.23	7.90
2.40	1.20	11.145	8.79
2.60	1.30	12.08	9.61
2.80	1.40	13.00	10.46
3.00	1.50	13.945	11.296
3.20	1.60	14.86	12.157
3.40	1.70	15.77	13.02
3.60	1.80	16.72	13.87
3.80	1.90	17.63	14.76
4.00	2.00	18.57	15.62

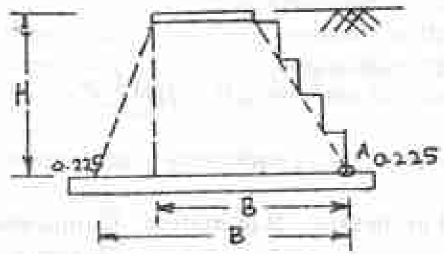
Table Front face vertical 1 1/2 : 1 surcharge - NSP Practice (Sluice - U.T. Head walls)



H in meters	B in meters	Comprehensive stress in t/m² Masonry
0.70	0.55	3.29
1.00	0.85	4.89
1.20	1.00	6.34
1.50	1.20	8.02
1.80	1.40	9.78
2.00	1.55	10.80
2.20	1.70	11.83
2.50	1.95	13.13
2.80	2.15	14.87
3.00	2.30	15.86
3.20	2.45	16.87
3.50	2.70	18.19
3.70	2.85	19.19
4.00	3.05	20.89

Table walls front face vertical - 1 ½ : 1 surcharge - NSP (Practice)





H in meters	B in meters	Comprehensive stress in t/m² Masonry	Soil
1.00	0.83	5.32	5.87
1.20	1.00	6.58	6.97
1.40	1.25	7.34	7.69
1.60	1.35	9.05	9.23
1.80	1.50	10.47	10.55
2.00	1.68	11.58	11.71
2.20	1.82	13.14	13.09
2.40	2.00	14.41	14.28
2.60	2.20	15.45	15.97
2.80	2.35	16.90	15.92
3.00	2.50	18.35	16.10

Front face vertical - Leval surcharge

Table Retaining walls front face vertical / batter level surcharge - SRSP Practice

Design parameters top width a = 6 m,
stepped back, 0 = 32 °

Weight of saturated earth	:	2080 kg/m³
Weight of masonry	:	2240 kg/m³
Weight of concrete	:	2400 kg/m³

$P_V = 0.073\ wh^2$, $PH = 0.1175H^2$, $Y = 0.4H$

Note: Back face (earth force) as stepped.

H in meters	B in meters	Comprehensive stress Masonry		in t/m² Soil
(4')	1.22	0.61 / 0.81	5.58 / 2.80	5.36
(6')	1.83	0.76 / 0.91	8.97 / 5.68	9.73
(8')	2.44	0.99 / 1.01	12.36 / 7.66	13.02
(9')	2.74	1.14 / 1.14	13.56 / 9.52	14.11
(10')	3.05	1.22 / 1.14	15.64 / 12.45	16.21
(12')	3.66	1.52 / 1.37	18.05 / 14.87	18.59
(14')	4.27	1.75 / 1.60	21.44 / 17.50	21.90
(15')	4.57	1.83 / 1.68	23.73 / 19.47	24.20

Note : Sections with front force vertical are economical upto (9' 2.74m height

Table: Retaining wall-front face batter 1 in 8 stepped back - level surcharge

Design parameters : $\phi = 32°$, $\delta = 16°$

Masonry - $PV = 0.0384\ wh^2$, $PH = 0.1338\ wh^2 = 0.4\ h$

Soil - $PV : 0.065\ wh^2$ $PH = 0.121\ wh^2$

H in meters		B in meters	Compressive stress in t/m² Masonry	Soil
(4')	1.22	0.74	3.74	6.27
(6')	1.83	0.82	7.60	9.38
(8')	2.44	1.07	8.75	9.85
(10')	3.05	1.14	13.46	14.90
(12')	3.66	1.37	16.16	17.63
(14')	4.27	1.60	18.86	20.36
(16')	4.88	1.83	21.54	23.14
(18')	5.49	2.06	24.23	25.92
(20')	6.10	2.29	26.89	28.70

Table : Retaining wall front face vertical and 2:1 surcharge - SRSP Practice -

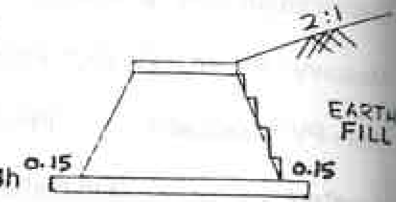
Design parameters : top width 0.6m (2') stepped back

Masonry	$PV = 0.1261\ wh^2$	$PH = 0.2017\ wh^2$
Soils	$PV = 0.1093\ wh^2$	$PH = 0.2057\ wh^2$

H in meters	B in meters	Compressive stress in t/m² Masonry	Soil
(4') 1.22	0.76	5.44	8.16
(6') 1.83	1.07	9.76	12.62
(8') 2.44	1.52	12.97	15.96
(10') 3.05	1.91	17.14	19.98

Table: Retaining walls front batter 1 in 10 - level surcharge (Returns - wings) - SRSP Practice-

Design parameters : top width 0.6m, stepped back



Unit weight of saturated earth : 2.10 t/m³
Unit weight of masonry : 2.25 t/m³
Unit weight of concrete : 2.40 t/m³
Masonry Pv = 0.073 wh² PH = 0.1175 wh², = 0.4h
Soils Pv = 0.065 wh² PH = 0.121 wh²

H in meters	B in meters	Comprehensive stress in t/m² Masonry	Soil
1.00	0.685	3.245	5.021
1.25	0.710	4.619	6.426
1.50	0.835	6.528	6.035
1.75	0.860	6.902	7.416
2.00	0.885	8.558	9.021
2.25	1.910	9.502	10.845
2.50	1.00	10.704	12.029
2.75	1.10	11.758	13.088
3.00	1.15	13.589	14.835
3.25	1.25	14.731	16.015
3.50	1.35	15.689	16.985

Table: Retaining walls- front batter 1 in 5. level surcharge stepped back (wings and returns) top width 0.6m - SRSP Practice

H in meters	B in meters	Comprehensive stress in t/m² Masonry	Soil
1.00	0.77	2.509	4.148
1.25	0.82	3.392	5.420
1.50	0.87	4.464	6.567
1.75	0.92	5.720	7.850

H in meters	B in meters	Comprehensive stress in t/m² Masonry	Soil
2.00	1.10	4.778	6.672
2.25	1.15	5.777	7.736
2.50	1.20	6.883	8.896
2.75	1.25	8.103	10.147
3.00	1.30	9.425	11.493
3.25	1.35	10.845	12.918
3.50	1.40	12.361	14.417
3.75	1.45	13.965	16.018
4.00	1.50	15.642	17.674
4.25	1.60	16.503	18.584
4.50	1.65	18.282	20.331
4.75	1.75	19.151	21.250
5.00	1.85	20.019	22.171
5.25	1.95	20.893	23.089
5.50	2.05	21.764	24.000
5.75	2.15	22.641	24.933
6.00	2.20	24.403	23.932
6.25	2.30	25.269	24.891
6.50	2.40	26.137	25.766
6.75	2.50	26.435	26.329
7.00	2.60	27.876	27.605
7.25	2.70	28.740	28.526
7.50	2.80	29.617	29.447
7.75	2.90	30.490	30.369
8.00	2.95	32.248	32.057

2.12.2 Design of Cantilever wall - RCC Retaining Walls

The first step is to evaluate the soil parameters. Next the trial section may be selected using Fig2.7 as a guideline. It is the usual practice to use Rankine's analysis for active earth pressure calculation when the height of the wall is 7m and taking $\delta = 0$. For wall over 7m in height, the Coulomb's equation may prove to be more economical.

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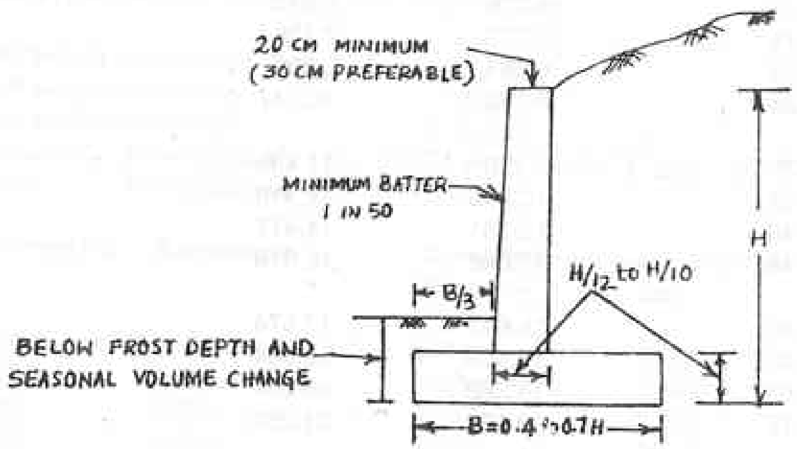

Fig.2.7

The base slab dimension should be such that the resultant of the vertic loads falls within the middle one-third. If it falls outside the middle third, the toe pressur becomes large and only a part of retaining wall will be effective. Figure 2.8 may be use as guide to select the toe dimension *a* and heal dimension *b* so that the resultant fal approximately with in middle one third. The use of Fig2.8 for selecting a and b is indicate therein. The forces acting on the cantilever retaining wall are shown in the Fig. 2.9 an 2.10. Based on trial dimensions, the stability of the wall must be checked for a factor o safety of 1.5. The forces involved in the stability analysis are shown in Fig2.11 and 2.12 If the factor of safety is inadequate for the trial section, the heel part may be increased o a heel key may be used.

Fig. 2.13 enables to estimate the depth of the heel key. The allowable bearing pressure next computed. Since the load is eccentric, the concept of effective width (B' = B-2e should be applied when evaluating the bearing pressure. The toe and heel pressure a then calculated.

Shear check is made for stem and heel moments are calculated using differential equation presented in fig 2.10. Finally knowing moments, the steel requirements may be calculate

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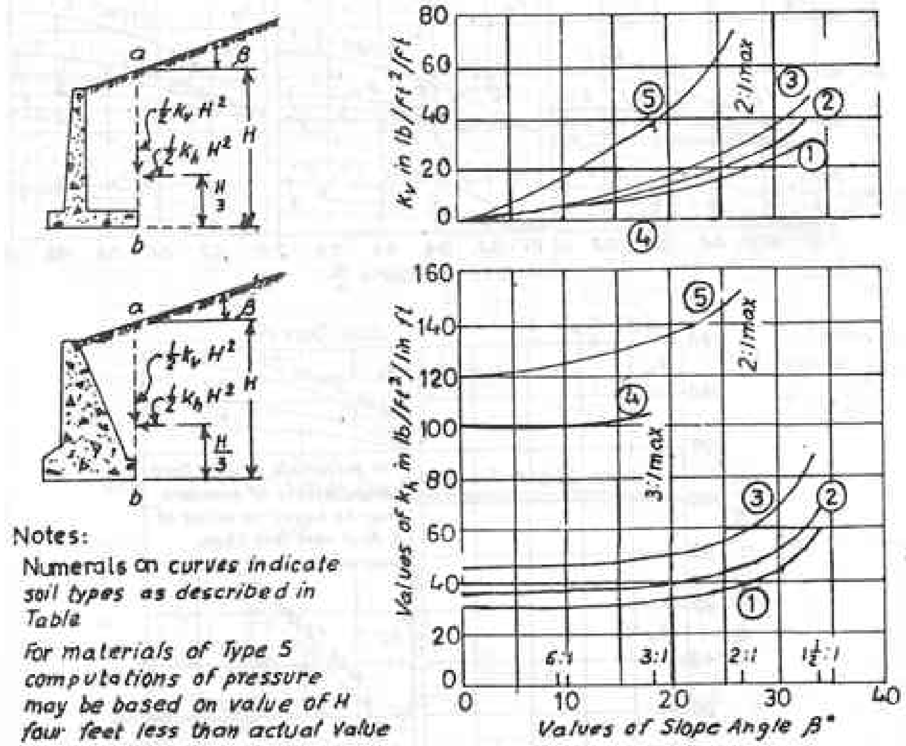

CHART FOR ESTIMATING PRESSURE OF BACKFILL AGAINT RETAINING WALLS SUPORTING BACKFILLS WITH PLANE SURFACE.

Fig.2.5

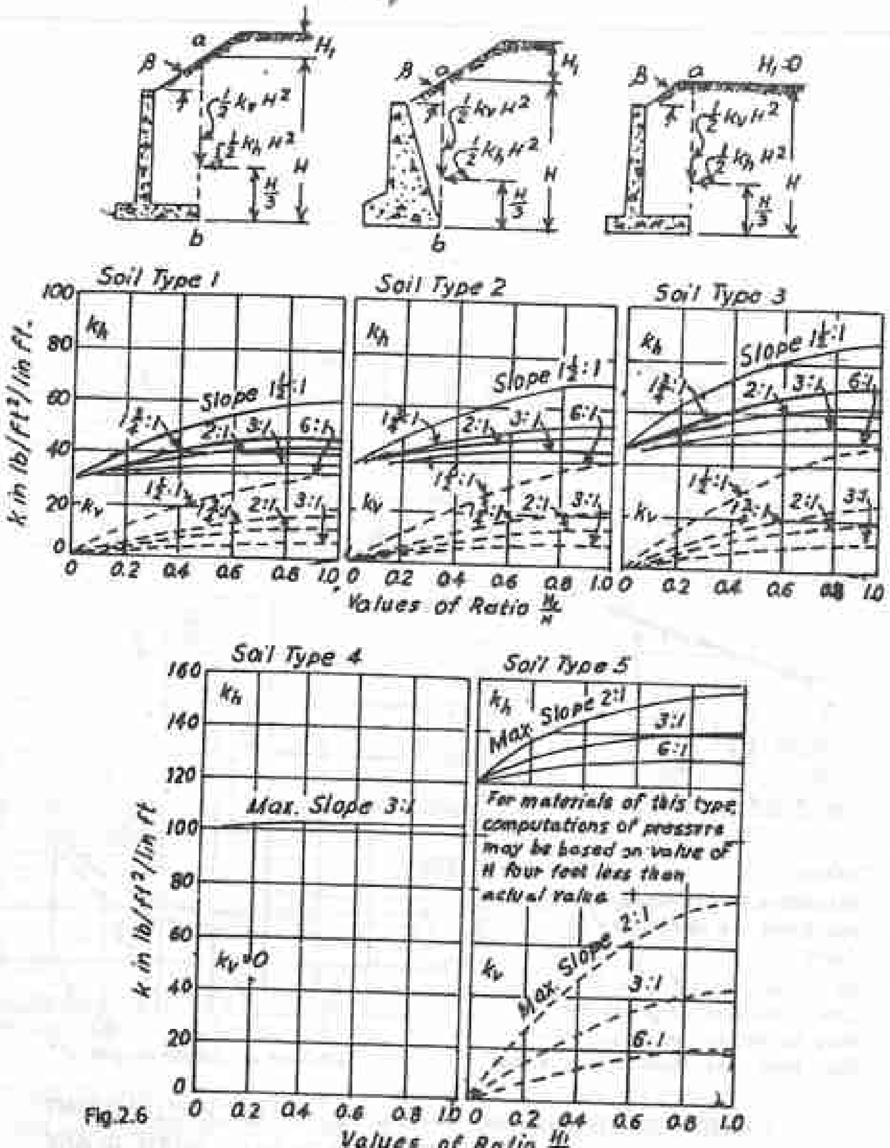

Fig.2.6

CHART FOR ESTIMATING PRESSURE OF BACKFILL AGAINT RETAINING WALLS SUPPORTING BACKFILLS WITH SURFACE WHICH SLOPES UPWARD FROM CREST OF WALL FOR LIMITED DISTANCE AND THEN BECOMES HORIZONTAL

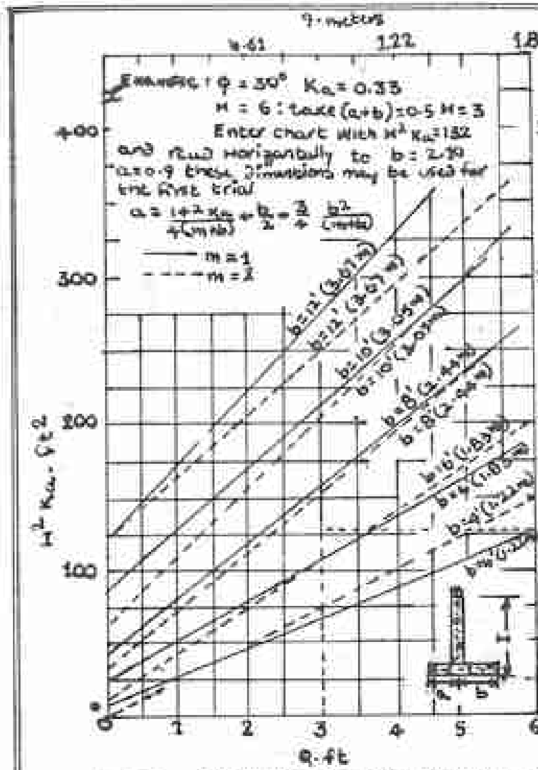

Fig.2.8

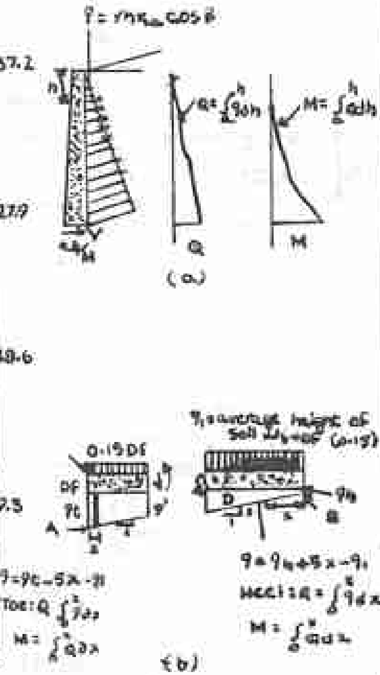

Fig.2.10

EARTH PRESSURE AND RETAINING STRUCTURES

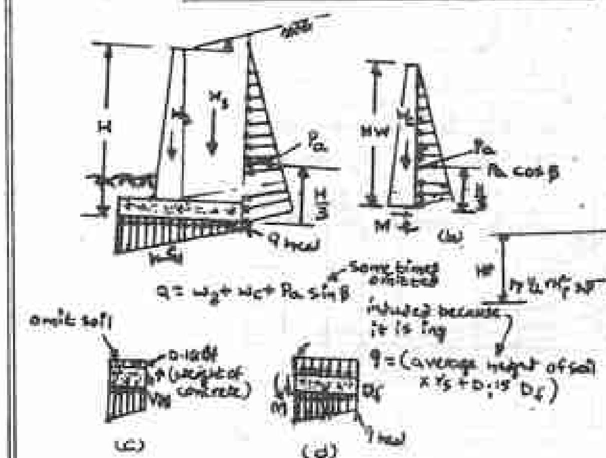

Fig 2.9

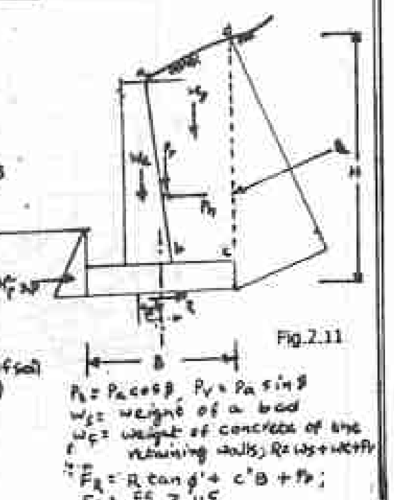

Fig.2.11

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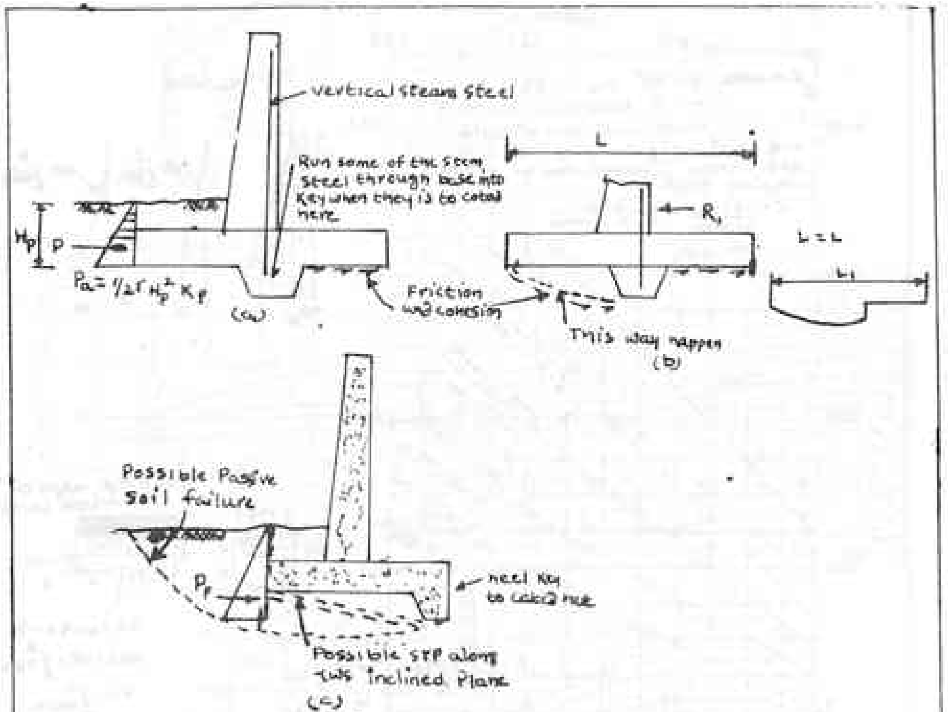

Fig.2.12

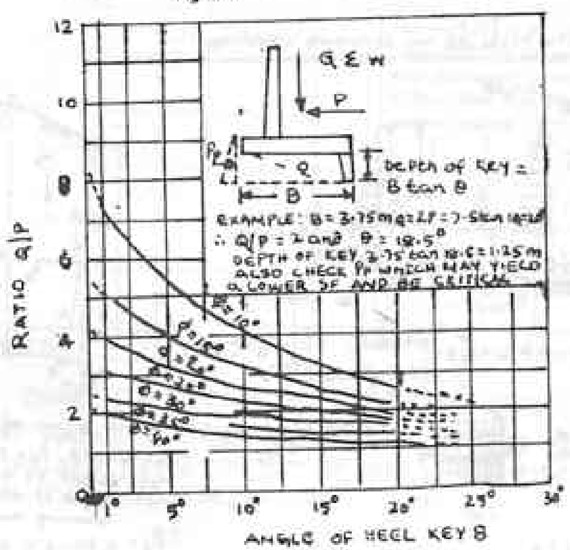

Fig.2.13

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Chapter - 3

IRRIGATION

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3.1 DISCHARGE CALCULATIONS

3.1.1 (1) Weirs : (Clear over fall)

$$Q = C\,L\,H^{3/2}$$

Where Q = Discharge over weir in cumecs.
L = Length of weir in metres.
H = Head over weir in metres.
C = 1.84 for sharp crest (for glacis fall)
1.71 for broad crest (for notch type regulator)
1.40 for broad crest (for flush escape)

3.1.1 (2) (a) Discharge for a drowned weir

General Formula

$$(2/3).C_1 . L\sqrt{2g}\,[\,(h+ha)^{3/2} - (ha)^{3/2}\,] + L.\,C_2\;2d\sqrt{2g(h+ha)} \quad \text{(FPS)}$$

or

$$3.10\,L.\,[\,(h+ha)^{3/2} - (ha)^{3/2}\,] + 8.\,C_2.d.(h+ha)^{1/2} \quad \text{(FPS)}$$

or

$$1.71\,L.\,[\,(h+ha)^{3/2} - (ha)^{3/2}\,] + 1.34\,Cd.L.(h+ha)^{1/2} \quad \text{(MKS)}$$

Where 'h' represents the known difference of water level between front and rear of th weir, 'ha' = head due to velocity of approach either gauged or otherwise known, d = depth of tail water over crest. Different values of C are given in C.E.I's No.2973/57C., dated 4th October, 1971 for various depths of tail water and a diagram is included therein in Malikpur Graph of I.R.I. (Punjab)

$C1 = 0.577$ FPS in the above formula.

Depths of tail water in mts		Value of C2 FPS	MKS
upto	1.50 m	0.60	0.331
	1.80 m	0.62	0.342
	2.10 m	0.66	0.364
	2.40 m	0.75	0.414
	2.70 m	0.84	0.464
	3.00 m	0.90	0.497
	3.30 m	0.93	0.513
	3.60 m	0.95	0.524

3.1.1 (2) (b)	C (FPS / MKS)	
100	1.90	1.050
95	2.46	1.358
90	2.70	1.490
85	2.86	1.580
80	2.98	1.645
55	3.12	1.723
25	3.14	1.733

Note: C, FPS X $\sqrt{0.3048}$ = C, FPS X 0.5521 = C, MKS

C (MKS) = C (FPS) X 0.5521

Discharge Formula $Q = C \times Bt \times D^{3/2}$

Bt = Clear Throat width ; D = U/S TEL - crest level

$$\text{Drowing Ratio (\%)} = \frac{\text{D/S TEL - crest level}}{\text{U/S TEL - crest level}} \times 100$$

C = Coefficient depending upon drowning ratio.

For intermediate values, read the Malikpur graph and convert into MKS system.

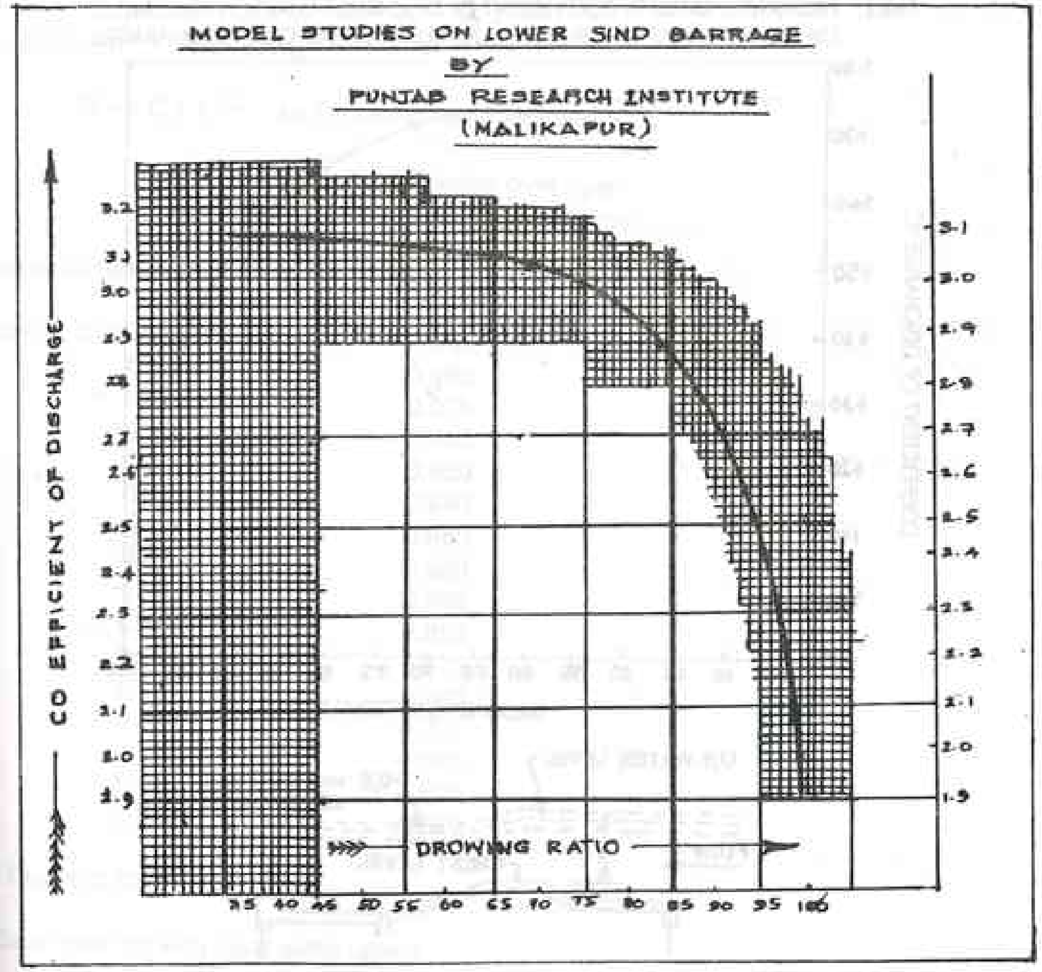
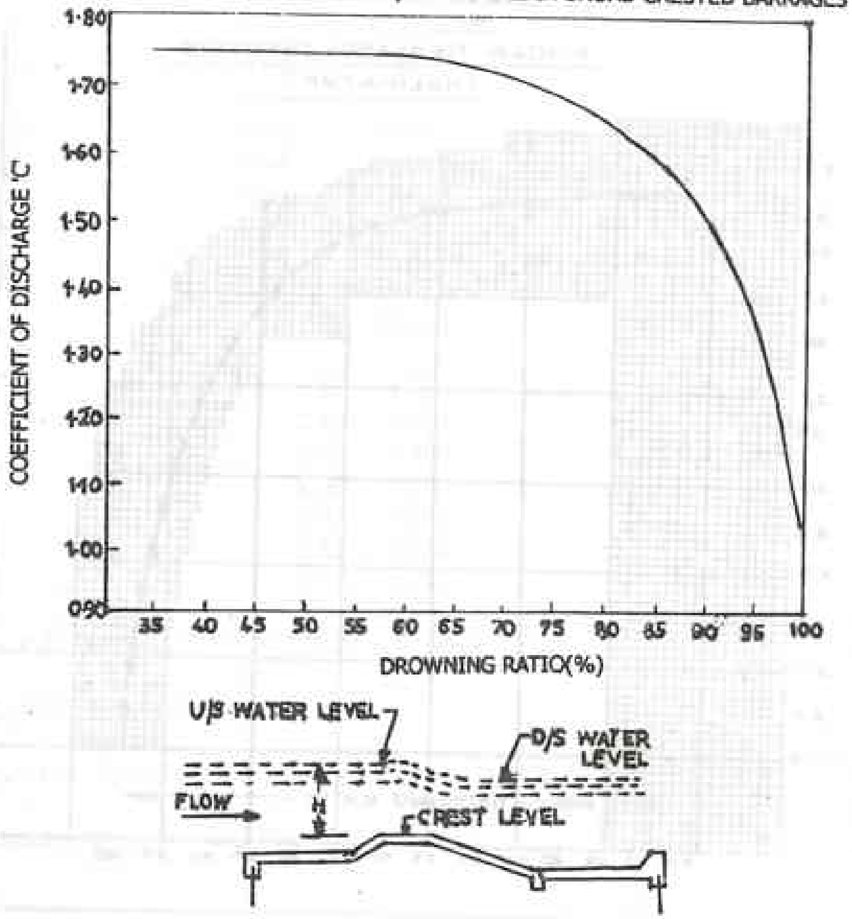


FIG.1 DROWNING RATIO Vs COEFFICIENT OF DISCHARGE CURVE(IN MKS UNITS) (BASED ON MALIKPUR CURVE) APPLICABLE IN BROAD CRESTED BARRAGES



Q=CLH$^{3/2}$

WHERE

Q= DISCHARGE IN CUMECS
C= COEFFICIENT OF DISCHARGE IN FREE CONDITION
L= CLEAR WATER WAY IN METER
H= HEAD OF WATER OVER CREST IN METER

$$\text{DROWNING RATIO} = \frac{\text{D/S WATER LEVEL - CREST LEVEL}}{\text{U/S WATER LEVEL - CREST LEVEL}}$$

3.1.1 (2) (c) Alternately,

$Q = r\,C\,L\,h^{3/2}$ for Drowning Ratio Method

d = depth of (rear) tail water over crest
h = depth of (U/S) front water over crest

Values of Submersion ratio d/h		Value of r
From	0.00 to 0.720	1.000
	0.74	0.985
	0.76	0.975
	0.78	0.965
	0.80	0.950
	0.82	0.940
	0.84	0.960
	0.86	0.980
	0.87	0.880
	0.88	0.850
	0.90	0.815
	0.92	0.780
	0.94	0.720
	0.96	0.640
	0.98	0.520
	0.995	0.340

3.2 Types of Wiers

1. Clear over fall with crest width upto 3'

	Discharge Formulae	C in MKS
General	$Q = C\,L\,H^{3/2}$	
	$Q = 10/3\ LH^{3/2}$ (FPS)	1.840

Where Q = Max flood discharge in cusecs
L = Length of weir in ft.
H = Spillag in ft.

2. Broad crested weir with crest width above 3'	$Q = 9/3\ LH^{3/2}$	1.66
3. Sloping apron	$Q = 8/3\ LH^{3/2}$	1.47 to 1.10
4. Flush escape	$Q = 7/3\ LH^{3/2}$	1.47 to 1.10
5. Standard High Coefficient weir (U/s face 40 ° slope etc.)	$Q=3.75(L-0.1\ nH)\ H^{1/2}$	2.07
6. Ogee-Weir or High Coefficient waeir	$Q=3.98(L-0.1\ nH)\ H^{1/2}$	2.20

Where 'n' is the no. of end contractions which is usually 2

3.3 Canal Regulators

$$Q = Cd \times \sqrt{2gh} \quad \text{(MKS)}$$

Cd = 0.75, h = Difference in Water levels

Linear waterway may be 50 to 60% of canal to prevent excessive afflux.

3.3.1 Pipe Outlets : d.Arch's equation for internal dia (Di) of pipe.

$$Di = [\ 0.00104 + (0.0002586 / Di)\ ]\ D^2$$

D = mean velocity of water through pipe

3.3.2. Regulators and barrages (Ruhlmann's formula) :

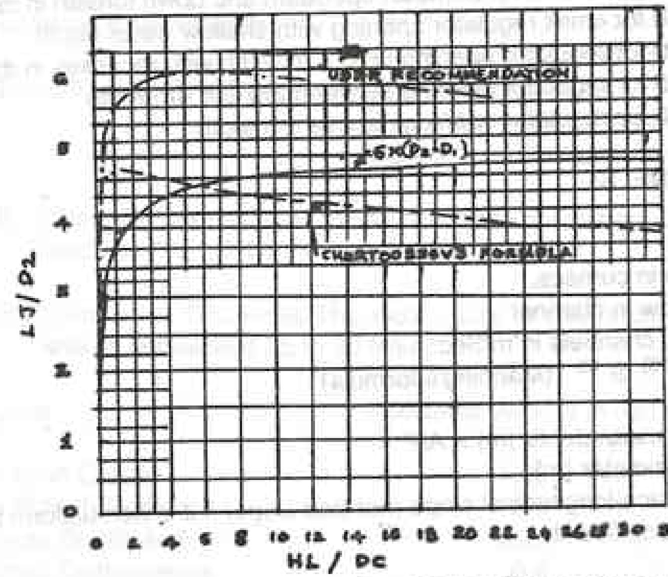
$$Q = 2/3\ CLe\ [2g\ (h+ha)^{3/2} - ha^{3/2}) + CLe\ H\sqrt{2g\ (h+ha)}$$

for submerged weirs.

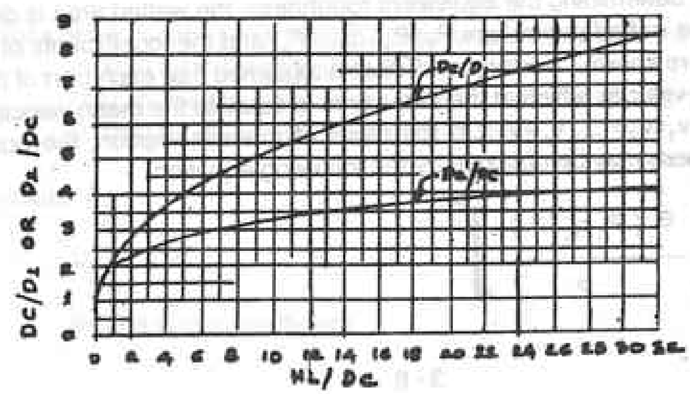
Le = effective width of regulator
= number of spans X span width (N.S)
H = d/s depth of water over crest
C = Coefficient = 0.95 for sharp pointed piers
= 0.90 for piers with square edges

3.4 Design of stilling basins of hydraulic jump type:
C.B.I.P. publication January 1984.

The curves for direct evaluation of H_L / Dc versus Dc/D1 or D2/Dc are furnished below to avoid approximation in evaluating EF2 in horizontal floors. Curve for determining length of stilling basin is also furnished below — (See curves in the figure.)



HEAD LOSS FACTOR VERSUS JUMP LENGTH FACTOR



HEAD LOSS FACTOR VERSUS DEPTH FACTORS

3.5 SLUICES:-

$Q = CAH^{0.5}$

Q = Discharge through sluices in cumecs.
A = Waterway of sluices in Sqm
H = Difference in water level between upstream and down stream in mts
C = 2.50 for small regulator opening with shallow water depth
= 2.80 for regulator opening upto 2.m width with recesses in the piers
= 3.20 for regulator opening between 2m and 4m width
= 3.70 for regulator openings above 4m width

3.6 OPEN CHANNELS:-

Q = A.V.
Where Q= Discharge in cumecs.
A = Area of flow in channel
V = Velocity in channels in m/Sec.
$V = (1/n).R^{(2/3)} S^{1/2}$ (Manning's formula)

Where R = Hydraulic mean depth (m) = A/P
P = Wetted perimeter (m)
S = Water surface-longitudinal slope (not bed slope) if it is non uniform flow)
n = Rugosity Coefficient

3.7 Composite roughness for canal sections (n)

For determining the equivalent roughness, the wetted area is divided in 'N' parts of which the wetted perimeters $P_1, P_2 \ldots\ldots P_n$ and the coefficients of roughness $n_1, n_2 --- n_n$ are known. Horton and Einstein assumed that each part of the area has the same mean velocity, which at the same time is equal to the mean velocity of the whole section; i.e., $v_1=v_2= \ldots. v_n=V$ On the basis of this assumption, the equivalent coefficient of roughness may be obtained by the following equation.

$$n = \left[\frac{\sum_{1}^{N} (P_n n_n^{1.5})}{P} \right]^{2/3}$$

$$= \frac{(P_1 n_1^{1.5} + P_2 n_2^{1.5} + \ldots\ldots P_n n_n^{1.5})^{2/3}}{P^{2/3}}$$

3.8 For World Bank Projects

Q = (0.7 +0.50 P) X CCA

Q = Discharge in lit/sec
P = % Ara of Paddy under irrigation
CCA = Total Cultivable command area

Note: (i) Channels are designed for day irrigation only where CCA is less than 200 hectares (i.e., 12 hours watering per day)

(ii) Distribution Channels: The max - size of field channels under an outlet should not exceed 25 to 30 lit/sec.

Types	Allowable Velocity in mt/Sec
In main Canals	1.05
In Branch Canals	0.9
Large Distributeries	0.75
Small Distributeries	0.6
Minor Distributeries	
Water courses field channels	0.45

Values of n for unlined channels in ordinary soils for various minimum discharges.
0.0250 for discharge upto 3 cusecs
0.0225 for discharge more than 3 cusecs and upto 15C/s
0.0200 for discharge beyond 15 cusecs

Chezy's formula

$$V = C\sqrt{R.S}$$

Where C = a coefficient,

Kutter's $C = \dfrac{23 + (1/n) + (0.00155/s)}{1 + [\,23 + (0.00155/s)\,]\,(n/R)}$ (MKS)

n = Rugosity Coefficient

3.9 Canal Drop

a) Dropwall top width, $a = (0.5d + 0.1524)$ to $(0.5\,d + 0.3048)$

d = depth of water over sill of notch in metres, a = top width in metres

b) Dropwall bottom width $b = (H + d) / \sqrt{\rho}$

d = depth of water over sill

ρ = Specific gravity of the wall material

H = height of wall in metres

b = bottom width in metres

c) Width of Notch at FSL $W \nless d/2$, W and d are in metres

w = Width

d = depth of water over sill of notch and pier has a top width of 3/4 the normal

d) Length of apron $L = 2d + 2\sqrt{dh}$ or $1.22 + 2\sqrt{dh}$ which ever is greater

d = depth over sill of notch in metres

L = length of apron in metres

h = difference between U/S & D/S water level in metres

e) Down stream side apron width

W = L + 0.5d or bed width of canal D/S water level in metres W,L & d are in metres

f) Thickness of apron

Minimum $t = \sqrt{(d+h)/1.811}$ t,d & h are in metres

g) Bed pitching U/S $L1 = 1.5\,d$,

D/S $L2 = 2d$

d = depth of water cushion in metres

h) Revetment U/S $L3 = 3d$ or 3.13 m whichever is greater

D/S $L4 = 4(d+h)$ or 6.1 m whichever is greater

d = depth of water over sill in metres.

i) Depth of water cushion $(dw + d1) = 0.905\, dc \sqrt{h}$

dw = depth of water cushion in metres

d1 = depth of D/S apron in metres

h = difference between U/S and D/S levels of water in metres

dc = depth of water over sill of notch in metres.

3.10 (a) AFFLUX "MOLE'S WORTH" FORMULA

$$h = \left[\frac{v2}{17.86} + 0.01525\right] \left[\left(\frac{A}{a}\right)^2 - 1\right]$$

h = Afflux in meters.

v = Velocity in meters per Sec. (in the rescricted water way).

A = Unrestricted waterway in sq. meters.

a = Restricted waterway in sq. meters.

3.10 (b) Afflux Calculations (Gauthay and Du Buat)

$$h = \frac{v^2}{2g} \left[\frac{a}{\mu\,A}\right]^2 - 1 \quad (\text{MKS})$$

$$Q = \mu\, H\, Le \sqrt{2g\,(h + ha)}$$

A = H.Le, but effectively μ A

Where

A = area below regulator (restricted water way)

a = area u/s of regulator

μ = $0.85 + 0.014\sqrt{s}$, for sharp edged piers

s = Clear span

= $0.76 + 0.029\sqrt{s}$, for semi circular cut waters

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$= 0.70 + 0.029\sqrt{s}$, for square edged piers

or for a/A = 2.40, $\mu = 0.696$ (Fourrey)
1.70, $\mu = 0.771$
1.40, $\mu = 0.800$
< 1.4 or 1.00, $\mu = 1.00$

3.11 SCOUR DEPTH

(a) Lacey's formula

$R = 1.346\,(q^2 / f)^{1/3}$ (MKS)

R = Probable normal scour depth in meters
q = intensify of discharge per meter run
f = Lacey's silt factor = $1.76\sqrt{md}$
md = mean diameter of spheres = equivalent to oblate spheroidal Particles of bed materials in m.m.

for Values of 'f', See 3.1.4 (d)

Scour factor generally adopted for design of aprons

		Range	Mean
1.	Upstream pucca floor	1.25 R to 1.75 R	1.5 R
2.	Down stream of pucca floor	1.75 R to 2.25 R	2.0 R
3.	Nose of guide banks	2.00 R to 2.50 R	2.25 R
4.	Transition from nose to straight reach	1.25 R to 1.75 R	1.50 R
5.	Straight reaches of guide banks	1.00 R to 1.50 R	1.25 R

(b) Lacey's formulae (MKS)

$R = 0.473\,(Q/f)^{0.33}$
$R = 2.50\,V^2 / f$, for channels.

Where R = Hydraulic mean depth (m)
Q = Total discharge in cumecs f = Lacey's silt factor as in previous case

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3.12. Channels in Earth - DESIGN ASPECTS

3.12. (a) Approximate hydraulic mean depth (S.A.Girshan) F.P.S.

$d = AQ^{0.25}$ to $AQ^{0.33}$, A varying between 0.68 to 1.02, generally $d = 0.70\,Q^{0.33}$

Q = Total discharge in c/s

3.12. (b) Lacey's regime formulae:- (MKS)

$V = 10.864\,R^{2/3}\,S^{1/3}$ for river discharges

$P = 4.75\,Q^{1/2}$, $V = (Qf / 140)^{1/6}$ for channels, or $Q^2 f = 140V^6$.

$A = 2.28 f^{-1/3}\,Q^{5/6}$

$V = 0.44\,f^{1/3}\,Q^{1/6}$

$S = 0.00030\,f^{5/3} / Q^{1/6}$

V = Velocity of flow (m/s)
A = Area of flow (sqm)
F = Lacey's Silt factor. See table given 3.1.4(d)
P = Wetted perimeter (m)
S = Water surface slope, (not bed slope)
R = Hydraulic mean depth (m)

3.12 (c) Modified equation on both Lecey and Kennedy theories:-

Wetted perimeter formula	$P = 2.67\,Q^{1/2}$	...1
Regime test formula	$V = 16\sqrt{R^2 S}$	...2
Slope formula	$S = 0.000542\,f^{5/2} / Q^{1/6}$	...3
Fundamental formula	$V = 1.151\,Q\sqrt{fR}$	...4

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3.12. (d) (i) VALUES OF LACEY'S 'f' SUITABLE FOR DIFFERENT SOILS (FROM CBI PUBLICATION NO.20)

Type of silt	
Very fine silt	0.40
Fine silt, Godavari western delta type	0.50
Fine silt, Jamuna canals	0.60
Fine silt, Krishna western delta type	0.70
Medium silt	0.85
Standard silt	1.00
Medium sand	1.25
Coarse sand	1.50
Fine bajri and sand	1.75
Heavy sand	2.00
Coarse bajri and sand	2.75
Coarse gravel	4.75
Gravel and bajri	9.00
Boulder and gravel	12.50
Boulder and shingle	15.00
Large boulders and shingle	24.15

(ii) VALUES OF COEFFICIENT OF RUGOSITY n IN KUTTER'S FORMULA

$V_o = m. D^n$

	n
Badly maintained field channels	0.030
Well maintained field channels	0.025
Earthen channel upto 500 cusecs of Q	0.0225
Earthen channels above 500 cusecs discharge	0.020

(iii) VALUES OF KENNEDY'S CONSTANTS FOR DIFFERENT REGIONS OF INDIA

($V_o = 0.84\ D^{0.64}$ F.P.S)

Region and type of silt	Value of m FPS	Value of m MKS	Value of n FPS	Value of n MKS
Punjab: sandy silt (silt on which kennedy made experiments)	0.84	-	0.64	-
Sind: loamy silt	0.63	-	0.64	-
Godavari: western delta fine silt	0.67	0.49	0.55	0.55
Krishna: western delta fine silt	0.93	0.55	0.52	0.52

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Cvr = 1.10 V_o to 0.90 V_o, 1.10V_o towards head reach and 0.90V_o towards tail reach, y f $\geqslant$ 1.00 and above and 0.9 V_o to 0.80 V_o if f < 1.00 and below.

(iv)The Critical velocity (Vo) for Godavari and Krishna Delta system (C.O.E.Manual Irrigation by Col.W.H.Ellis) M.K.S.

S.No.	Description	Critical Velocity (Vo) FPS	Critical Velocity (Vo) MKS
	$V_o = CD^m$		
1.	Godavari Delta	$0.67\ d^{0.55}$	$0.489\ d^{0.55}$
2.	Krishna Delta	$0.93\ d^{0.52}$	$0.546\ d^{0.52}$

V, mean = V, allowable = 1.1 to 0.9 Vo, keeping higher values towards head reaches and lower ones towards D/S.

Values of C (FPS)	depending on bed grade
For Fine Silt	0.63
Light Sandy Silt	0.82
Fine Sandy Silt, Standar Silt	0.84
Light Coarse Sandy Silt	0.90
Sandy Loam, Standard Silt	1.00
Coarse Silt, Coarse Sand	1.07
Sand and Bajri Small	1.2 - 1.5
Bajri and gravel	1.3 - 3.0
Gravel & Boulders	3.0 - 3.50

3.13 LOSS OF HEAD.

SYPHONS unwin's formula (MKS)

$h = (1 + f_1 + f_2 + L/R)\ V^2/2g$

L = Length of barrel ; R = Hydraulic mean depth of barrel

V = Maximum velocity through barrel

f_1 = 0.505 for unshaped mouth, (CS of barrel and C.S. of mouth are same), for bell mouth, f_1 = 0.08

$f_2 = a\ (1+b/R)$ = Coefficient for head loss through barrel, while f_1 is for entry loss.

	a	b
For smooth cement plaster or planed wood	0.00316	0.0305
Ashlar or brick work	0.00401	0.070
Smooth iron pipe	0.00497	0.025
for encrusted iron pipe	0.00906	0.02506
Rubble masory or stone pitching	0.00507	0.250
RCC Hume pipes	0.003	0.010

3.14 SPILLWAYS :

(a) Ogee weir or High co-efficient weir :

$Q = 3.98\ (L - 0.1\ n\ H)$ n = No.of end contractions (generally 2)

Coordinates for Unit head on crest - (Standard Creager Profile)

	U/s face vertical		U/s face inclned at 45 °	
	Lower Nappe	Upper Nappe	Lower Nappe	Upper Nappe
0.00	0.126	-0.831	0.043	-0.781
0.10	0.036	-0.803	0.010	-0.756
0.20	0.007	-0.772	0.000	-0.724
0.30	0.000	-0.740	0.005	-0.689
0.40	0.007	-0.702	0.023	-0.648
0.60	0.063	-0.620	0.090	-0.552
0.80	0.153	-0.511	0.193	-0.435
1.00	0.267	-0.380	0.333	-0.293
1.20	0.410	-0.219	0.500	-0.120
1.40	0.590	-0.030	0.700	+0.075
1.70	0.920	+0.305	1.050	+0.438
2.00	1.310	+0.693	1.470	+0.860
2.50	2.100	+1.500	2.340	+1.710
3.00	3.110	+2.500	3.390	+2.760
3.50	4.260	+3.660	4.610	+4.000
4.00	5.610	+5.000	6.040	+5.420
4.50	7.150	+6.540	7.610	+7.070

Note: 1. Multiply the quantity in the table by the head on the crest.

2. Origin of coordinates is intersection of u/s face and horizontal through top of crest.

(b) Elementary triangular profile of a Dam/weir

a = Top width of weir = $3d / 2\rho$ min;

or a = $(H^{0.5} + d^{0.5}) / 1.811$

Where d is spillage in ft./m

ρ = Specific gravity of masonry

H = Height of the body wall in ft/m

b = bottom width = $(H + d) / \rho^{0.5}$

(c) STANDARDS REGARDING THE DEPTH OF FLOOD (FREE BOARD IN TANKS)

Spillage and height of free board are as follows: (F.P.S/M.K.S)

Catchments	Depth of flow over weir (spillage)		free board	
	FPS	MKS	FPS	MKS
1. 0 to 25 Sq.Miles ...	3'	0.90	4'	1.20
2. 25 to 75 Sq.Miles...	4'	1.20	5'	1.50
3. above 75 Sq.Miles..	5'	1.50	6'	1.80

(d) Sub-structure - allowable stresses in soils:

B.C. Soils	0.5 to 1.0 ton/sft	5 to 10 t/m^2
HG soils	0.5 to 3.0 ton/sft	25 to 30 t/m^2
SDR	3.0 to 3.5 ton/sft	30 to 35 t/m^2
HDR	3.5 to 4.0 ton/sft	35 to 40 t/m^2
FR	4 to 6 ton/sft	40 to 60 t/m^2
HR	6 to 10 ton/sft	60 to 100 t/m^2

(e) Coefficient of discharge 'C' for standard Creager profile - Ogee spillway

$Q = C\,L\,(H)^{3/2}$ H in m, Q = cumecs

H/Hd	C	H/Hd	C	H/Hd	C
0.1	1.67	0.8	2.12	1.5	2.35
0.2	1.77	0.9	2.16	1.6	2.38
0.3	1.85	1.0	2.20	1.7	2.40
0.4	1.94	1.1	2.23	1.8	2.42
0.5	1.98	1.2	2.26	1.9	2.44
0.6	2.03	1.3	2.29	2.0	2.46
0.7	2.08	1.4	2.33		

Note:
1. Velocity of approach, ignored
2. Nappe adheres to d/s face
3. No end contractions
4. H = total head on crest in m, Hd = design head
5. For actual heads considerably higher than the design heads, the nappe adheres to d/s face tas per Model studies.

(f) Reduction in coefficient of discharge of ogee weir due to submergence

$Q = C'\,L\,H^{3/2}$ in cumecs

H/D	C'/C	H/D	C'/C	Remarks
0.1	1.0			H = u/s water depth over crest
0.2	1.0	0.7	0.93	
0.3	0.99	0.8	0.85	D = d/s water depth over crest
0.4	0.98	0.9	0.67	
0.5	0.97	0.95	0.47	
0.6	0.96	1.00	0.00	

Note: H/D = Degree of submergence
C' = submerged weir coefficient of ogee profile
C = free overfall weir coefficient
In case of ogee weir, C will be coefficient of discharge for the designed head.

3.15 HYDRO POWER EVALUATION

Theoretical output - maximum available,

$P_t = P\,QH\ /\ 75$ metric horse power

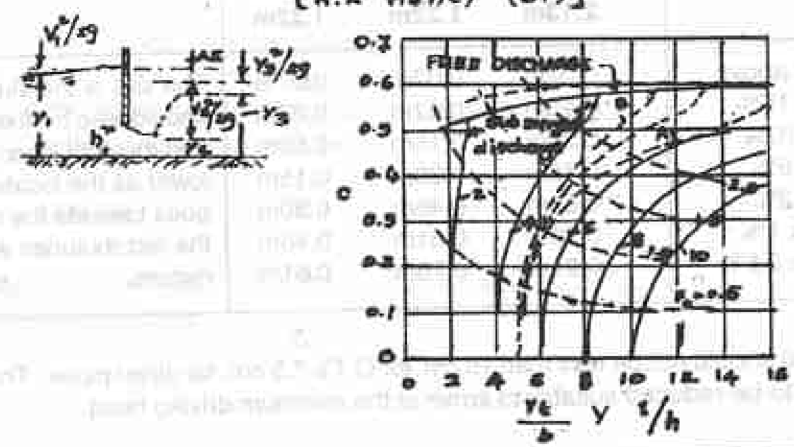
Where P_t = Theoretical output in metric horse power
P = Unit weight of water in Kgs/Cum
Q = flow through turbine/Cum/Sec
H = Head available in metres
Effective output (in metric horse power)

Pt = ePt = (e P / 75) x QH

Where e = efficiency, for estimate purposes, assumed as 75%
Pt = 10 QH = metric horse power (1 metric horse power = 0.7355 kw)
Available Hydro Power = 7,355 QH Kilowatts.

3.16 VERTICAL LIFT GATES - SLUICES :

DISCHARGE COEFFICIENT FOR VERTICAL SLUICE GATE
[H.R Heney (61)]



Discharge Coefficient for Vertical Sluice gate, $Q = C L h\sqrt{2 g y_1}$

Where Q = Discharge
C = Coefficient of Discharge
L = Length of gate
h = Height of gate opening
y_1 = U/s Depth of flow
Fo = Froude number of the flow through the gate opening

Note: The form of this equation is the same for both free and submerged flow. y_1 should be replaced by the effective Head or the difference between U/s and D/s depths.

3.17 LOCATION OF SILLS IN OFFTAKE SLUICES:

The height of the sills of the sluices above the bed level of respective parent Canals are to be fixed from the table given below :

Percentage of off-take discharge to parent canal Discharge	Height of Sills above the bed of parent canal when the FSD in the parent canal is			REMARKS
	above 2.13m	2.13 to 1.22m	below 1.22m	
15% and above	0.07m	0.07m	0.07m	The sills of the sluices should also be fixed such that they get lower and lower as the location goes towards the end of the distributories and minors.
10% to 15%	0.15m	0.07m	0.07m	
5% to 10%	0.30m	0.15m	0.07m	
2% to 5%	0.46m	0.30m	0.15m	
1% to 2%	0.61m	0.46m	0.30m	
0.5% to 1%	0.76m	0.61m	0.46m	
Less than 0.5%	0.91m	0.76m	0.61m	

Driving head should not be less than 15 cm. for O.T's 7.5 cm. for direct pipes. The height of the sill is to be reduced suitably to arrive at the minimum driving head.

3.18 (a) RELATIONS BETWEEN DIFFERENT EXPRESSIONS FOR DUTY OF WATER

One Cusec flowing one day	=	0.0864 Million cubic feet
	=	2 acre feet (1.98 exactly)
	=	24 acre inches (23.79 exactly)
One Million cubic feet	=	11.574 cusecs flowing for one day
	=	23 acre feet (22.96 exactly)
One acre foot or 12 acre inches	=	1/2 cusec flowing for 24 hours nearly

One acre per Million cubic feet is equivalent (approximately) to a duty of 12 acres per cusec for a crop period (base) of 140 days.

One acrefoot (one foot depth of water on one acre area) = 4840 X 9 X 1 = 43,560 cft.
One cusec day = 1 x 24 x 60 x 60 = 86,400 cft. = 1.98 acre ft. Say = 2.00 acre ft.
One hectare-metre (one metre depth on one hectare area) = 10,000 cu.m.

Q_1 cusecs discharge flowing to fill C_1 TMcft capacity reservoir, the period required $R_1 = C_1 \times (11.575 / Q_1)$ days.

<u>(b) Duty of Water:-</u>

Relation between base period, duty and delta is: $\Delta = d = 8.64\, B / D$ (MKS)

Where D = Duty in over hectares / cumec ; B = Base period in days
$\Delta = d$ = Depth over irrigated area (m)

$\Delta = d = 24\, B / D$ (FPS)

Where D = Duty in acres / Cusec ; $\Delta = d$ = Equivalent depth in inches
B = Base period in days

(c) FLOW OF WATER "CONVERSION TABLE"

One Cusec Equivalent to	Cubic Foot	Cubic Yard	Cubic Meters	Gallons
Per Second	1	0.3703	0.2832	6.25
Per Minute	60	2.222	1.6999	3.75
Per Hour	3600	133.33	101.95	22500
Per Day	86400	3200	2446	540000
Per Week	604800	22400	17125	3780000
Per 28 days	2419200	89600	68500	15720000
Per 30 days	2592000	95000	73392	16800000
Per 31 days	2678400	99200	75838	17340000
Per 365 days	31.536 Million	116800	892958	197.1 Million

3.19 Standards for Minor Irrigation Projects

1. YIELD CALCULATION:

Yield at a tank site is calculated using Strange's tables. Based on the topography and slopes, the catchments are classified as good, average or bad. Only 90% of the average rainfal (which is called the Monsoon rainfall) is considered for computation of the yields.

2. STORAGE PLANNING : (No. of fillings of the tank) :

(i) (a) Two filling where the average rainfall is more than 35"
(b) One and half where the average rainfall is between 25" and 35"
(c) One where the average rainfall is less than 25".

(ii) In case where irrigation water is used mainly after the Monsoon for maturing the monsoon crop, the entire requirement of water including losses in the monsoon period has to be stored.

3. DEAD STORAGE:

Usually 10% of the Live Storage is allowed.

4. Allowance for absorption and evaporation losses from the reservoir is 10% of the Live Storage (capacity) irrespective of the rainfall and the number of fills.

5. AREA IRRIGATED PER M.cft. OF WATER:

For the areas where the monsoon rainfall is upto 30" :

(a) Wet crops - 6 acres per Mcft (1 st Crop) and 3 acres per Mcft for 2 nd crop.

(b) Irrigated dry crops - Kharif (Summer Crops) - 20 acres/M. cft. (June to October) Rabi (Winter Crop) - 10 acres/M. cft. (November to March)

For the areas where the monsoon rainfall exceeds 30", irrigation demand will have to be worked out considering the total demand of 66 for the paddy crops inclusive of rainfall, to be distributed equally over each month of the crop period. Additional 6" should be added for 1st month for trans plantation purposes.

Capacity of the tank is determined by using the formula

$$C = (1/6) \sum (A1 + A2 + \sqrt{A1\ A2}) \text{ Cubic Units}$$

Where C - Capacity of the tank in units,
A1 & A2 are the areas of two successive contours in acres and I is the contour interval in feet.

<u>3.19</u> <u>(a) Utilisable yield for Minor Storage works:</u>

Mullin's equation $(1.25 - X)^2 + (0.25 + y)^2 = 1.625$

X = ratio of reservoir capacity to yield ;
Y = ratio of utilisable yield to total yield

Minor storage work is to have an ayacut of 200 ha.

(b) Tank duties for Minor Irrigation works:

Crop	Rain fall in cms.	Duty in M^3/Ha.
a. Rice - (Rabi)	35 cms	17500
(Kharif)	35 - 70 cms	14000
	70 - 100 cms	12000
	> 100 cms	10000
b. Rice Rabi (Tabi)	-	12000
c.Sugar cane - Perennial (Dofasla)	-	30000

(c) The following Duties may be had as general guidance

Crop	Duty in	M.Cft/acre	Mcum/Ha
Kharif Paddy	...	0.20	0.01399
Rabi Paddy	...	0.39	0.027289
Sugarcane	...	0.49	0.034286
Garden Crops	...	0.087	0.006088
Kharif	...	0.03	0.002099
Rabi	...	0.066	0.004618
Cotton	...	0.098	0.006857
Green Manure (Winter)	...	0.025	0.001749
Green Manure (Summer)	...	0.040	0.002799

(d) Utilisable yield for "Diversion works"

Total Yield million cum	Below 1	1 to 5	5 to 15	15 to 50
Probable utilisable Yield	10%	20%	30%	40%

3.20 (a) Crop water requirements in Tungabhadra lowlevelcanal command area (G.O.Ms.No.21/CAD II Dt.16-02-1976) (FPS)

Crop	Month	Duty (Rabi I.D) Acres/Cusec
Ground nut	February	120
Ground nut	March & April	100
Hybrid millets	February	150
Hybrid millets	March & April	120
Cotton	November & February	150

(b)Crop water requirements - WALAMTARI, A.P.

Name of crop	Days for maturity	Water requirement in inch/cm			
		Daily		Total	
		Inches	Cms	Inches	Cms
Sugar cane	365	0.26	0.66	95.00	241.30
Cotton	202	0.21	0.53	42.2	107.19
Rice	98	0.43	1.09	41.7	105.92
Tobacco	132	0.30	0.76	39.2	99.57
Chillies	202	0.19	0.48	38.8	98.55
Ragi	127	0.23	0.58	29.8	75.69
Potato	88	0.30	0.76	26.7	67.82
Gnut	124	0.21	0.53	26.1	66.29
Jower	154	0.23	0.58	25.7	65.28
Maize	100	0.18	0.46	17.8	45.21
Wheat	88	0.17	0.43	14.8	37.59
Mustard	88	0.12	0.30	10.6	26.92

3.21 I.S.7112 - 1973 : Unlined canals in alluvial soils

a) free board 0.5 m for Q less than 10 cumecs
0.75 m for Q greater than 10 cumecs

b) Top width of banks

Discharge in cumecs	Minimum top width of bank in m	
	Inspection bank	Non-Inspection bank
0.15 - 7.5	5	1.5
7.5 - 10.0	5	2.5
10 - 15.0	6	2.5
15 - 30.0	7	3.5
30 and above	8	5.0

c) Hydraulic gradient : minimum 4 H to 1 V with 0.30m cover, minimum

d) Dowel bank height and top width 0.5m, side slopes 1½ : 1

e) Regime type fitted equations for design of unlined canals (All India canals)

S = slope = $0.000315 / Q^{0.1651}$
P = Wetted Perimeter = $4.30\ Q^{0.5231}$
R = HMD = $0.515\ Q^{0.3406}$

3.22 DATA FOR IMPORTANT CROPS IN INDIA

Crop	Sowing Time	Harvesting Time	Seed required per hectare in kg	Average yield per hectare in tonnes	Total water depth in cms (Including rainfall)	Irrigation requirements
KHARIF						
Maize	June	Sept-Oct	13.5	1.1 to 1.7	45	Usually nil except for 'paleo'
Early rice	June	October	90	0.9 to 1.10 of 'dhan (husk about 25%)	'90-145	when rains have a long gap
Transplanted rice	July	November	90	1.35 to 1.60 of dhan	'125-150	when rains have a gap and in October
New varieties of rice	July	End of October	90	3.00 to 4.00	125	Continuous submergence to a depth of about 5 cms., gives best results
Spiked millets (Bajra)	July Aug	End of Oct	1.5 to 2.0	0.55 to 0.70	'25-50	Seldom Irrigated
Great millets (Juar)	July	Usually sown for fodder and cut green more than once	5	0.7 if allowed to ripen	30	After a cutting
Arhar	July August	March-April	15	0.7	30	Not irrigated usually
Urd, Moong	Late Aug	Nov	15	0.55	'25-30	Not irrigated usually for urd there is an early variety also
Groundnut	May - June	Continued from Nov. to Jan.	10 to 15	0.35 to 0.45	'65-80	For sowing, normally not required afterwards
Til	July - Aug.	Nov. to January	10 to 15	0.35		Seldom Irrigated

DATA FOR IMPORTANT CROPS IN INDIA

Crop	Sowing Time	Harvesting Time	Seed required per hectare in kg	Average yield per hectare in tonnes	Total water depth in cms (Including rainfall)	Irrigation requirements
			RABI CROPS			
Wheat	October	Oct - Nov	90 to 130	1.35	37.5	Three waterings or less depending on rainfall.
Wheat (New varieties)	November	March - April	90 to 130	3.5 to 4.00	40	Four waterings, less effective rainfall
Barley	October	March - April	90 to 130	1.35	45	Not usually Irrigated
Gram	September	March	40 to 50	1.1	30	Seldom Irrigated
Peas	Oct - Nov	March	40 to 50	1.2	35	Usually Irrigated
Potatoes	October	Feb	185	15 to 25	60 to 90	to be watered every 7 to 10 days
Mustard	Oct.	Feb. - march	10 to 15	0.7	45	Generally sown mixed with wheat and irrigated with it.
Tobacco	Seedlings planted out in Oct - Feb	Feb or May		1	60 to 70	To be irrigated frequently
			PERENNIAL			
Sugarcane	Feb - March	Dec. - March Next year	2750 kg. of cut pieces	25 to 30	270	Irrigation essential during summer later if rains break.

3.23 STANDARDS OF CANAL CROSS SECTIONS (MKS)

Items in metres	Main and Branch Canals		Major Distributories		Minor Distributories	
	150 to 30 cumecs	30 to 10 cumecs	10 to 5 cumecs	5 to 1 cumecs	1 to 0.3 cumecs	Below 0.3 cumecs
Maximum crest width of bank	3	2.5	2.25	2	1.5	1
Free board	0.9	0.75	0.6	0.5	0.4	0.3
Width of berms Min	6 + 1/4th of the combined side slopes of cutting and embankment					
Max	0.6 + 1/2 the width of combined slopes					
Width of Roadway	6	5	5	3.5	3.5	Nil
Depth of each cover over saturation gradient	1	0.8	0.5	0.5	0.5	0.5
Width of land to be acquired clear of banks when canal is in cutting deeper than balancing depth of cutting	5			Half the height of the bank above ground subject to minimum of 1.5		
Width of land to be acquired clear of bank when canal is in less than balancing depth of cutting	Full height of bank+5			Full height of bank above ground, 1.5 minimum.		

Berm width may also be adopted as under:-

Upto 4.25 cumecs. discharge	=	0.6 + 1/2 F.S.depth.
4.25 to 28 cumecs	=	1.25 + 1/2 F.S.depth
28 cumecs and above	=	1.25 + F.S. discharge/28 + 1/2 F.S.depth

Duala or Dowel in the form of a small earthen ridge is provided to prevent vehicles from falling into the channel. It is provided on the inner edge of the service road. The top width of daula is about 30 to 60 cm and height 40 cm. If the bed of the channel is kept at such a depth below ground level that the earth obtained from excavation is exactly equal to the amount of the earth required for making the banks, such a section is called economical section as it entails minimum earthwork and that depth of canal is known as the balancing depth.

3.23.1 <u>Formula for balancing depth of cutting. (para 427, Ellis manual, F.P.S).</u>

$$D^2 - (3h + B + 2b)\,D = -h\,(B + 3/2\,h).$$

Where,

h = **vertical** height of top of bank above bed of the channel.

b = bed width of channel

B = Top width of bank

D = balancing depth of cutting in feet

Slope in cutting 1:1 and slopes of the two equal banks are 1 ½ : 1.

3.23.2 <u>Values of Rugosity coefficient : (n) for unlined canals (Clause D1.1.2)</u>

S.N.	Type of canal	n Medium	Normal	maximum
i.	*Earth straight and uniform*			
a.	Clean, recently completed	0.016	0.018	0.020
b.	Clean, after weathering	0.018	0.022	0.025
c.	Gravel, uniform section clean	0.022	0.025	0.030
d.	With short grass, few weeds	0.022	0.027	0.033
ii.	*Earth winding and slugish*			
a.	Novegetation	0.023	0.025	0.030
b.	Grass, some weeds	0.025	0.030	0.033
c.	Dense, weeds or aquatic plants in deep channels	0.030	0.035	0.035
d.	Earth bottom and rubble sides	0.030	0.035	0.040
e.	Stony bottom and weedy banks	0.025	0.035	0.040
f.	Cobble bottom and clean sides	0.030	0.040	0.050
iii.	*Drag line excavated or dredged*			
a.	No vegetation	0.025	0.028	0.033
b.	Light brush on banks	0.035	0.050	0.060
iv.	*Channels' not maintained (weeds and brush uncut)*			
a.	Dense weeds, high as flow depth	0.050	0.080	0.120
b.	Clean bottom, bush on sides	0.040	0.050	0.080
c.	Same, highest stage of flow	0.043	0.070	0.110
d.	Dense bush, high stage	0.080	0.100	0.140

Note:

1. For normal alluvial soils, it is usual in India to assume a value of $n = 0.02$ for bigger canals $Q > 15$ cumecs and $n = 0.0225$ for smaller canals $Q < 15$ cumecs.
2. A suitable value of n should be adopted keeping in view the local conditions and the above values as a guide.

3.23.3 I.S. 4745 - 1968 LINED CANALS:

a) Transmission loss may generally be assumed to be 0.60 m^3/s/million m^2 of wetted area for lined canals.

Note: Transmission loss of 0.60 m^3 /s/million m^2 of wetted area of lined canal is an assumed value although the actual value may be much less and different for different lining specifications. For the canals of Bhakra Nangal Project for estimating the seepage loss, the following formula was used.

$K = 1.25\ Q^{0.056}$ (FPS) $K = 0.4654\ Q^{0.056}$ (MKS)

	FPS	MKS
Where	K = the loss in cusecs/million sq.feet of wetted area Q = Discharge in cusecs	loss in cusecs/s/million m^2 of wetted area and Q = Discharge in cumecs.

b) Side slopes: Lining is usually made to rest on slopes corresponding to the angle of repose of the natural soil so that the slopes shall be such that no earth pressure is exerted on the back of the lining.

c) Free board : 0.75m above FSL upto top of lining.

d) Bank width:
a. 8m for cutting and filling reaches of main canals
b. 6.5m for cutting reaches for branch canals
c. 6.5m on left side and 5m on right side for filling reaches of branch canals.

e) Velocity : About 2m/s. for lined canals is recommended.

CV Ratios are not applicable to lined canals but the possibility of silting cannot be neglected and hence the CVR should be aimed at higher than unity.

f) Velocity (mean) = $R^{2/3}\ S^{1/2}\ /\ n$
R = Hydraulic mean depth n = rugosity coefficient as in 6.2.18 (3) (b)

Amendment I September, 1970: The figures specified in this standard will require to be revised after the value of rugosity coefficients of various types of lining are investigated

by the CBIP. See Technical Report 12 of CBIP on Determination of Rugosity coefficient for lined and unlined channels - September, 1974.

g) Catchwater drains : Effective system of catch water drains shall be provided to protect damage due to rain.

h) Under drainage: For a lined canal where the ground water level is higher or likely to be higher than water level in side the canal so as to cause damaging differential pressures on the lining, or where the sub grade is sufficiently impermeable to prevent free drainage of the under side of lining in case of rapid draw down, pressure relief arrangements for under drainage shall be provided in accordance with IS 4558 - 1968.

3.23.4 VALUES OF RUGOSITY COEFFICIENT (n) FOR LINED CHANNEL WITH STRAIGHT ALIGNMENT : (5.2)

Surface characteristics	Value of n
1. Concrete with surface as indicated below:	
a) Formed, no finish	0.013 - 0.017
b) Trovel finish	0.012 - 0.014
c) Float finish	0.013 - 0.015
d) Float finish some gravel on bottom	0.015 - 0.017
e) Gunite, good section	0.016 - 0.019
f) Gunite, wavy section	0.018 - 0.022
2. Concrete bottom float finished sides as indivated below:	
a) Dressed stone in mortar	0.015 - 0.017
b) Random stone in mortar	0.017 - 0.020
c) Cement rubble masonry	0.020 - 0.025
d) Cement rubble masonry plastered	0.016 - 0.020
3. Gravel bottom sides as indicated below:	
a) Formed concrete	0.017 - 0.020
b) Fandom stone in mortar	0.020 - 0.023
c) Dry rubble (rip rap)	0.023 - 0.033

4. Brick .... 0.014 - 0.017

5. Asphalt
   a) Smooth 0.013
   b) Rough 0.016

6. Wood planed clean .... 0.011 - 0.013

7. Concrete lined excavated rock with
   a) Good section .... 0.017 - 0.020
   b) Irregular section .... 0.022 - 0.027

Note: With channels of alignment other than straight, loss of head by resistance forces will be increased. A small increase to the value of n may be made to allow for additional loss of energy.

3.24 CBIP Technical Report No.12 september 1974

(a) *Unlined Canals : Factors affecting 'n'*

(i) Surface roughness (ii) Channel irregularity (iii) Vegetation (iv) Channel alignment (v)Silting & Scouring (vi) Stage and discharge (vii) Suspended material and bed load

(b) Recommendations values of 'n'

Lined canals

Boulder lined canals	=	0.025
Tile lined canals	=	0.019
Concrete line canals	=	0.015 - 0.017
Masonry	=	0.018 - 0.020
Dry rough stone	=	0.023 - 0.030

(c) Unlined canals

Unlined earthern canals not in regime and showing silting	=	0.025
Unlined Canals	=	0.025 - 0.030
Unlined surface with projections	=	0.030 - 0.050

3.25 Canal Lining:

(a) Insitu concrete lining:

The minimum thickness of concrete lining based on canal capacity shall be as follows:

Canal Capacity m^3 / sec	Thickness of M15 concrete		Thickness of M10 concrete	
	Controlled cm	Ordinary cm	Controlled cm	Ordinary cm
0 to 5	5.0	6.5	7.5	7.5
5 to 15	6.5	6.5	7.5	7.5
15 to 50	8.0	1.0	10.0	10.0
50 to 100	9	10	12.5	12.5
1000 & above	10	10	12.5	15.0

Note: The above thicknesses do not take care of uplift of floor and overturning of sides.

(b) Precast Concrete Slab lining :

The face of the slab shall be either square or rectangular. Nominal dimensions of the slab shall be as specified below.

Length mm	Breadth mm	Thickness mm
500	250	60
500	250	50
250	250	50

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3.26 International Commission on Irrigation and Drainage

II Congress - C.7 India - Design of channels:

Critical Velocity:-

a) Non silting and non scouring velocity

b) i) Lindley's formulae

$V_o = 0.95\ D^{0.51}$

$V_o = 0.51\ B^{0.355}$

$B = 3.80\ D^{1.61}$

ii) Gerald Lacey:- Regime channels -

$S = 0.0005469\ .f^{5/3} / Q^{1/6}$ (FPS)

$S = 0.0003759\ .\ f^{3/2} / R^{1/2}$ (FPS)

c) Limitations of Lacey's formula:-

Minimum f (silt factor)	Minimum Value of discharge in cusecs (FPS) in Trapezoidal sections with side slopes 1½ : 1	1:1
0.50	13.80	18.80
0.75	6.14	8.37
1.00	3.45	4.70
2.00	0.86	1.18

No attempt need be made to design channels of smaller discharge than the above with Lacey's formulae.

d) Berms top widths and free boards - (1 ½ : 1 to 3 : 1 slopes) (FPS):-

Main canal - Branch Canal Distributories and Minors

	5000 to 1000 c/s	1000 to 300c/s	300 150c/s	150 10c/s	less than 10c/s
i) Berm width in feet	4 + Q/1000+D/2	4+D/2	4+D/2	2+D/2	2+D/2 occasionally no berm
ii) Top width in feet	15	6	6	4	3
iii) Free board in feet	3	3	3	2 to 1½	1 to ½

Note: More the top width, less the free board.

e) Dowel banks : 1' (0.3m) height, 1½ : 1 side slopes.

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3.27 Design practices of irrigation canals (I.C.I.D. 1972) :-

A.P. Practice:

1. Design procedure : Trial design (M.K.S)

$Y = C\ Q^{1/3}$ Y = 'epth of water in canal

Q. Cum/sec	C	n value	Remarks
upto 14.16 (500 c/s)	0.70	0.025	In rock cuts
14.66 - 28.32 (500 - 1000 c/s)	0.69	0.0225	n = 0.035
28.32 - 56.64 (1000 - 2000 c/s)	0.66		
56.64 - 84.96 (2000 - 3000 c/s)	0.64		

2. CC (1:4:8) of 45 to 60 cm. thickness for bed protection of drainage way with C.C. cut offs at either end.

3. Flares provided in transitions of vent ways : (minimum)
   u/s 1.5 : 1 and above d/s 2 : 1 and above

4. Batters provided in abutments, wings, piers & returns: (Masonry)
   Water faces Vertical :
   1/6 or 1/5 to 1/3 depending on moments due to earth pressure and height.

   Earthen faces :
   1/3 to 1/2 depending on earth pressure, moments top widths 45 cm to 1.20 m.

   No tension is allowed but IRC section II suggests 1 Kg/cm² for CR stone masonry in CM (1:3) and 0.7 kg/cm for CRS in CM (1:4)

5. Hydrostatic pressures : 50% in Rocks. Pressure relief valves in regulator floors, energy dissipation buckets are provided. Weep holes in retaining walls are provided.

6. Masonry walls : RR masonry in CM (1:6) or (1:5) and pointed in CM (1:4) and with water face of front 45 cm built in CRS in CM (1:5).

7. Fluming ratio in canal regulators/drops etc., : Limited to 0.60 (60%) in order to facilitate smooth transistions and reduce afflux, u/s of the structure.

8. a. Thickness of regulator piers: 0.21 (2H-S) X S (FPS) (Bligh)
S = clear span, H = depth of water in vents.

b. Thickness of piers as per Leliavisky : (FPS / MKS)

Span of opening in ft.	Depth of water in (ft)/ m	15' / 4.5m	20'/6m	25'/7.5m	30'/9m
10' / 3m	pier thickness ft / m	2.5/0.76	2.7/0.82	2.9/0.88	3.1/0.94
15'/4.5m		3.6/1.1	3.9/1.19	4.2/1.28	4.5/1.37
20'/6m		4.6/1.40	5.0/1.52	5.4/1.65	5.8/1.77
25'/7.5m		5.5/1.68	6/1.83	6.5/1.78	6.7/2.04

9. Fill materials behind abutments:

$\phi = 28^{o}$; $\delta = 16^{o}$ (wall frictions generally not considered)

$Y = 2100\ kg/m^3$, K = permeability not more than 3m / year.

The earthen faces of walls are given stepped offsets instead of clean sloping surface to provide grip in the earthen banks.

3.28 (a) Suitable side slopes for channels built in various kinds of material (VEN TE CHOW)

S.No.	Material	Side slope
1	Rock	nearly vertical
2	Muck and peat soils	1/4 : 1
3	Stiff clay or earth with concrete lining	½: 1 to 1 : 1
4	Earth with stone lining or earth for large channels	1 : 1
5	Foam clay or earth for small ditches	1 ½ : 1
6	Loose sandy earth	2 : 1
7	Sandy loam or porous clay	3 : 1

(b) Recommended side slopes in Canals - C.W.P.R.S. Pune

Type of soil	Embankment	Cutting
a) Ordinary	2 : 1 to 3 : 1	1.5 : 1 to 2 : 1
b) Light loose sand, average sandy soil	2 : 1 to 3 : 1	1.5 to 1
c) Sandy loams, B.C.	2 : 1	1.5 : 1 to 1 : 1
d) Sandy soil - Gravel	2 : 1	1 : 1
e) Muram - hard soil	1.5 : 1	0.75 : 1
f) Rock	-	0.25 : 1 to 0.75 : 1

(c) Top width of Banks (CE/Irrigation, Circular, dated 09-02-82)

S.no	Discharge m3/sec	Free board m	Minimum Top width of bank		
			Inspection bank in m	Non-inspection bank in m	Disty System
1.	0.30 and below	0.30	1.00	1.00	0.9/0.9
2.	0.30 to 1.50	0.35	2.00	1.25	1.8/1.2
3.	1.50 to 7.50	0.45	5.00	1.50	3.6/1.8
4.	7.50 to 10.00	0.60	5.00	2.50	4.5/1.8
5.	10.00 to 15.00	0.60	6.00	2.50	4.5/1.8
6.	15.00 to 30.00	0.75	7.00	3.50	4.5/1.8
7.	30.00 and above	0.75-0.90	8.00	5.00	4.5/5.6

For ridge canals, small distributories and minors carrying less than 1.5 cumecs, it is generally not economical to construct a service road on top of banks as this usually requires more material than excavation provides. In such cases service road may be provvided on natural ground surface adjacent to the bank towards ground surface, or adjacent to the bank towards the command area side.

3.29 C.W.C. Bed width to depth ratios as per draft manual on Irrigation channels, C.W.C. Water-wing, New Delhi, 1960.

S.No.	Discharge in Cum/second	B/D	S.No.	Discharge in Cum/second	B/D
1	0.283	2.9	13	11.327	0.57
2	0.425	3.0	14	14.158	6.0
3	0.566	3.2	15	28.317	7.4
4	0.850	3.4	16	33.980	7.9
5	1.133	3.6	17	39.644	8.2
6	1.416	3.7	18	45.307	8.6
7	2.124	4.0	19	50.634	9.0
8	2.832	4.2	20	56.634	9.5
9	3.540	4.4	21	70.792	10.1
10	4.247	4.6	22	84.950	11.0
11	5.663	4.8	23	283.170	19.2
12	8.495	5.1			

3.30 (a) (i) Maximum water velocities (m/sec) in earth canals

Nature of Canal bed	With a discharge in m^3/s 0.5	1.0	2.0	3.0	4.0	10	15	20	30
1. Silt, fine sand, light sandy loam	0.37	0.39	0.41	0.43	0.45	0.47	0.49	0.50	0.52
2. Medium Sandy ground	0.46	0.49	0.52	0.54	0.56	0.59	0.61	0.63	0.65
3. Light loam	0.53	0.56	0.59	0.61	0.64	0.68	0.70	0.72	0.74
4. Medium loam Medium loess, Coarse Sand	0.59	0.63	0.67	0.69	0.72	0.75	0.79	0.81	0.84

5. Heavy Loam, Light Clay, Close grain Loess, Very Coarse sand	0.67	0.71	0.75	0.78	0.81	0.86	0.89	0.90	0.94
6. Fine Shingle or gravel	0.73	0.77	0.82	0.84	0.88	0.93	0.96	0.98	1.02
7. Thick Medium Clay medium gravel	0.82	0.87	0.92	0.95	0.99	1.05	1.09	1.11	1.16
8. Heavy Clay (tertiary) coarse shingle or gravel	1.26	1.34	1.42	1.47	1.53	1.62	1.68	1.72	1.79

Source : FOOD & AGRICULTURAL ORGANIZATION OF UNITED NATIONS, ROME, 1971 MANUAL

(b) Maximum non-erosive water velocities in earthen Canals:

1. Fine sand under quick sand condition .... 0.2 to 0.3 mps
2. Sandy soil.. 0.3 to 0.75
3. Sandy loam .... 0.75 - 0.90
4. Loam to clay loam .... 0.85 - 1.10
5. Stiff clay .... 1.1 - 1.5 mps

Source : Same as above.

3.31 C.B.I.P.: Design of unlined incised canals: Technical Report No.7 June, 1976.

Maximum and Minimum permissible velocities
CWC & CWPRS Recommendations : all over India :

Type of Soils .... **Maximum Values m/sec**
Ordinary soils .... 0.60 to 0.91
Sandy soils .... 0.30 to 0.60
B.C.sandy loam, Kankar .... 0.60 to 0.91
Muram and hard soils .... 0.91 to 1.06
Gravel 1.21
Disintegrated rock 1.52
Disintegrated rock .... 1.52 (depending on size of rock)

Minimum value of $V = 0.8\ V_{max.}$

Width to depth ratios : to minimise adsorption and evaporation and for more economical design.

b/d = 1.25 to 5.0 are beneficial

b/d ratio curve or Table for 0.28 to 283 cumecs - See 6.2.18. (b).

3.32 Radii of Curves in Unlined Channels

Curves in unlined channels should be as gentle as polssible, as there is a tendency to silt on the inside and to scour on the outside of the Curve.

Minimum radii of Curves in unlined channels (I.S.I.)

Capacity of Channel Cusecs	Cumecs	Minimum radius of the Curve Feet	Metres
Over 3000	Over 85	5000	1500
3000 to 1000	85 to 30	3000	900
1000 to 500	30 to 15	2000	600
500 to 100	15 to 3	1000	300
100 to 10	3 to 0.3	500	150
Less than 10	Less than 0.3	300	100

Note: Generally R = 20 B where B is water surface width may be the minimum allowable.

3.33 Permissible Canal Velocities (Etchevery) FPS

Original material excavated for canal	Clear water no delitus	Water transporting colloidal silts	Water transporting non-colloidal silts, sand gravel etc.	Non-Erodible velocities
Very light loose sand	-	-	-	0.75 - 1.50
Fine sand (non colloidal)	1.50	2.50	1.50	2.00 - 2.50
Sandy Loam	1.75	2.50	2.00	2.50 - 2,75
Silt Loam	2.00	3.00	2.00	2.50 - 2.75
Alluvial silts when non colloidal	2.00	3.50	2.00	2.75 - 3.00
Ordinary firm loam	2.50	3.50	2.25	3.00 - 3.75
Fine gravel	2.50	5.00	3.75	4.00 - 5.00
Stiff clay (very colloidal)	3.75	5.00	3.00	4.0 - 5.0
Alluvial Soils when colloidal	3.75	5.00	3.00	4.0 - 5.0
Graded silt	4.00	5.50	5.00	4.0 - 5.0
Coarse gravel (non colloidal)	4.00	6.00	6.50	5.00 - 6.00
Shingle (non colloidal	5.00	6.50	6.50	5.0 - 6.0
Cemented gravel Soft slate soft sedeimentary rock	-	-	-	6.0 - 7.00
Hard rock	-	-	-	10.0 - 15.0
Concrete	-	-	-	15.0 - 20.0

These figures are for depths of 3 feet and less: To get M.K.S. values, multiply by 0.3048

3.34.1 Assessment of seepage losses in canals - Recent thesis and literature

a) USBR : Moritz's formula - $S = 0.2\ c\sqrt{Q/V.}$ (FPS)

S = seepage loss in c/s per mile length of canal

C = depth on feet lost from canal prism in 24 hours. varying between 0.34 to 2.20 from Cemented gravel to sands.

Minimum value of $V = 0.8\, V_{max.}$

Width to depth ratios : to minimise adsorption and evaporation and for more economical design.

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Silt Loam	2.00	3.00	2.00	2.50 - 2.75
Alluvial silts when non colloidal	2.00	3.50	2.00	2.75 - 3.00
Ordinary firm loam	2.50	3.50	2.25	3.00 - 3.75
Fine gravel	2.50	5.00	3.75	4.00 - 5.00
Stiff clay (very colloidal)	3.75	5.00	3.00	4.0 - 5.0
Alluvial Soils when colloidal	3.75	5.00	3.00	4.0 - 5.0
Graded silt	4.00	5.50	5.00	4.0 - 5.0
Coarse gravel (non colloidal)	4.00	6.00	6.50	5.00 - 6.00
Shingle (non colloidal	5.00	6.50	6.50	5.0 - 6.0
Cemented gravel Soft slate soft sedeimentary rock	-	-	-	6.0 - 7.00
Hard rock	-	-	-	10.0 - 15.0
Concrete	-	-	-	15.0 - 20.0

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C = depth on feet lost from canal prism in 24 hours. varying between 0.34 to 2.20 from Cemented gravel to sands.

b) Vedirnikov's Method (USSR)

$q = K (B + (A + 2m) . H)$ $= K (Bs + AH)$

Where, Bs = surface width of canal
H = depth of water in canal
m = cos (α), α = side slope angle)
A = a function depending on B/H to be read from graph

c) U.P.Practice :
Q1 : 1/200 $(B+D)^{2/3}$ (MKS)
Q1 : Seepage loss in cumecs/kilometer length of canal

d) Punjab & Haryana:
$K1 = 1.9 Q^{1/6}$ (MKS)
K1 = Seepage loss in cumecs/million square meter of wetted area

e) Bhakra Nangal :
Lined $K1 = 1.25 Q^{0.056}$ (FPS)
Unlined $q1 = 0.00928 L (Q)^{0.5625}$ (MKS), L in kms.

f) Dyces formula:
$P + c .d^{0.5}$ (FPS) P = seepage loss per million square feet of wetted area
C = 3.0 adopted

g) Irgan's formula:
P = Cd . { WL / 10,00,000 }
P = S.L.in cusecs in a reach of length L in miles
W = width of water surface in feet
C = 3.5 adopted.

h) Practice in A.P.Projects:
Canals - lined 0.6 cumecs/million square metres of wetted surface
Canals - unlined : 1.85 to 2.40 cumecs/million square metres of wetted surface
20% extra for Disty systems.

(i) I.R.S. : Punjab :

S = Cad S = SL in m^3 /sec/10^6 m^2
= C $(B+D)^{2/3}/8$ = CL P $R^{1/2}$

lined canals:

i)	Mortar lining	SL =	0.061
ii)	Brick lining		0.051
iii)	Cement pointing in brick		0.188
iv)	Precast c.c.slabs		0.13
v)	B.W.in clay and cement plaster		0.30

j) C.B.IOP. 49th Meeting 1979 - Publication 133 -
S.L in Main canals 7% of head discharge
S.L in distributories & monors 8% of head discharge
S.L in water courses 20%

3.34.2 Estimated seepage loss in canals/km (in % of discharge allowed)

Type of soils/banks	Discharge through canal (m3 / sec) upto 1	1 to 10	11 to 100	over 100
Gravelley soils	20%	6%	2%	1%
Semi Pervious soils	10%	3	1	0.5
Clayey soils	6%	2	0.6	0.3
Rock cuts	2%	0.6	0.2	0.1
Lined concrete	0.2%	0.60%	0.62%	0.01%

Loss in old canals is 20 - 30% less than in new canals.

3.35 Loss of water from reservoirs due to evaporation in mm.

Jan, Feb .... 100 mm. each	June .... 175 mm. each
March .... 175 mm. each	July, Aug, Sept. . 150 cm. each
April .... 225 mm. each	Oct. .... 125 cm. each
May .... 250 mm. each	Nov, Dec. .... 100 cm each

Annual Total Loss = 1800 cm = 1.80 m

3.36.1 Channel Losses
Conveyance losses According to Etcheverry and Harding

Character of material	LOSSES Cft/Sft of wetted perimeter in 24hrs	Cusecs/Million sft of wetted perimeter (to nearest cusec)	Cumecs/million sq.m of wetted perimeter
1. Impervious clay loam	0.25 to 0.35	3 to 4	0.92 to 1.2
2. Medium clay loam under laid with hard pan at depth of not over 2 to 3 ft. below bed	0.35 to 0.50	4 to 6	1.2 to 1.8
3. Ordinary clay loam silty soil, or lava ash loam	0.5 to 0.75	6 to 9	1.8 to 2.7
4. Gravely or sandy clay loam, Cemented gravel, sand & clay	0.75 to 1.00	9.0 to 12.0	2.7 to 3.6
5. Loose sandy soils	1.5 to 1.00	17 to 20	5.2 to 6.1
6. Sandy loam	1 to 1.5	12 to 17	3.6 to 5.2
7. Gravelly sandy soils	2 to 2.5	23 to 29	7.0 to 8.8
8. Porous gravelly soils	2.5 to 3.0	29 to 35	8.8 to 10.6
9. Very gravelly soils	3.2 to 6.00	35 to 70	10.6 to 21.2

3.36.2 Canal losses : Serge Leliavsky - Design of Canals & barrages

a) Strange's approximation of canal losses : transmission losses in canals (FPS)

Q in canal in cfs	% of discharge which is lost per mile length of canal
> 100	0.25%
50 - 100	0.50%
25 - 50	1%
10 - 25	2%
< 10	4%

3.36.3 Bucklay : for each 100 cft issued at Head regulator:

15 cft/sec is lost in main canal
7 cft/sec is lost in distributories
22 cft/sec is lost in village water causes

Out of 56 cft, 27 is wasted by excessive irrigation or so and 29 cft is profitably used in irrigation purposes.

c) Moles worth formula : $QL = CLP \ m^{0.5}$ (MKS)

$m = H\ MD$, C = coefficient, 0.0015 for stiff clay and 0.003 for sandy soils or $K = C\ m^{0.5}$

3.36.4 Comparative Water Tightness of different types of linings (FPS/MKS) (I.C.I.D. PP. 306 - 307)

SN	Type of lining	Thickness FPS	Thickness MKS	Seepage loss cusecs/million sft of wetted perimeter: Initial FPS	Initial MKS	Minimum FPS	Minimum MKS	Final after cracking FPS	Final after cracking MKS
1	Unlined	-	-	22.4	6.71	4	1.2	-	-
2	Two layers of brick tiles with 1 ½" of 1:5:12 concrete sandwiched in between	5 1/8	13	0.63	0.192	0.12	0.037	2.10	0.64
3	As above with ½" of 1:5 cement mortar in place of 1:5:12 concrete	5 1/8	13	0.27	0.082	0.02	0.006	2.30	0.7
4	Concrete 1:3:6	4.00	10.00	0.22	0.267	0.01	0.003	0.42	0.128
5	Concrete 1:5:12	4.00	10.00	1.3	0.396	0.26	0.008	0.47	0.146
6	CC 1:4:8	4.00	10.00	-	-	-	-	-	0.135

3.37 I.S.1905 -1978/1983 (Code of practice) Structural Safety on shallow foundations:-

Pier foundations : (such as in CM-CD works etc.,)
Plain concrete pier - Height shall not be more than 12 times the least horozontal dimension.

a) Stresses allowable are as per those specified in IS 456 of 1979 provided the height of pier does not exceed six times the least horizontal dimension.

b) If height exceeds six times the least horizontal dimension, the maximum allowable stresses shall be determined by

fc' = fc (1.3 - H/20D) where
fc' = Maximum allowable stress
fc = allowable stress speficied in IS 456 - 1979.
H = Height of pier and
D = least horizontal dimension

Note: The above provision shall not apply for piers where the least horizontal dimension is 1.8m or more.

RCC piers: RCC piers shall be treated as axially loaded short columns and permissible loads calculated as per IS 456 - 1979 subject to:

i) Where the height of pier exceeds 18 times the least dimension, the max. load shall not exceed:

P' = P (1.5 - H/36D)
Where P' = permissible load on axially loaded short column
H = height of pier measured from top of bell if any, to the level of cut off pier, and
D = least dimension of pier

3.38 HEAD LOSS usually allowable in metres for hydraulic structures on Main Canals (while approving hydraulic particulars of canals)

a) C.M.works:

i) SL bridges .... 0.02 .... Normal section of canal retained.
ii) D.L.bridges .... 0.03 to 0.04 .... HL due to piers etc.,
iii) Cross regulators
with or without offtake .. 0.05 - 0.15........... depending on fluming ratio
iv) Trough Aqueducts .... 0.10 - 0.15........... depending on length of trough, velocity allowed and transitions

b) C.D.works:

i) Syphon aqueduts : HL : as per Unwin's formula Afflux as per Moles-worths formula

ii) Urainage syphons (non pressure flow) : If canal is flumed as in type II or III U.Is HL as per Manning's in transitions and barrel by TEL Method.

Restricted to 0.000 but actually it may go upto 0.03 depending on its length and fluming in canal.

iii) Super passages : Generally, normal section of canal is retained.

a) with Headway : HL is 0.00 but if canal is flumed for economic considerations HL in canal may go upto 0.02

b) Superpassages without headway : In major projects like N.S.P., S.R.B.C. etc., drainage bed comes below FSL of canal. The canal section is thus partially syphoned. HL is to be minimised in such cases. HL about 0.10m.

Methods:

a) Provide inlet-outlet arrangements if feasible, maintaining normal section of canal
b) Widen the canal at the crossing with necessary transitions
c) Depress the canal bed slightly below the barrel and give lift wall or reverse slope later to minimise regrading & losses.
d) Adopt both (b) and (c) above, if HL is still high

e) Compute HL in approaches with Mannings formula and under the partial syphon with Unwins formula and sum up.

f) Compare with afflux calculations by Moles worth formula

g) Adopt higher of (e) or (f) and prodive free board for trough of S.P. so that canal water does not escape into drain by spilling into it.

iv) Inlets\Outlets : Nil generally silt traps must be arranged and silty water let down into natural drain below.

v) Surplus escapes : Nil generally Silt traps must be arranged, for, the parent canal section gets reduced in course of time, resulting in heading up and lesser discharging capacity.

Note : During actual design, the HL is to be calculated by any retional method/model studies and restricted to a minimum by suitable design practices. If it is not reducable or restricted to that provided in the approved HPs, it has to be adjusted elsewhere with the consent of the sanctioning authority for the satisfactory performance of the canal with its structure.

3.39 CODE OF PRACTICE FOR DESGIN OF CROSS DRAINAGE WORKS IS : 7784 - 1983

Aquduct is defined as a cross drainage work in which the carrier channel is carried over the drainage channel and the bottom of the canal trough or the covering over the drainage openings is above the high flood level in the drainage channel.

3.39.1 TYPES OF AQUEDUCTS

(a) Depending on the shape of canal passing over the drainage, an aqueduct may be classified into the following three types.

(b) Type 1-In this type the canal continues over the drainage in its normal earthen section including the banks and earthen slopes. In this case the length of the culverts through which the drainage water is passed under the canal shall be sufficient not only to carry the water section but also to carry the earthen bank of the canal with their slopes.

(c) Type 2-In this type also the canal continues in its earthen section over the drainage, but the outer slopes of banks are replaced by retaining walls, thereby reducing the length of drainage culvert by that extent.

(d) Type 3-In this type the earthen banks are discontinued over the drainage and the canal water is carried in masonry or concrete trough, box, barrel, pipe or any other suitable section. The sides of the trough are connected on either side of the work to earthen banks of the canal by means of wing walls. Generally the canal is flumed to effect economy in this type .

3.39.2 The choice of the type of aqueduct would depend on considerations of economy which in turn would depend mainly upon the size of the drainage to be passed in rèlation to the size of the canal and the foundation strata.

(a) Over a small drainage, aqueduct of Type 1 may be suitable as nominal wings would more than compensate the increased cost due to the length of drainage culverts which have maximum length (across the canal) in this type.

(b) Over a large river an aqueduct of Type 3 may be more economical as the length of drainage culverts across the canal is minimum and the savvings made in cost of drainage culverts would be greater than the increased cost of canal and drainage wing walls.

(c) For intermediate conditions, Type 2 may work out economical in comparison to Type 1 and 3.

3.39.3 Discharge through the aqueduct

$Q = cb\, 2g[\ (2/3).\ \{he+(v_1^2/2g)^{3/2} - (v_1^2/2g)^{3/2}\} + d\ (\ he + v_2^{\ 2} / 2g)^{1/2}]$

$he = c\ [v^2 - v_1^{\ 2}/2g]$, v = allowable velocity in the duct
$= 2\ v_1$ or 1.5m/s whichever is min

Q = Discharge
d = depth of water
v_1 = velocity in canal
he = head lost at entry
b= mean width of aqueduct
c = co-efficient of discharge which varies from 0.9 to 0.95

The co-efficient C will be 0.5 for sharp cornered entrance
0.25 for round cornered entrance
0.1 for ordinary design
0.05 for bell mouth flumed entrance

3.39.4

Design of syphons : $Q = AV$

A = Area of barrel

V = Mean velocity in the barrel

H = Fall in meters

f1 = Co-efficient for loss of head due to friction at entrance = 0.088 for bell mouth; 0.505 for cylindrical mouth

f2 = Co-efficient for loss of head in overcoming surface friction in the barrel.

3.39.5 LAYOUT

(a) The layout of aqueduct shall be so fixed that it shall preferably be in a straight reach of drain/stream, which crosses the channel at right angles as far as possible.

(b) Bank connections to canal and drain/stream should be provided depending upon the properties of the soil available in the area.

(c) Transitions for canal should preferably be provided with 2 : 1 and 3 : 1 on upstream and downstream side but not flatter than 3 : 1 and 5 : 1 respectively.

(d) Wing walls of drainage should be suitably connected to high ground. But facing walls should be so located that height should be comparatively lower to reduce cost. Wing walls of drainage in Type 1 and 2 should be higher than high flood level, H.F.L.

(e) The drain/stream should be directed towards the structure by suitable training work like training walls, guide banks spurs, etc. The canal banks adjacent to work should be protected by suitable protective measures such as turfing, pitching, and launching apron whenever necessary.

3.39.6 CANAL SECTION AND ITS FLUMING

The canal embankment adjoining the aqueduct should have adequate provisions to avoid possibility of any breach and to minimise seepage. The outer slope of

bank should have a clear cover of 600 to 900 mm over the designed phreatic line (often referred to as hydraulic gradient, see IS : 7894 —1975 for details). High bank (say, of more than 6m height above ground level) should be checked for slope stability (see IS : 7894-1975) and normal provisions of filter and rock toe should be made. In cases where difference between the canal full supply level and HFL is small, canal bank should be checked for the condition when drain is in high floods and the canal is dry.

Note 1 - Free board should be provided for trough as per ISI Code

Note 2 - Road way width should be provided as per I.R.C.Code.

Note 3 - Central wall of trough may be omitted by providing R.C.C. colms on each pier based on techno economic studies.

(b) The canal may not be flumed to less than 75 percent of bed width. However, where the velocity and loss of head permit, greater fluming may be allowed ensuring subcritical flow condition.

(c) The flumed portion of the trough shall be joined with the normal section with proper transitions to minimize head loss at entry and exit. The transition should correspond to a minimum splay of 3 : 1 on the upstream side and 5 : 1 on the down stream side. It should be ensured that the flow follows the boundaries of the wings.

(d) The vertical slope approaches may not be steeper than 1 in 3 on the entry side and about 1 in 4 on the exit side. The minimum permissible velocity allowed in the barrel depending upon the size of the bed meterials in the drain may be derived from Tables 1 and 2 in IS : 7784 (Part 2/Sec.5) - 1980. For Type 1 and 2 a water seal of 1.5 times the change in velocity head with a minimum of 100mm is allowed over the tope of the barrels at start to prevent air entering the barrel.

(e) In exceptional cases, the limiting velocity may exceed 3m/s provided this does not cause excessive loss of head and the structure can withstand the higher velocity and abrasion.

(f) The head loss through the barrels shall be calculated in accordance with IS : 7784 (Part1) - 1975. The waterway shall be so adjusted that the afflux does not exceed the limits of submergence tolerances of the environments.

(g) In drainages when the course is undefined, as sometimes it happens in bouldery or alluvial regions, fluming from 60 to 90 percent of the Lacey's waterway may be tried.

3.39.7 Joints.

(a) Joints shall be provided across and along the barrel length. The maximum spacing of these joints in either direction shall be limited to 20m. A gap of 10 to 15mm with water stops at all the joints accross and along the barrel should be provided to accommodate the movements. The position and details of joints shall be in accordance with 10 of IS : 7784 (Part 1) - 1975.

(b) In the case of barrels resting on compressible soils, collars encircling the plain joint should be provided. This will protect the waterstop from vertical shear due to excessive settlement. In the case of syphons of multiple barrels more than one unit, the collars may be designed to be flexible.

(c) In case of multibarrels, units of three or four barrels can be adopted side by side with longitudinal joints having water stops all around. Water stops shall also be provided at the junction of RCC barrels with transition walls.

3.40 Dam Safety : (Small dams)
(Engineering Foundation conference - ASCE (1852) Jan. 1975 - England)

3.40.1 Phase.I Field inspection - evaluate - design and performance data

Give particular attention to :

a. Evidence of distress
b. Excess leakage and seepage
c. slope instability
d. excessive settlement
e. tilting and cracking of concrete structures and
f. other physical features indicating instability.
g. Review of design and adequacy of design parameters
h. Construction - quality etc.,
i. Loads - Experienced vs designed
j. Evaluation of geological and foundation conditions
k. Changes in technology since original design

l. Seismology - check w.r.t. seismic considerations

Phase.II Investigation - Field sampling - Lab testing - stability analysis.

Give attention to:
Adversely oriented joints
Slicken sides or fissured material, faults, seams or soft materials - week layers - excess pore water pressures - clay shales etc.,
Dam can be unsafe by failure due to :
Overturning, foundation failure, Uncontrolled seepage, shear failure etc.,

3.40.2 Recommended factors of safety : Embankments:

Case	Leading condition	Factor of safety	shear strength
I	Sudden dwawdown from spillway crest or top of gates to MDDL	1.2	minimum composite of R & S shear factors
II	Partial pool with assumed horizontal steady seepage saturation	1.5	R + S/2 for R < S
III	Steady seepage from spillway crest or top of gates with KH/KV = 9 assumed	1.5	S for R < S
IV	E.Q (cases II and III with seismic loading)	1.0	same as case. II

3.40.3 Failure of Dams : Due to :

a) Seepage failure : about 1/3 to 1/2 of the dams (analysed) studied have failed due to Seepage.

b) Structural failure : about 30% of the cases (less in dams below 50' height)
c) Construction inadequancies (more in dams below 50')
d) Maintenance - inadequate -
f) Inspection - improper or insufficient.

3.41. NAGARJUNASAGAR DESIGN PRACTICES

3.41.1 Earthwork - Banking - partial banking - side slopes standards - 1½ : 1 both faces

Soil groups	Height above GL
a) GC, GM, SM, CL, CH	5 m
b) GC, SC, CL, CH	6 m
c) GC and SC	8 m

If foundation soil is weaker than fill material, provide flatter slopes of 1 3/4 : 1 to 2 : 1 in cutting and banking, on water face or berms at GL.

3.41.2 Duties - under offtakes:-

Area under offtakes hectare / Acre	ID cum wet crops Ha / cumec	ID Ha/cumec / Acre/cusec	Wet crops only Ha/cumec / Acre/cusec
12000 / 30000	850 / 60	850 / 60	715 / 50
12000 to 4000 / 30000 to 10000	900 / 62	925 / 65	760 / 53
4000 to 40 / 10000 to 100	1000 / 62	1000 / 70	800 / 56
40 and below / 100 and less	1400 / 100	1075 / 75	850 / 60

The requirements of I.D (khariff) is about 2½" per irrigation at an interval of 10 to 12 days which is equivalent to 100 Ac / cusecs at field or 60 acres/cusec at the head of distributory.

The requirements for khariff paddy (wet) is about 4" in a 10 day period which worksout 60 acres / cusec at field which corresponds to 50 acres / cusec at the head of distributory.

3.41.3 Design of channel section: Due to reservoir fed channels, the limiting velocity is non scouring rather than not silting in upland areas.

Kennedy's $V_o = Cd^{0.50}$ (USBR) (FPS)
S.R.S.P. also adopted the same.

a) C : 0.84 for BC, silty or softer soils - 0.47 in MKS
b) C : 1.10 for muram, gravelly and harder soils - 0.60 in MKS

Note: C = 0.50 for canals where Q is less than 2.83 cumecs (100 c/s) for all soils uniformly.

3.41.4 Direct pipes for discharges of 3 cumecs and below for better control over irrigation water utilisation:

3.41.5 Assumptions under safe bearing capacities of soils:-

a. B.C. soils .... 8 tonnes/m^2
b. HG lime kankar .... 30 tonnes/m^2
c. SDR .... 35 tonnes/m^2
d. HDR .... 50 tonnes/m^2
e. F & F rock .... 60 tonnes/m^2
f. HR and sheet rock .... 100 tonnes/m^2

3.41.6 Tension in CRS masonry in CM (1:5) - upto 0.70 kg/cm^2 (7 t/m^2) or 10 lbs/sq. inch. and for CRS masonry in CM (1:6) tension upto 5 to 8 psi may be allowed for abutments, wings and returns.

3.41.7 Structural design constants : using HYSD Tor 40 bars

M20 (1 : 1½ : 3) mix.	**M15 (1 : 2 : 4) mix.**
(a) Stress in steel on both faces of thin members and on tension face of (thin or thick) slabs in contact with water=1300 kg/cm^2	stress in steel for road bridges=1900 kg/cm^2 for other structures bars above 20mm=2100 kg/cm^2 upto 20mm = 2300 kg/cm^2

b. On face away from liquid face
of thick member = 1790 kg/cm^2

Stress in concrete = 65 kg/cm^2
shear stress = 6.5 kg/cm^2
Local bond stress = 16.50 kg/cm^2
Modular ration m = 14
Cracking stress in concrete = 16.5 kg/cm^2

Stress in concrete=50 kg/cm^2
Shear stress = 5 kg/cm^2
local bond stress = 14.5 kg/cm^2
Modular ratio m = 18 kg/cm^2

3.41.8 Replacement of cement by fly ash:
a. CC (1:4:8) or (1:3:6) to the extent of 25% by weight
b. CM (1:5) to the extent of 20% by weight

3.41.9 Foundation concrete CC (1:4:8) :
For cut offs or foundations etc., 40% of 40 mm size and 60% of 75 mm size aggregate shall be used.

3.41.10 Wearing coats:
a. Plain - M 15 grade 75 mm thick using 20 mm size aggregate in barrel floors and transition floors monolithic with foundation concrete.
b. M 15 grade - 150 mm thick using 10 mm nominal steel @ 30 cm c/c both ways in cross regulator floors - monolithic with floor concrete.
c. M 20 grade 40 mm thick in trough slabs
d. M 20 grade 'Tell-tale' coloured CC cones in wearing coat over deck slabs of bridges.

3.4111 Bearing surface: Bearing surfaces of piers and abutments over which RCC slabs are laid shall be rendered perfectly smooth in CM (1:3) and thick kraft paper shall be laid over them.

3.41.12 Weep holes: 75 X 150 mm spaced at 1500 mm c/c staggered both horozontally and vertically shall be provided.

3.41.13 Expansion joints: 12 mm thick with pvc or rubber water stops shall be provided in RCC deck slabs over abutments and piers and filled with mastic filler.

3.41.14 Holding down bolts: May be spaced at 900 mm c/c to anchor RCC slab under water trough to piers and abutments, where the slab is continuous. The spacing may be increased to 1800 mm c/c in the case of piers and abutments supporting free ends of the slab.

3.41.15 Drainage spouts: 50 mm dia and 100 mm dia. drainage spouts shall be provided for single and double lane bridges respectively at the centre of span on both sides of bridge decking.

3.41.16 Approach slab for road bridges: M 15 grade, 15 cm. thick slab on well consolidated muram of 225 mm thickness with 12 mm bars @ 15 cm c/c both ways at top and bottom resting on the dwarf wall of the abutment and extending for a length of not less than 3.66m into the approach may be provided as stipulated in clause 217-3 of IRC bridge code sec.II. in which case live load surcharge need not be considered in the design of bridge abutment.

3.41.17 Guard stones: 20 X 15 X 125 cm long guard stones shall be provided either side of road approaches at 300 cm c/c for the portion of the road where the height of embankment is more than 1.80m.

3.41.18 Pressure relief holes: Pressure relief pipes of 50 mm dia. shall be provided in the floor of cross regulators and aqueducts at 3m c/c staggered with 60 X 60 X 60 cm filters in no fines concrete below.

3.41.19 Radii of curvature: Minimum radius of curvature be adopted is 20 times the width of water at FSL, as the relevant parameter is the water width rather than bed width of. canal.

3.41.20 Normal canal section in full cutting reaches:

a. Width of inspection paths 4 m with side drain

b. side drain 30 cm. bottom width and 60 cm depth

c. clear distance between edge of drain and the spoil bank may be kept as one metre.

d. Inspection path shall not be more than 9 m higher than canal bed. If a deep cut, inspection path may be located at FSD + 1 to 1 1/2m.

3.41.21 Eddylosses at structures: Coefficients of losses:
Transitions - splays in wings 2 1/2 : 1 to 3 : 1 at water surface. Curved wings for the inlet transition of offtake to ensure smooth flows. Warped transitions with face vertical at the beginning and same batter as slope of canal at d/s transition for the outlets.

Type of transition	Coefficient for transition losses Inlet	Outlet	losses in barrel Extrance	Exit
1. Smooth transition				
a. O.T.sluices	0.10	0.20	.050	1.00
b. U,Ts	0.10	0.20	0.10	0.20
c. Aqueducts	0.10	0.20	0.25	0.25
d. Square ended type	0.30	0.75	-	-
2. For earth transition connecting a canal section to a pipe and pipe structure without concrete transitions for submerged condition	0.50	1.00	-	-
3. Sudden expansion and contraction	0.25	0.25	-	-

Note: All coefficients are to be applied to the differential velocity heads before and after transitions.

3.41.22 Drops on distributories - (Core wall type drops have failed badly)

(a). In Muram, SDR, HDR, etc corewall type for Q less than 3 cumecs and height of drop less than 1.5 m core walls be kept 0.50 m above FSL and keyed 1 m into firm ground.

(b). Silty/clayey soils:

i) drops greater than 1.5 m for all discharges	wing and returns be invariably provided
ii) Drops less than 1.5 m abd Q > 1 cumec	wings and returns
iii) Drops less than 1.5 m and Q < 1 cumec	core wall type

3.41.23 Specifications for consolidation of banks and key trenches

Sl.No	Q	FSL of branch or distributory	Type of consolidation	Specification of key trenches
1	> 5000 c/s	FSL above GL	P.R. consolidation upto 1' above FSL	One main trench of 6'x2' with 1/4 to 1 side slopes in centre of banks slightly towatds the water side and key trenches of 2'x2' with 1/4 : 1 side slopes at 10' c/c on either side of main trench
2	Between 5000 c/s to 1500 c/s	i) FSL above GL or upto 1' above GL	SS 20A	Normal key trench of 3'x2' with 1/4 : 1 side slopes at centre of bank and 2'x2' key trenches with 1/4 : 1 side slopes on either side at 10' c/c.
		ii) If FSL is 1' to 10' above GL	Stone Roller upto 1' above FSL	K.T. is in (1)
		iii) If FSL is more than 10' above GL	Power Roller upto 1' above FSL	Same as in (1)
3	Below 1500 c/s	FSL 4' above GL FSL 4' to 10' above GL FSL beyond 10' above GL	SS 20A S.R. P.R.	Same as in 2(1) Same as in (1) Same as in (1)

Key trenches need not be provided within 3' from the toe of the bank. Proctor's density of 95% for consolidation with PR and 90% with SR.Extra quantity for consolidation by the PR and SR is 10%.

3.42 Requirements of labour, material and machinery in project execution : CANALS -

	Structures of value Rs.1. lakh	Earthwork of value Rs.1 lakh
Personnel work charged	100	100
Skilled	600	10
Semi-skilled	2000	8000
Unskilled	2000	100
b. Principal materials -		
Gelatine	200 kgs	1400 kgs
Detonators	2000 nos	14000 nos
Drill rods	4 nos	26 nos
Cement	60 MT	-
Steel	7 MT	-
c. Machinery -		
Jack hammer	50 Hrs	900 Hrs
Compressor 7 cum/minute (250 cfm capacity)	25 Hrs	450 Hrs
d. Proportionate costs -		
Labour : Materials : Machinery	33 : 66 : 1	60 : 20 : 20
e. Cost of structures Vs E/W -		
Main canal	40%	60%
Branch canal	45%	55%
Major disty.	50%	50%
Minor disty.	60%	40%

Note: The rates are as per 1976. Necessary cost escalation may be added.

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Chapter IV

HYDROLOGY

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4.1.0 WATER AVAILABILITY STUDIES : DATA REQUIRED :

1. Catchment Area Plan.
2. Location of raingauge stations.
3. Theissen Polygon of influencing stations in and around catchment. With this influencing areas of each Raingauge Stations are arrived.
4. Monthly rainfalls for the influencing stations.
5. Gauged flows at Project site.
6. Up stream utilisation month wise.

4.1.1 STEP BY STEP PROCEDURE :

1. From the above data monthly or monsoon weighted average rainfalls are arrived.
2. By adding upstream utilisations to gauged flows gross yields are arrived. From gross flows and weighted average rainfall relationship is arrived. The same relationship is used to arrive at gross flows where weighted average rainfall is available and gauged flows or upstream utilisations or both not available.
3. Net yields are arrived by deducting upstream utilisation in proportion to dependable yield that is whenever the yield is more than dependable yield complete upstream utilisation is deducted and where it is less prorata deduction may be resorted to.
4. In the case where observed yields are available observed yields may be implemented.

   In case if no particulars are available, a parallel catchment rainfall runoff relationship is used to estimate the yields for the catchment under consideration.

   The annual yields from the catchment both generated and observed are arranged in descending order the 75% dependable yield may be arrived which will be considered as representative yield for that catchment.

4.1.2 RAINFALL - RUNOFF RELATION SHIPS :

1. Bivariate. 2. Multivariate.

1. BIVARIATE :

Monthly rainfall of the same month Vs discharge.

(i) Linear relation ship

$Y = ax + b$ a & b are constants.

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X = Rainfall in inches/Cm., Y = Discharge in inches/Cm.

(ii) Quadratic relationship :

$Y = ax^2 + bx + c$ a, b, and c are constants.

(iii) Exponential relation ship :

$Y = ax^b$ a nd b are constants.

$\underline{\text{Log } Y} = \underline{\text{Log } a} + \underline{b \log x}$

P A Q

$P = A + bQ$

Now this is reduced to linear relationship and may be solved as above.

4.1.3 MULTIVARIATE :

In this the current month rainfall and previous month rainfall is used.

$Z = ax + by + c$

X = current month rainfall. Y = previous month rainfall. Z = runoff.

a, b and c are constants.

By solving the above values of a, b, c are obtained and expected values of Z are calculated.

An example on rainfall Runoff linear relationship is here with given. By substituting representative rainfall of catchment in the linear equation Runoff for the Period can be arrived.

Year	Rain fall X	Runoff Y	X^2	XY
1967	23.89	2.90	570.73	69.28
1968	37.15	15.18	1380.12	563.94
1969	38.82	13.40	1506.99	520.19
1970	26.26	3.90	689.59	102.41
1971	27.25	4.77	742.56	129.98
1972	46.66	16.88	2177.16	787.62
1973	31.42	6.88	987.22	216.17
1974	41.69	13.35	1738.06	556.56
	$\sum X$=273.14	$\sum Y$=77.26	$\sum X^2$=9793.43	$\sum XY$=2,946.15

$Y = aX + b$

$\sum Y = a \sum X + nb$

$\sum XY = a \sum X^2 + b \sum X$

Substituting the values of $\sum X$, $\sum Y$, $\sum X^2$ and $\sum XY$

77.26 = 273.14a + 8b; .... (1)

2946.15 = 9793.43a + 273.14b ....(2)

Solving the above equations,

a = 0.6591, and b = -12.8464

The equation is given by

Y = 0.6591X - 12.8464

Regression coefficient is given by

$$Rxy = \frac{\sum XY - n X Y}{(\sum X^2 - nX^2)(\sum Y^2 - nY^2)}$$

substituting the values

Rxy = 0.96 for the above example.

4.2.0 ESTIMATION OF DESIGN FLOOD

Estimating design flood for spill ways may be Probable Maximum Flood (PMF) based on Probable Maximum Precipitation (PMP) or Standard Project Flood (SPF) based on standard Maximum Precipitation (SMP). PMP is an estimate of the Physical upper limit to storm rainfall over a basin. It is generally determined by studying the rainfall of past sever Storms tin the region and increasing this rainfall in accordance with meteorologically possible increases in the atmospheric factors contributing to storm rainfall. Standard Project Precipitation is the largest precipitation characteristic of the region in which the reasonably basin lies or the largest storm which has occurred in the region of the basin during the period of rainfall records. In case of gauged catchment the PMF at the Project site is worked out where the maximum capacity of the reservoir is more than 50,000 ac.ft (2.178 TMC or 61.673 M cum).

4.2.1 UNIT HYDROGRAPH METHOD FOR GAUGED CATCHMENTS.

The Unit Hydrograph Principle is defined as a hydrograph of direct runoff resulting from 1 Cm. (or 1") of effective rainfall of a specified duration generated uniformly over the basin at the uniform rate. The basic data required for Unit hydrograph method of flood estimation is the hourly / daily data of 2 or 3 flood events occurred in the catchment and the corresponding rainfall data of the all stations influencing the catchment. It is also necessary to have the hourly rainfall data from SRRG station available in the catchment or in the neighboring catchment for the concurrent flow events periods selected. If adequate data are not available for the river in question, the data available for a number of sites on stream in the hydrometeriological region can be utilised for this purpose.

From the observed flood event a trial unit Hydrograph (UG) is derived with the following parameters :

1. Unit Duration
2. Peak of UG
3. Base flow of UG
4. Time to peak

Note : The volume of the Unit Hydrograph is equal to 1 cm run-off from the catchment.

The hourly rainfall increments are calculated and the ordinates of DSROH (Direct Surface Run-off Hydrograph) are computed and these are to be adjusted so that the volume hydrograph is equal to 1 cm runoff. The Unit Hydrograph is finalised.

The storm in the catchment is selected either from the past rainfall records as collected from IMD, for which the hourly distribution is adopted based on the SRRG station available nearby, preferably in the catchment. A clock hour correctness of 13% may be applied for one day storm. The maximisation factor of 15% to 20% may also be applied to storm depth values. The infiltration losses @ 0.05"/hour may be deducted from the rainfall increments which are worked out based on storm distribution. Depth duration curve is to be drawn and the rainfall increments for the required unit time duration are to be read from depth duration curve. The rainfall increments are to be arranged in the critical sequence by arranging the peak rainfall increment against the peak ordinates of Unit Hydrograph and so on. The order of increment are to be reversed such that the same are to be multiplied with the UG ordinates to workout the flood storm hydrograph. The quantum of 5 c/s persq.mile are to be added to the maximum out flow as base flow and hence the PMF is worked out. The limitation of this study is when the CA of project in question does not exceed 5000 sq.km. In case if the CA is large it is to be divided into different region for which the flood storm hydrograph is to be calculated separately. Allowing the lag time, the above flood storm hydrograph ordinates are to be compounded and the PMF is to be worked out as per the procedure worked out above.

In case of ungauged catchments the design flood is worked out as per the methods indicated below :

4.2.2 SYNTHTIC U.G. : The Synthtic UG is developed based on known physical characteristics of the basis. A reference UG is to be selected either from the catchment or near by catchment having the same hydrometeorological characteristics. The synders constant Cp, Ct are to be calculated from reference UG

$t_p = Ct\ (LLc / s^{0.5})^{0.38}$

$t_{PR} = t_p + (t_R - t_r)/4$

$q_{pr} = 640cp / t_{PR}$

Generally, the values of Ct = 1.2 for mountains drainage area
= 0.72 for foot hill areas
= 0.35 for valley region

The synthetic UG is to be derived with the following parameters

1. Unit duration, t
2. Peak of UG QP
3. Base period tb
4. Time to peak tp

The ordinates as read from the above UG are to be multiplied with the net rainfall increments as explained above on the flood storm hydrograph are to be worked out from which the PMF for the unguaged catchment can be assessed.

4.2.3 TRIANGULAR UNIT HYDROGRAPH : This method is applicable for the unguaged catchment having the area upto 200 sq.miles where the observed data for the flood period is available. The empirical formula is as follows :

Tb = 2.67 tp

tp = D/2 + 0.6 tc

where, $tc = [11.9\ L^3 / H]^{0.385}$, qp = 484 Aq / tp

D = Unit duration, tp = time is in hours from starting to peak

tb = time base of triangular UG

With the above parameters triangular UG is derived with the selected storm and its distribution as explained in PMF method the flood storm hydrograph is to be derived from which the Maximum flood can be worked out.

4.2.4 ESTIMATION OF DESIGN FLOOD BY SMALL CATCHMENT METHOD AS PER CWC REPORTS:

The estimation of design flood for small catchment and medium catchment having the size from 25 to 2000 sq.km based on synthetic UG is explained in detail in the flood estimation reports for small catchments for sub basins. By Directorate of Hydrology (small catchment) CWC. The report nos. 3g, 3f, 4a, 4b, 3h, 3e covers A.P. state and these reports may be referred to in estimating the design flood for small catchment.
The physiographic parameters that are considered in the above study are
L, Lc, s = (Statistical slope) Ap. Am, qp, TB etc. With the UG is drawn and the UG ordinates are read from the unit Hydrograph base flow is added to the UG Ordinate. The storm can be selected from the isopluvial maps published in the report. The procedure for obtaining the rainfall increments is also appended in the report.
A typical example of flood calculation by
(1) flood estimation for small catchments method as per CWC reports and (2) Triangular unit Hydrograph method for Maddileru Project are appended.

4.2.5 Example

Report accompanying designflood calculation of Maddileru reservoir at Adavibrahmannapalli Ananthapur district.

Maddileru river originates from somraju Hills situated at Thangedukunta and Allepalli village limits and flows towards north easterly direction up to kadiri and takes a turn to northerly direction until it join chitravathi river. The length of the river up to the proposed reservoir at ADAVIBRAHMANPALLY (v) IS 69.33 km. This catchment area upto the reservoir is 529.04 SQ.MILES OR 1371.32 SQ.KM. The centre of gravity of the catchment is determined graphically and it is project to the river and the length of the river from this point to the reservoir site (LC) is measured as 40.65km. The statistical slope of the reservoir (S) is arrived from S.I. sheets as 13.53 (Table 1- Design flood is calculated based on the design office report of C.W.C.No.. K/6/1982of the sub zone 3-h of Krishna and Pennar basin. The various parameters of synthetic unit hydrograph is arrived as per formulae given in the report and the unit hydrograph is drawn. (Fig.2). Table-2 show the synthetic unit hydrograph ordinates. The maximum rainfall of the catchment is determined from the isopluvial maps of 50 years return period for 1 hr, 3 hour, 6hr., 9hr., 12,. 15hr,. 18 hour and 24 hour duration passing through the catchment area and the value for TD=7 hour duration is read as 143.3 mm. or 14.33cm. This point rainfall is converted to areal rainfall by multiplying with a reduction factor of 0.77 based on the report which is 14.33 x 0.77= 11.03cm. These calculations are shown in table-3. Table-4 shows the time distribution in put storm. A design loss rate of 0.5 cm./hr. assumed as suggested in the report. The rainfall excess unit are shown in col.4 of table-5. Computation of peak flood is shown in table-6. Computation of design flood hydrograph is shown in table-7. Thus the design flood is arrived as 3,940.75 cumecs or 1.40 laks cusecs.

M.F.D.Calculations of Maddileru:-

1) Basic Data :-

a) Sub Zone :- Krishna & Penna basins (sub zone -3 h)

b) Name of site :- Meddileru reservoir scheme across Maddileru near Adivibrahamana palli, H/o Dorigallu) in Kadiri (Tq.) Anantapur district.

c) Name of the river :- Maddileru river

d) Topography :- Moderate

e) Shape of C.A. :- Triangular

2) Collection of Information from contour plans :-

L = 69.33 KM

Lc = 40.65 KM

C.A. = 1371.32 sq. Km

Latitude = 14° - 21' - 25" North

Longitude = 78° - 3' 13" East.

2.1 Calculation of statistical slope :-

Refer table - I for calculation of statistical slope S = 13.53

3) Calculation of synthetic unit hydrograph :-

$$LLc/\sqrt{S} = 69.33*40.65/\sqrt{13.53} = 766.25.$$

Draw Unit hydrograph on a graph paper with the above known values of tp, tm, qp, w50,

TABLE - 1 COMPUTATION OF STATISTICAL SLOPES(S)

Sl. No	Distance from the site to the bed level of stream measured along the main stream in KM	The level at the point of intersection in the RL which the bed level represented in M	The length of the segment difference between consecutive length in KM (Li)	The difference between RLs of preceeding and succeeding levels in Col.2 M Si	Slope of the Segment (Si) Col.5/Col4 M/ KM	Li/ Si
1	2	3	4	5	6	7
1	0	1159		41	19.52	0.475
2	2.1	1200	2.1	50	79.37	0.071
3	2.73	1250	0.63	50	27.73	0.342
4	4.53	1300	1.8	50	35.71	0.234
5	5.93	1350	1.4	50	38.46	0.21
6	7.23	1400	1.3	50	14.29	0.926
7	10.73	1450	3.5	50	35.71	0.234
8	12.13	1500	1.4	50	5.95	3.444
9	20.53	1550	8.4	50	10	1.581
10	25.53	1600	5	50	20	0.559
11	28.03	1650	2.5	100	12.19	2.349
12	36.23	1750	8.2	50	16.67	0.735
13	39.23	1800	3	100	19.23	1.166
14	44.43	1900	5.2	50	9.09	1.824
15	49.93	1950	5.5	50	11.36	1.306
16	54.33	2000	4.4	50	50	0.141
17	55.33	2050	1	50	12.5	1.131
18	59.33	2100	4	50	21.74	0.493
19	61.63	2150	2.3	50	12.5	1.131
20	65.63	2200	4	50	31.25	0.286
21	67.23	2250	1.6	250	119.05	0.192
22	69.33	2500	2.1			18.85
TOTAL			69.33			

wR50 w75 and TB, to get the surface runoff 1.00 cm. fig. 2 shows the synthetic U.G.H. The ordinatesare noted in table -2 columns.

$$S = \frac{L^2}{(Li/Si)^2} = (69.33/18.85)^2 = (3.678)^2 = 13.53$$

$$L_c = 25.25 \text{ miles} \times 1.609 = 40.65 \text{ KM}$$

$$LLc/\sqrt{s} = 69.33*40.65/\sqrt{13.53} = \frac{69.33 \times 40.65}{3.678} = 766.25$$

$$tp = 0.258\ [L.Lc/\sqrt{s}\ ]^{0.49}$$
$$= 0.258 \times 25.902 = 6.683$$

$$t_p = 6.683 \text{ or say } 6.5 \text{ hrs.}$$
$$t_m = 6.5 + 0.5 = 7.00 \text{ hrs.}$$
$$q_p = 1.017\ (tp)^{-0.52}$$

$$q_p = \frac{1.017}{6.683^{0.52}} = \frac{1.017}{2.685} = 0.38$$

$Qp = q_p \times A$

$Qp = 0.38 \times 1371.32 = 521.01$ or 521.00 cum.

$Qp = 521$ cumecs.

$W50 = 2.396\ (q_p)^{-1.08}$

$$W50 = \frac{2.396}{(0.38)^{1.08}} = \frac{2.396}{0.352} = 6.619$$

$W\,50 = 6.81$

$$W75 = 1.427\ (qp)^{-1.08} = \frac{1.427}{(0.38)^{1.08}} = 4.054$$

$W75 = 4.054$

$$WR\,50 = 0.750\ (q_p)^{-1.25}$$
$$= \frac{0.750}{(0.38)^{1.25}} = 2.51$$

$WR\,50 = 2.51$

$$WR\ 75 = 0.557\ (q_p)^{-1.12} = \frac{0.557}{(0.38)^{1.12}} = 1.648$$

$WR\,75 = 1.648$

$$T_B = 7.193\ \ (tp)^{0.53}$$
$$T_B = 7.193 \times (6.683)^{0.53} = 7.193 \times 2.737 = 19.687$$
$$T_B = 19.687 \text{ or say } 20 \text{ hrs.}$$

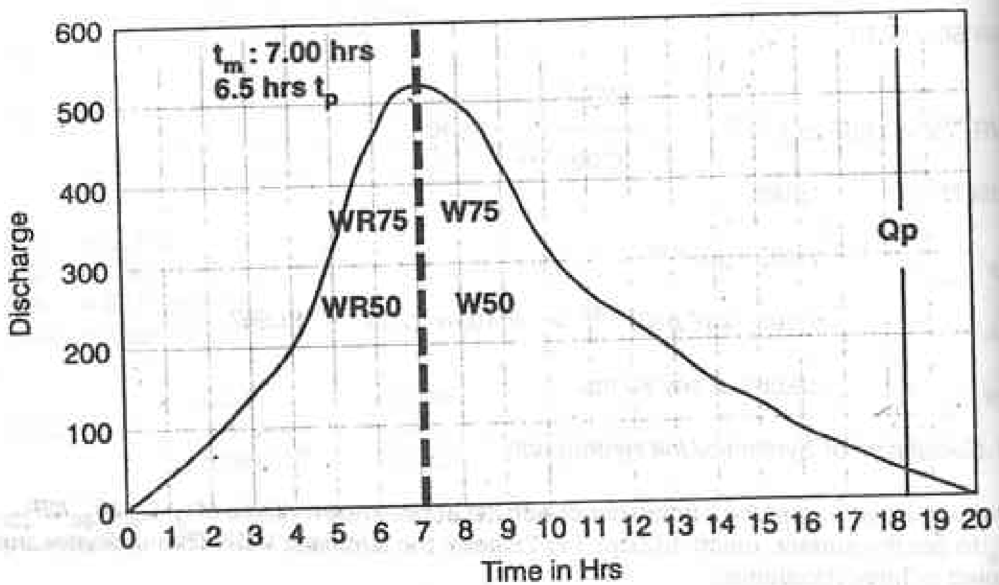
3. Calculation of Synthetic Unit Hydrograph

Draw Unit hydrograph on a graph paper with the above known values of t_p, Q_p, W_{50} WR_{75}, T_b to get the surface runoff 1.00cm. Fig 2 shows the synthetic UGH. The ordinates are noted in Table-2 columns.

4) Computaion of Design storm input :-

TD = Design storm duration
TD = 1.1 x tp = 1.1 x 6.683 = 7.35 hrs. or 7.00 hrs.
TD = 7.00 hours.
BASE FLOW :- The average baseflow of 0.5 cumecs/sq.km.
Base Flow : C.A. x 0.05 = is recommended for adoption in the subzone
= 1371.32 x 0.05
Base Flow = 68.57 cumecs.

FLOOD HYDROGRAPH



a) 50 years Td - hour point rainfall :-

Locate the catchment, under study in plates 8a to 8 h of the C.W.C booklet for subzone - 3 h and read the point rainfall values for difference durations. Enter the point rainfall values in col.2 against the various duration in col.1. Inter polate point rainfall value for 7 hour and enter in col.3 (Table.3)

TABLE - 2 SYNTHETIC UNIT HYDROGRAPH ORDINATES

Sl.No	Time in Hrs	Synthetic U.G. Ordinate	Representative U.G. Ordinate
1	0	0	
2	1	40	
3	2	86	
4	3	140	
5	4	200	
6	5	320	
7	6	460	
8	7	521	
9	8	498	
10	9	420	
11	10	320	
12	11	260	
13	12	224	
14	13	186	
15	14	146	
16	15	120	
17	16	86	
18	17	64	
19	18	40	
20	19	20	
21	20	0	
		4151	

Run off = 4151x0.36/1371.32 = 1.09

TABLE - 3

DEPTH DURATION

Sl.No	Duration	Point rainfall value of 50 year return period storm from IMD Maps plates (mm)	Read from Depth Duration
1	2	3	4
	1	70	
	3	140	
	6	140	
	9	150	143.3mm -0.143 m
	12	160	or
	15	160	14.33 cm.
	18	160	
	24	180	

i.e 7 hours duration storm rainfall = 143.3mm or 14.33 cm.

Storm Areal rainfall :-

Areal to point rainfall ratio for 7 hours duration corresponding to C.A. of 1371.32 sq.km. is 77%
Areal rainfall = 14.33 x 0.77 = 11.03 cm .

Time distribution in put storms :-

The design storm rainfall duration (TD) is 7 hours,

Therefore mean average time distribution curve of storm rainfall for storm duration 7 to 12 hours,. In fig.9 of C.W.C. booklet subzone - 3h for the design storm rain fall d = 11.03 cm for the design storm duration TD = 7 hours calculated in table -4

TABLE - 4 TIME DISTRIBUTION OF IN PUT STORM

Sl. No.	Time in hours.	Percentage of storm duration Col. 2x100/TD	Cumulative % age to total rainfall	Cumulative rainfall depth D x col.4/100	Rainfall increments.
1	2	3	4	5	6
0	0	0	0	0	0
	1	14	45	4.96	4.96
	2	28.57	68	7.50	2.54
	3	42.86	78	8.60	1.10
	4	57.14	88	9.71	1.11
	5	71.43	92	10.15	0.44
	6	85.71	96	10.56	0.41
	7	100.00	100	11.03	0.47
					11.03

Time at 1 hour interval upto TD = 7 hours was entered in col.1
Cumulative percentage in storm duration corresponding to time in hours in Col.1 was calculated was entered in col.2.

The cumulative percentage of total rainfall corresponding to cumulative percentage of storm duration in col.2 was read from Fig.9 of C.W.C booklet for subzone - 3 h and entered in col.3. cumulative rainfall depth corresponding to cumulative % of total rainfall in Col.3 was calculated and entered in Col.4 Hourly rainfall increments were calculated by subtracting the two successive rainfall values from one another at 1 hour interval and entered in Col.5.

V) Design loss rate : The C.A. under study is more than 75 sq.km. The design loss rate of 0.5 cm/hr as suggested in CWC booklet was adopted.

VI) Rainfall Excess Hydrograph : A constant loss rate of 0.5 cm/hr was deducted from hourly rainfall increment is col.4 of Table - 5 to obtain rainfall excess units in Col.6 of Table - 6 were arranged in critical order by placing maximum rainfall excess unit against maximum discharge ordinate of synthetic unit hygrograph. The current maximum rainfall excess unit was placed against the next lower discharge of unit hydrograph.. Likewise the remaining rainfall excess unit were arranged in similar manner (col.3 and Col.7 of Table-5). The hourly rainfall excess sequence so obtained in Col.7 was reversed and shown in col.8 starting from 1 to 20 hours.

Computation of design flood U.G.H.

Refer Table-7 for the computation of design flood hydrograph. SUH ordinates are entered in col.2 against time in Col.1 critical rainfall excess sequences from Col.8 of Table-5 was entered from col.4 to 10 (table-7). Direct surface runoff ordinates were calculated corresponding to hourly rainfall excess units with a successive lag of 1 hour. Enter the horizontal total in col.8. Base flow at a rate of 0.05 cumecs/sq.km for 1371.32 sq.km works out to 68.57 cumecs col.9.

The base flow added to the total surface runoff ordinates in col.11 to get the design flood flow in col.10. The maximum value of col10 is 3940.75 cumecs which is 50 years peak discharge.

For calculation of 50 years peak discharge only, the rainfall excess units in Col.7 of Tab-6 are multiplied with corresponding unit hydrograph ordinates of Col.3 of tab-5 are shown in Tab-6. Thus col.4 of Tab-6 at the end shows the 50 years peak discharge for the basin as 3940.75 cumecs or 1,39,167.58 c/s or 1.4 lakhs.

Table - 5 Computation of Critical Sequences of Rainfall Excess

Sl.No.	Time in Hrs	Synthetic Unit Hydrograph Ordinates (Cumecs)	Gross Rainfall increm-ents (Cms)	Constant Loss rate (cm/hr)	Rain fall excess Units Col(4-5) (Cms)	Sequence rainfall excess Units (cms)	Reverse sequence of Col
1	2	3	4	5	6	7	8
1	0	0	0.00	0.0	0.00	0.00	0.00
2	1	40	4.96	0.5	4.46		0.60
3	2	86	2.54	0.5	2.04		2.04
4	3	140	1.10	0.5	0.60		4.46
5	4	200	1.11	0.5	0.61		0.61
6	5	320	0.44	0.5	0		
7	6	460	0.41	0.5	0	0.61	
8	7	521	0.47	0.5	0	4.46	
9	8	498				2.04	
10	9	420				0.60	
11	10	320					
12	11	260					
13	12	224					
14	13	186					
15	14	146					
16	15	120					
17	16	86					
18	17	64					
19	18	40					
20	19	20					
21	20	0					

Table - 6 Computation of Peak Flood

Sl.No	Time in Hours	U.G. Ordinates in Cumecs	Effective rainfall (cms)	Direct runoff cumecs
1	2	3	4	5
1	0	0		
2	1	40		
3	2	86		
4	3	140		
5	4	200		
6	5	320		
7	6	460	0.61	280.60
8	7	521	4.46	2323.66
9	8	498	2.04	1015.92
10	9	420	0.60	252.00
11	10	320		
12	11	260		**3872.18**
13	12	224	Add base flow	68.57
14	13	186		**3940.75**
15	14	146	or 1,39,167.58 c/s	
16	15	120	or 1.40 lakh c/s.	
17	16	86		
18	17	64		
19	18	40		
20	19	20		
21	20	0		

Table -7 Computation of Design Flood Hydrograph of Maddileru

Sl.No	Time in Hrs	One Hour Synthetic Hydrograph ordinate Cumecs	Rainfall excess Increments				Total Surface flow	Base flow	Design flood Hyderograph
			0.6	2.04	4.46	0.61			
1	2	3	4	5	6	7	8	9	10
1	0	0	0				0		
2	1	40	24	0			24	68.57	92.57
3	2	86	51.6	81.6	0		133.2	68.57	201.77
4	3	140	84	175.44	178.4	0	437.84	68.57	506.41
5	4	200	120	285.6	383.56	24.4	813.56	68.57	882.13
6	5	320	192	408	624.4	52.46	1276.86	68.57	1345.43
7	6	460	276	652.8	392	85.4	1906.2	68.57	1974.77
8	7	521	312.6	938.4	1427.2	122	2800.2	68.57	2868.77
9	8	498	298.8	1062.84	2051.6	195.2	3608.44	68.57	3677.01
10	9	420	252	1015.92	2333.66	280.6	3872.18	68.57	3940.75
11	10	320	192	856.8	2221.08	317.81	3587.69	68.57	3656.26
12	11	260	156	652.8	1873.2	303.78	2985.76	68.57	3054.35
13	12	224	134.4	530.4	1427.2	256.2	2348.2	68.57	2416.77
14	13	186	111.6	456.96	1159.6	195.2	1923.35	68.57	1991.93
15	14	146	87.6	370.44	999.04	158.6	1624.68	68.57	1693.25
16	15	120	72	297.84	829.56	136.64	1336.04	68.57	1404.61
17	16	86	51.6	244.8	651.16	113.46	1061.02	68.57	1129.56
18	17	64	38.4	175.44	535.2	89.06	838.1	68.57	906.67
19	18	40	24	130.56	383.56	73.2	611.32	68.57	679.89
20	19	20	12	81.6	285.44	52.46	431.5	68.57	500.07
21	20	0	0	40.8	178.44	39.04	258.24	68.57	326.81
22	21			0	89.2	24.4	113.6	68.57	182.17
23	22				0	12.2	18.8	68.57	80.77
24	23					0	0	68.57	68.57

Design flood = 3940.76 cumecs or 1.40 lakh cusecs

Triangular Unit Hydrograph Method

Maddileru Reservoir Project

Capacity of Reservoir : 1.1593 TMC
Since it is less than 2 TMC , Maximisation factor is not considered.
Clock hour correction: 13% is considered.

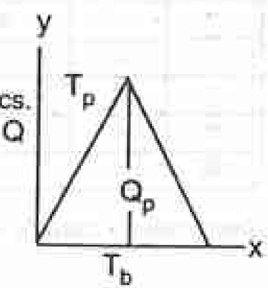
Maximum Rainfall = 6.05 inches at Bukkapatnam Rain gauges station for arriving at MFD. (Chitravati Balancing Reservoir) 277.18641
After clock hour correction M.F. storm = 6.05 x 1.13 = 6.837 inches
C.A. = 529.04 Sq. miles
Length of Vagu = 42.8 miles
H = 3620 - 1177.34 = 2442.66 ft.

$$Tc = \left(\frac{11.9 \times 42.8^3}{2442.66} \right)^{0.385} = 9.865$$

Lag Time i.e. from centre of rainfall excess to the time of peak, L = 0.6xTc
= 0.6x9.865
= 5.919 Hrs
= or 6 Hrs.

Units duration (or) rainfall excess period = D
Generally D = (1/3 or 1/4 Tc)
Considering D = 1/4 Tc = 1/4 x 9.865 = 2.466 or Say 2 hrs.
Now we have the following characteristics
A = 519.68 Sq. miles; Tc = 9.865; D = 2 Hrs; d = 1 unit
Tp = D/2+ 0.6xTc = 2/2 + 5.919 = 6.919 Hours or 7 Hours
Tb = 2.67Tp = 2.67x7 = 18.69 or 19 Hrs.

$$Qp = \frac{484xAxQ}{Tp} = \frac{484x519.68 \times 1}{7} = 36579 \text{ Cusecs.}$$



Qp = 36579 Cusecs
Tp = 7 Hrs.
Tb = 19 Hrs
D = 2 Hrs

Storm Distribution **Nellore SRRG**

Hours	Hourly percentage	2 Hrs	4 Hrs	
0	0	0	0	
1	10		10	
2	9.5	19.5		
3	8.00		17.5	
4	8.00	16.00		35.5
5	7.50		15.50	
6	7.00	14.50		
7	6.00		13.00	
8	4.50	10.50		25.00
9	4.00		8.50	
10	3.50	7.50		
11	3.00		6.50	
12	3.00	6.00		13.5
13	2.50		5.50	
14	2.50	5.00		
15	2.50		5.00	
16	2.50	5.00		10.00
17	2.50		5.00	
18	2.50	5.00		
19	2.00		4.50	
20	2.00	4.00		9.00
21	2.00		4.00	
22	2.00	4.00		
23	2.00		4.00	
24	1.00	3.00	1.00	7.00
25	0.00	100.00	100.00	

Note : Infiltration losses at 0.05" / Hour.

Statement showing the Cumulative Percentage vide Depth Duration Curve

Cumulative Reading %	2 Hours Discharge %	2hr Storm Rainfall increment 6.837 in	Rainfall excess after deducting 0.05in/ Hour towards infiltration loss
0	0	0	0
10	10	0.684	0.584
27.5	17.50	1.196	1.096
43.00	15.50	1.060	0.960
56.00	13.00	0.889	0.789
64.50	8.50	0.582	0.482
71.00	6.50	0.445	0.345
76.50	5.50	0.376	0.276
81.50	5.00	0.342	0.242
86.50	5.00	0.342	0.242
91.00	4.50	0.307	0.207
95.00	4.00	0.273	0.173
99.00	4.00	0.273	0.173
100.00	1.00	0.068	0.000
100.00	0	0	0.000
		6.837	

Compounding of Hydrograph

Hours	TUHG Ordinates in 1000 Cusecs	Increments peak to peak	Reverse Order
		0.207	0
0	0	0.242	0.173
1	5.226	0.345	0.173
3	15.677	0.789	0.242
5	26.130	0.960	0.276
7	28.450	1.096	0.482
9	36.579	0.584	0.584
11	24.386	0.482	1.096
13	18.290	0.276	0.960
15	12.190	0.242	0.789
17	6.100	0.173	0.345
19	0	0.173	0.242
		0	0.207

Flood Hydrograph Ordinates of Triangular Unit Hydrograph Compounding for the Catchment of Monneru - Storm Distribution

Rainfall increments in Reversal order													Total	Base Flow	Grand Total
0	0.173	0.173	0.242	0.276	0.482	0.584	1.096	0.96	0.789	0.345	0.242	0.207	1000 Cusecs	1000 cusecs @ 5 cusecs/sqm	Cusecs
0													0	2.598	2598
0	0												0	2.598	2598
0	0.904	0											0.904	2.598	3502
0	2.712	0.904	0										3.616	2.598	6214
0	4.52	2.712	1.265	0									8.497	2.598	11095
0	4.922	4.52	3.794	1.442	0								14.578	2.598	17276
0	6.328	4.922	6.323	4.327	2.519	0							24.419	2.598	27017
0	4.219	6.328	6.885	7.212	7.556	3.032	0						35.252	2.598	37850
0	3.164	4.219	8.852	7.852	12.595	9.155	5.728	0					51.565	2.598	54163
	2.109	3.164	5.901	10.096	13.713	15.26	17.18	5.017	0				72.442	2.598	75040
	1.055	2.109	4.420	8.731	17.831	16.61	28.63	15.06	4.123	0			96.378	2.598	98976
	0	1.055	2.95	5.048	11.754	21.36	31.18	25.08	17.36	1.803	0		112.60	2.598	115205
		0	1.476	3.364	8.816	14.24	40.09	27.31	20.61	5.409	1.265	0	122.5	2.598	125189
			0	1.584	5.876	10.66	26.72	35.11	22.44	9.015	3.794	1.082	116.42	2.598	119020
				0	2.94	7.119	20.04	23.41	28.86	9.815	6.323	3.245	101.76	2.598	104385
					0	3.562	13.36	17.55	19.24	12.62	6.885	5.409	78.635	2.598	81233
						0	6.686	11.70	14.43	8.413	8.852	5.889	56.973	2.598	58571
							0	5.856	9.618	6.31	5.901	7.572	35.257	2.598	37855
								0	4.813	4.206	4.426	5.048	18.493	2.598	21091
									0	2.105	2.95	3.786	8.841	2.598	11439
										0	1.476	2.523	3.999	2.598	6597
											0	1.263	1.263	2.598	3861
												0		2.598	2598

Maximum Flood Discharge = 1,25,189.00 Cusecs or Say 1,25,200 Cusecs

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Statement showing the Cumulative Percentage vide Depth Duration Curve

Cumulative Reading %	2 Hours Discharge %	2hr Storm Rainfall increment 6.837 in	Rainfall excess after deducting 0.05in/ Hour towards infiltration loss
0	0	0	0
10	10	0.684	0.584
27.5	17.50	1.196	1.096
43.00	15.50	1.060	0.960
56.00	13.00	0.889	0.789
64.50	8.50	0.582	0.482
71.00	6.50	0.445	0.345
76.50	5.50	0.376	0.276
81.50	5.00	0.342	0.242
86.50	5.00	0.342	0.242
91.00	4.50	0.307	0.207
95.00	4.00	0.273	0.173
99.00	4.00	0.273	0.173
100.00	1.00	0.068	0.000
100.00	0	0	0.000
		6.837	

Compounding of Hydrograph

Hours	TUHG Ordinates in 1000 Cusecs	Increments peak to peak	Reverse Order
0	0	0.207	0
1	5.226	0.242	0.173
3	15.677	0.345	0.173
5	26.130	0.789	0.242
7	28.450	0.960	0.276
9	36.579	1.096	0.482
11	24.386	0.584	0.584
13	18.290	0.482	1.096
15	12.190	0.276	0.960
17	6.100	0.242	0.789
19	0	0.173	0.345
		0.173	0.242
		0	0.207

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Flood Hydrograph Ordinates of Triangular Unit Hydrograph Compounding for the Catchment of Monneru - Storm Distribution

Rainfall increments in Reversal order													Total	Base Flow	Grand Total
0	0.173	0.173	0.242	0.276	0.482	0.584	1.096	0.96	0.789	0.345	0.242	0.207	1000 Cusecs	1000 cusecs @ 5 cusecs/sqm	Cusecs
0													0	2.598	2598
0	0												0	2.598	2598
0	0.904	0											0.904	2.598	3502
0	2.712	0.904	0										3.616	2.598	6214
0	4.52	2.712	1.265	0									8.497	2.598	11095
0	4.922	4.52	3.794	1.442	0								14.579	2.598	17276
0	6.328	4.922	6.323	4.327	2.519	0							24.419	2.598	27017
0	4.219	6.328	6.885	7.212	7.556	3.052	0						35.252	2.598	37850
0	3.164	4.219	8.852	7.852	12.595	9.155	5.728	0					51.565	2.598	54163
	2.109	3.164	5.901	10.096	13.713	15.26	17.18	5.017	0				72.442	2.598	75040
	1.055	2.109	4.426	0.731	17.631	16.61	28.63	15.05	4.123	0			96.378	2.598	98976
	0	1.055	2.95	3.048	11.794	21.36	31.18	25.08	12.36	1.803	0		112.60	2.598	115205
		0	1.476	3.364	8.816	14.24	40.09	27.31	20.61	5.409	1.265	0	122.5	2.598	125189
			0	1.684	5.876	10.68	26.72	35.11	22.44	8.015	3.794	1.082	116.42	2.598	119020
				0	2.94	7.119	20.04	23.41	28.86	9.815	6.323	3.245	101.78	2.598	104385
					0	3.562	13.36	17.55	19.24	12.62	6.885	5.409	78.635	2.598	81233
						0	8.686	11.70	14.43	8.413	8.852	5.889	55.973	2.598	58571
							0	5.856	9.618	6.31	5.901	7.572	35.257	2.598	37855
								0	4.813	4.206	4.426	5.048	18.493	2.598	21091
									0	2.105	2.95	3.786	8.841	2.598	11439
										0	1.476	2.523	3.999	2.598	6597
											0	1.263	1.263	2.598	3861
												0		2.598	2598

Maximum Flood Discharge = 1,25,189.00 Cusecs or Say 1,25,200 Cusecs

Solving the above equations,

a = 0.6591, and b = -12.8464

The equation is given by

Y = 0.6591X - 12.8464

Regression coefficient is given by

$$R_{xy} = \frac{\Sigma XY - n\,X\,Y}{(\Sigma X^2 - nX^2)(\Sigma Y^2 - nY^2)}$$

substituting the values

Rxy = 0.96 for the above example.

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Chapter V

Design of Channels

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SECTION 5.1.0

5.1.1 Flow through Open Channels

The term "open channel" applies to any passage through which water is flowing when the free surface of water is in contact with the atmosphere throughout.

i) Water flowing through an open channel is subjected to frictional resistance of wetted surface of the channel. In channels of regular cross section the velocity of flow is constant.

ii) The velocity of flow will vary at different points of the cross section of the channel, being smaller towards the sides. All calculations on the flow through channels are based on the mean velocity of flow at any cross section.

5.1.2 Formula for flow in Open Channel

i) The hydraulic gradient is equal to the slope of the channel if the latter is uniform. The head due to the slope of the channel is assumed to be lost in overcoming frictional resistance of bed and sides. Following equation for flow of water in channels has been proposed by Chezy in 1769.

$$V = C\,(RS)^{0.5}$$

where, V = mean velocity of flow in m/sec
C = a coefficient
R = Hydraulic mean depth (area over perimeter)
S = Slope of channel.

ii) value of coefficient 'C' depends upon the shape and surface of the channel. Statistical analysis has been made from time to time by various authors to determine the value of Chezy's 'C' out of which the values expressed by Ganguillet and Kutter and later Manning, Manning's "C " has become popular and can be safely adopted in design of channels.

According to Manning $C = R^{1/6} / n$

substituting value of C in Chezys equation

$$V = 1/n\; R^{2/3}\, S^{1/2} \qquad \text{Manning's formula.}$$

Where 'n' is roughness coefficient. The value of 'n' varies according to physical roughness of the sides and bottom of the channel and influenced by such factors as i) channel curvature ii) changes in size and shape of cross section iii) obstruction such as debris, roots iv) vegetation etc.

iii) In design of channels the value of 'n' to be adopted also depends upon the channel condition expected in future. The channel condition is governed by the weeds, silt or scouring problems and standards of maintenance.

vi) Values of 'n' observed for a few channels in Uttar Pradesh are found to be between 0.025 to 0.03. It is desirable to adopt 'n' equal to 0.025 in design of irrigation channels in general and a higher value (0,0275) for smaller channels. Generally n equal to 0.018 is being adopted for channels lined with Concrete.

v) The values of coefficient of rugosity recommended for use in Manning formula in varying situations is presented in table B-4

5.1. 3 Trapezoidal Section :
Practical considerations necessitate making a channel of Trapezoidal section. The corners are rounded off in lined canals.

5.1 4 Economical Trapezoidal Section :
i) The most economical section for a trapezoidal channel will be when the discharge is maximum for a given excavation. The condition for this can be found by assuming the area to be constant.
Bed width (b) to depth (d) ratio corresponding to inner side slopes of section are as follows :

where inner side slopes are 0.5 H to 1 V ;	b/d	= 1.2
where inner side slopes are 1 H to 1 V	b/d	= 0.8
where inner side slopes are 1.5 H to 1 V	b/d	= 0.6

5.1.5 Cross Section of least absorption.
i) In unlined channels absorption losses are significant. The absorption per unit of time in any channel of uniform cross sectional area becomes smaller as b/d increases. For channels of equal cross sectional areas (with 0.5 H to 1 V side slopes) the cross section giving least absorption has a proportion of bed width to depth of four.

5.1.6 Proportioning of bed width to depth :

General problem is to design a channel which will carry the fixed discharge and with smallest absorption losses commensurate with economy of construction. Evaporation losses are small and may be neglected. As for absorption is concerned any value of b/d between 1 and 5 may be adopted.

ii) When aligning a channel in very flat country it may be necessary to make it wider and shallower than would, otherwise be economical (or desirable).

iii) Where reservoirs are the source of supply there is no problem of silting; Channels are designed for velocities subject to non-scouring limits. In such situations it is desirable to make channels deep in proportion to their width.

iv) bed width to depth ratios cannot be rigidly fixed because these will depend upon various factors namely;
a) the cost of excavation (rocky strata etc)
b) the cost of acquisition of lands.
c) whether the canal will be lined
d) seepage characteristics of the canal prism

B/d ratios recommended by C.W.P.C are presented in Table B-3 and U.S.B.R. practice is in Table - C 3 section .

Where other considerations permit, channels may be designed with a proportion of bed width to depth between 1 to 5.

5.1. 7 Side slopes :
i) The side slopes depend on the local soil characteristics

a) Inner side slopes.
The inner side slopes are to be designed to with stand, the sudden draw down condition during operation of canal. In case of alluvial canals for purpose of design, 0.5 H to 1.V inner side slopes are adopted assuming that the canal attains a slope of 0.5 H to 1, after running in regime. Inner, side slopes proposed generally for canals in cutting are 1 to 1 to, 1.5 to 1.

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b) bank side slopes.

The bank side slopes are to be designed for the condition, that, the canal is running full, and banks are saturated due to rain fall. Canals in filling are generally proposed with inner side slopes 1.5 H to 1 and with outer side slopes ranging 1.5 H to 1; 2 H to 1.

5.1.8 Free Board :

Sufficient free board is to be provided taking into consideration canal size and location, rain water inflow, water surface fluctuations caused by regulators, wind action soil characteristics service road requirement etc. A minimum free board of 0.5 m for discharges of less than 10 cumecs and 0.75 m for discharges greater than 10 cumecs is recommended. The free board is to be measured from the full supply level to top of the bank.

5.1.9 Bank Top Width :

The minimum values recommended for top width of banks are presented in table A.1. For canals carrying less than 1.5 cumecs it is generally not economical to construct a service road on top of bank as this requires more materials than the excavation provides. In such cases service road may be provided on natural ground surface adjacent to the bank.

Table A.1. Top width of banks.

Discharge range in Cumecs	Minimum bank top width in metres: Inspection bank	Non inspection bank
0.15 - 7.5	5.0	1.5
7.5 - 10.0	5.0	2.5
10.0 - 15.0	6.0	2.5
15.0 - 30.0	7.0	3.5
30.0 and above	8.0	5.0

5.1.10 berms

Berms along earthern channels are provided to reduce the bank loads which may cause sloughing off earth in to the canal section.

Berms may help, to limit the height of rise of water level at a time of fluctuations and to protect the bank from the direct attack of canal water. However berms provided below full supply level make it difficult to maintain desired water levels.

Berms are omitted:

i) In small distributary channels.

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ii) Channels running wholly are almost wholly, in banking

iii) In partial cutting reaches where cutting slope confirms with inner slope of banks.

Wherever berms are proposed the width of berms is arrived at adopting the following formula.

Width of berm = $C + (r_2 - r_1)$ d

Where C is constant, D the actual depth of cutting at each place and r_2 and r_1 the horizontal components of the slopes of bank and cutting respectively in one metre vertical.

Where the inside slope of cutting and bank are 1 H to 1 V and 1.5 H to 1 V respectively berm width is equal to :

$C + (1.5 - 1)\ D = C + 0.5\ D$

Depending upon the carrying capacity of channel and local situations, C values ranging from 0.6 to 2 m are being adopted in partial cutting reaches if the inner side slope of bank and side slope adopted for cutting are different invariably berms are to be provided so as to keep the central line of bank and centre line of channel parallel. (Even when value of C is nill)

In some projects same side slopes are adopted in cutting portion and to inner side slopes of bank, thus confirming the entire canal section to a uniform side slope and berms are avoided.

5.1.11 Full cutting reaches (more than FSD + FB) and position of berms.

Where depth of cutting is less than 9. m the berms (inspection path)my be provided at ground level. Where depth of cutting is more than 9 metres the berms (inspection path) to be provided at F. S .D + 1to 1.5 metre depth of cutting.

BERNS ON REAR SLOPE OF BANKS:

Berms have to be provided on the rearside of bank to provide sufficient cover over hydraulic gradient line where ever found necessary

5.1.12 HYDRAULIC GRADIENT LINE:

The gradient depends mainly on the characteristics and relative placement of the different type of material in the embankment . Following empirical values may be used for banks of less than 5 m high.

For silty soils 4:1
For silty sands 5:1
For silty soils 6:1

For embankments of more than 5 m high the true position of Hydraulic gradient line is to be worked out by laboratory tests.

5.1.13 CLASSIFICATION OF CHANNELS:

For purpose of design, irrigation canals can be broadly classified in to following three categories.

1. Unlined canals in alluvial soil. (Regime flow).
2. Unlined canals in non-erodible strata . (Permissible velocity)
3. Lined canals.

Silting problem arise in channels taking off from diversion works and also in channels in alluvial soil. These channels are to be designed on the principle of minium permissible or the non-silting velocity. Design procedure presented in section B.

Channels taking off from storage works usually carry silt free water. These are to be designed on principle of maximum permissible velocity , the velocity that will not cause erosion of the channel body . Design procedure is presented in section C.

Lined canals are designed for a velocity of about 2 metres per second . Transmission losses are assumed to be about $0.6m^3$ per million square metre of wet terrain lined canals.

5.1.14 IRRIGATION CANALS:

An irrigation channel is to be designed so as to supply water to the crops effectively and economically.

Channel capacity at any point is to be fixed so as to meet the water requirements of crops as well as the transmission losses below the point , taking in to account the proposed mode of delivery.

5.1.15 WATER REQUIREMENTS:

water requirements of crops are not uniform, they vary widely in accordance with variations in climate, in particular rain fall and also soil characteristics . Further crops at their various stages of growth require different quantities of water and the requirements are maximum during growth and flowering periods . Water requirements of crops include evapotranspiration requirements , application losses , and any other special needs.

5.1.16 DUTY

The relation between the quantity of water , the area of crop irrigated is denoted by duty of water and it is expressed in any of the following units.

i)Depth of water over cropped area (depth in centi metres or metres)

ii)Volume of water per unit of cropped area (Cubic metres per hectare area)

iii) Cropped area for a unit discharge during crop period. (A in hectares per cumec)

It is necessary to specify the point at which duty is expressed and also the crop period. Duty expressed at a field out let shall take in to account the net water requirement of crops and field application losses . Duty expressed at any other point (distributary head, Major head etc) shall take in to account , the water requirements of crops, the field application losses and transmission losses below the point up to field outlet.

5.1.17 DESIGN CAPACITY OF CHANNEL:

Information on following forms the basis for arriving at the design capacity of irrigation channel.

Estimated 10 day peak water requirements of crops during a crop season. Alternatively guidance in scheduling of irrigation (depth and frequency) for different crops from the agricultural research farms on similar soil and agroclimatic conditions.

Mode of delivery of water, that is continuous , rotational or delivery on demand.

Estimated seepage losses in the canal system or observed data on similar soil conditions.

Until sufficient reliable observed data on water requirements of crops and seepage losses is available, the design discharge is to be arrived at on the basis of past experience.

5.1.18 Channel capacity on average duty approach

In some of the earlier projects average duties have been adopted for arriving at the canal capacity based on the past experience in similar soil and climatological conditions . In the upper reaches of command area of N.S. Left canals following average duties have been adopted.

i) Wet crops (paddy) 914 hectares per cumec (64 acres per cusec)

ii) Irrigated Dry 2071 hectares per cumec (145 acres per cusec)

The above duties include transmission losses in distributaries below 15 cumecs carrying capacity. Allowances towards conveyance losses was made at 1.83 cumecs per million square metres of wetted area in canals of more than 15 cumecs carrying capacity only (transmission losses at 6 cusecs per million square feet of wetted area).

Peak requirements were proposed to be met by encroaching in to free board of canal temporarily and by staggering the crops.

While in the case of paddy the peak demand for 10 days period and an average demand for the season may not differ materially , there will be large variation in the case of I.D. Crops between the average seasonal demand and 10 days peak demand particularly in the Kharif season.Transmission losses in the majors, minors and subminor distributaries are observed to be about 20 per cent or more depending upon the length of canals.

In the lower reaches of N.S. left canal ayacut sliding scale duties have been adopted considering the following

1) Canals commanding bigger areas of land involving larger lengths of distributaries, system to be based on lower duty

ii) canals commanding smaller area required smaller lengths of distributaries to be based on higher duties

Sliding scale of duties in lower reaches of N.S. left canal (Continuous flow)

Area under offtakes in Hectares	**Duty for I.D. or I.D. cum wet Crops Hectar per cumec**	**Duty for wet areas Hectares Per cumec**
Below 40 hectares	1075 (75 acres/cusec)	850 (60 acres/cusec)
40 to 4000 hectares	1000 (70 acres/cusec)	800 (56 acres/cusec)
4000 to 12000 hectares	925 (65 acres/cusec)	760 (53 acres/cusec)
Above 12000 hectares	850 (60 acres/cusec)	715 (50 acres/cusec)

5.1.19 Channel Capacity on Peak Duty approach :

In Srirama Sagar Project following peak duties were adopted for arriving at the capacity of a channel. (continuous flow)

i) Paddy 571 Hectares per cumec
(40 acres per cusec)

ii) Irrigated Dry 1428 Hectares per cumec
(100 acres per cusec)

iii) Paddy and Irrigated Dry.

Where a canal serves both wet and I.D. areas maximum discharge requirements during overlap period are estimated with duties 714 Hect/cumec for paddy and 1643 Hectare per cumec for I.D.

5.1.20 Arriving at the capacity of channel

The proposed system layout and position of offtakes, the area proposed for irrigation under each offtake in each distributary is plotted on irrigation map. Discharge required in each length of distributary to keep the system fully supplied at the time of greatest demand is worked out commencing from tail end. Where regulation is effected by turns the carrying capacity of channel is suitably increased to provide for non-supply during periods of closure. Canal capacity to be as per approved design operation plan.

5.1.21 Classification of canals based on generation

Present classification of canals is on the basis of their generation

i) Canals taking off from the point of diversion from left and right flanks of river are called as left main canal and right main canal respectively.

ii) Canals taking off from main canal having a capacity of 15 cumecs or more (500 cuseus in older system) and upto the point of 15 cumecs limit are called as branch canals.

iii) Canals taking off from main or branch canal with a head discharge of more than 0.04 cumecs are termed as major distributaries

iv) Canals taking off from major distributaries and serving more than 40 hectares are termed as minor distributaries.

v) Canals serving less than 40 hectares ayacut are called field channels and denoted by numbering left or right side pipes.

Branch canals, majors and minors are named after a prominent place (or village) near about their tail end.

For clarity and convenience, for standardization and facility of communication, classification of canals on the basis of their capacity is desirable. Classification recommended by Commission for Irrigation Utilization on the basis of suggestion made by C.W.C. are as follows.

i) main canal, Contour canal taking off from the point of diversion.

ii) Branch canal. A distributary canal with a capacity of more than 15 cumecs and generally though not necessarily aligned along a ridge.

iii) Major distributary. Generally a ridge canal taking off from main canal or branch canal with carrying capacity of more than 3 cumecs and less than 15 cumecs.

iv) minor distributary, generally a ridge canal, with a carrying capacity of more than I cumecs and less than 3 cumecs.

v) Sub-distributary. Generally a ridge canal with a carrying capacity not exceeding one cumec.

vi) Field channel. Canal serving less than 40 hectares of irrigated area.

SECTION - 5 . 2 .0

Design of Channels on principles of Regime flow

5.2.1. A regime channel may be defined as a channel which will get neither silted up nor scoured, but will maintain its shape and section for a specified discharge.

In practice, the concept applies only in a limited sense.

In case where a canal is proposed to take off from a diversion work, the nature of transported silt carried from the parent river defines the channel shape.

In case where canal is proposed to take off from a reservoir (with clear water), the soil through which the channel traverses will define its shape. (Provided it is incoherent alluvial soil). If the soil is clayey, the channel will retain the original shape to which it is excavated.

It is desirable to fix the shape of channel to suit the nature of boundary materials so that the channel is stable. However, to avoid certain drawbacks, the channel section is proportioned by simple rules and the longitudinal slope of channel is adjusted accordingly. (Draw backs. a) low hydraulic efficiency due to wider channel and less depth,
b) excessive seepage and evaporation losses).

5.2 2. K.G. Kennedy deduced from his observations that for non-slitting and non-scouring channels in steady regime, there is always one velocity which is called the 'Critical velocity', Vo . The general form of kennedy's equation of critical velocity is

$$V_o = C d^m$$

Where Vo = Critical velocity in m/sec (at which, the channel will neither silt nor scour)
C and m vary according to slit grade. Recommended values of C and m for Krishna delta, Godavari delta and Punjab silts are as under

	C	m
Krishna delta	0.53	0.52
Godavari delta	0.39	0.55
Punjab silt	0.55	0.64

In table B-1 values of critical velocities corresponding to depth of flow in a channel are presented.

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Table B-1

Critical velocity $V_o = Cd^m$

Vo in meters per second

Depth of flow d in metres	Krishna delta	Godavary delta	Panjub Silt
0.2	0.23	0.16	0.20
0.3	0.28	0.20	0.25
0.4	0.33	0.24	0.31
0.5	0.37	0.27	0.35
0.6	0.41	0.29	0.40
0.7	0.44	0.32	0.44
0.8	0.47	0.35	0.48
0.9	0.50	0.37	0.51
1.0	0.53	0.39	0.55
1.1	0.56	0.41	0.59
1.2	0.58	0.43	0.62
1.3	0.61	0.45	0.65
1.4	0.63	0.47	0.68
1.5	0.65	0.49	0.71
2.0	0.76	0.57	0.86
2.5	0.85	0.64	0.99
3.0	0.94	0.71	
4.0		0.84	
5.0		0.95	

5.2.3 Limits of the allowable velocity

The channel sections are to be designed for velocities not exceeding 1.1Vo and not less than 0.9 Vo

In channels where navigation is proposed the velocity should be about 0.8 meters and less.

Maximum permissible velocities

In B.C. soils 0.5m/Second

In hard soils 1.0 m/Second.

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5.2.4 Depth of flow

Depth of flow 'd' in a channel is a function of discharge (Q) and the formula suggested to arrive at the 1st approximation of 'd' is

$$d = kQ^{1/3}$$

where d = depth of flow in meters

K = 0.7 (Constant)

Q = discharge in cumecs.

The depth arrived at by adopting above formula is for guidance and values can be varied according to individual situations.

In table B-2 depths of flow corresponding to discharge in the channel are presented for first approximation.

Table B-2

Depth of flow corresponding to discharge :

Q Litres/second	d in Cetimeters	Q in Cumecs	d in meters
20	20	4	1.1
80	30	5	1.2
200	40	6	1.3
400	50	8	1.4
600	60	10	1.5
1000	70	23	2.0
1500	80	80	3.0
2100	90	190	4.0
3000	100	400	5.0

5.2.5 Bedwidth to depth ratios

Cross sections which are stable and in regime appear to have certain relationship between bed width to depth of flow.

To avoid the necessity of making assumptions, best fitted equations of regime channels based on data of stable channels for every region are important from practical considerations. Bed width to depth ratios generally adopted are presented in table B-3 for design of channels

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until best fitted equations are developed for each region. The values may vary depending upon individual needs.

Table B-3

Q in Litres/Second	b/d	Q in Cumecs	b/d
300	2.9	2	4.0
400	3.0	4	4.9
600	3.2	10	5.4
900	3.4	15	6.0
1000	3.5	30	7.4

5.2.6 Inner side slopes for design.
Slit transporting channels have tendency to assume semi elliptical sections. The finer the silt, more does the section approximate to semicircle. Therefore for purpose of design the practice is to assume 0.5H to 1V side slopes.

5.2.7 Side slopes for construction
Side slopes to be adopted for construction depend upon the soil characteristics. The side slopes are to be designed to with stand the following conditions.
i) Sudden draw down condition for inner slopes
ii) Channel running full with banks saturated due to rainfall.

5.2.8. Channels are usually excavated in 1H to !V side slopes, it being assumed that after silting they will have side slopes, of approximately 0.5 H to 1V. However in sandy soils the side slopes in cutting may be 1.5H to 1V or flatter.

Canals in filling in general are proposed with 1.5 H to 1V but in loose sand they shall be 2H to 1V are flatter.

5.2.9 Rugosity Coefficient n.
Foe normal alluvial soils, it is usual in India to assume following values of n depending upon the carrying capacity of canals.
For canals of carrying capacity more than 15 Cumecs n 0.02
For canals of carrying capacity less than 15 Cumecs n 0.0225
In some projects values of n adopted are as follows

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Canals of 1.5 to 100 Cumecs, n 0.0225
Canals carrying less than 1.5 Cumec, n 0.025
Very small channels (below 0.3 cumecs capacity) n 0.0275

Basically the value of n depends upon the physical roughness of the canal and standards of maintenance during its operation. (Table B4)
Assuming average standards of maintenance it is better that canals are designed with n = 0.025.

5.2.10 Bed fall
The average slope of the ground is to be determined from the longitudinal section of the ground. This would be the maximum slope which can be provided on the canal. Flatter slopes have to be adopted if the velocity generated is higher than permissible one due to adoption of average slope of the ground.

5.2.11 **Flow formula.** Manning's formula which is derived from Chezy's fundamental equation is being adopted in design of open channels.

$$V = (1/n).\, R^{2/3} .S^{1/2}$$

where V = Mean velocity of flow in meters per second
n = Rugosity coefficient
R = Hydraulic mean depth (Area over perimeter, A/P)
S = Bed fall, 1 in S
Discharge Q = A X V in Cumecs.

5.2.12 Berms (Water side)
Berms are provided along the earthen channels to reduce the bank loads.
a) Berms are provided at full supply level
i) when the canal is in full cutting, that is ground level is above F.S.L.
ii) When the canal is in full embankment that is the bed level and F.S.L. are above the ground level.
b) Berms are provided at ground level when the F.S.L. of canal is above ground level and bed level below ground level.

In some projects berms are proposed at bed level of canal when the ground level is at or below the bed level.

Berms provided below full supply level of canal interfere with the regime of flow and make the maintenance of designed water levels difficult without resorting to some other temporary measures. This will happen untill natural silt berms, are formed.

5.2.13 Width of berms (Water Side)
general practice is to provide minimum width of (r_2-r_1) D to ensure the central line of canal and its banks are parallel.
r_2 and r_1 are horizontal components of the slopes of the bank and cutting respectively and D the depth of cutting at the point of reference.
However berms widths equal to
C + (r_2-r_1) D are usually proposed.
C is a constant (and in different projects) and depending upon the carrying capacity, values have been assigned to C.

5.2. 14 No Berms in small distributaries (Water side)
Berms may be omitted in small channels running wholly or almost wholly in banking unless the irrigation water is so heavily charged with silt so s to give prospect of rapid formation of silt berms.

In small channels with out berms (C = nil) it is desirable to make r_2 and r_1 equal.

5.2.15. Design Example :
design of channel of one cumec carrying capacity proposed through **alluvial** soil (or canal carrying silt laden water).

Refer tables as indicated - If necessary adopt interpolation

1st Step	Depth of flow from table B-2. 1 at approximation corresponding to Q, d = 70 cm.		
2 nd step	permissible velocity from table B-1 corresponding d.		
	d = 70 cm.	V (Krishna)	= 44 cm/sec.
		V (Godavari)	= 32 cm/sec.
	d = 80 cm.	V (Krishna)	= 47 cm/sec.
		V (Godavari)	= 35 cm/sec.
3 rd Step	Bed width from table B-3		

	Ratio corresponding to Q b = ratio X d say b = 280 cms.
4 th Step	Inner side slopes for purpose of design, (para B6) 0.5 H to 1 V.
5 th Step	Value of rugosity coefficient, n, (para B9) n = 0.025
6 th Step	Sections suitable, picked up from tables, B4 to B10

i)	d = 72 cm. v = 50.1 cm/s	b = 240 cm. Q = 995 L/Sec	b/d = 3.3	S = 1 in 2500
ii)	d = 74 cm. v = 50.8 cm/s	b = 240 cm. Q = 1041 L/Sec.	b/d = 3.2	S = 1 in 2500
iii)	d = 75 cm. v = 47.0 cm/s	b = 250 cm. Q = 1012 L/sec	b/d = 3.3	S = 1 in 3000
iv)	d = 80 cm v = 38.8 cm.	b = 300 cm Q = 1055 L/sec	b/d = 3.8	S = 1 in 5000
v)	d = 82 cm. v = 38.8 cm/s	b = 280 cm. Q = 1021 L/sec	b/d = 3.4	S = 1 in 5000
vi)	d = 84 cm. v = 39.3cm/s	b = 280 cm. Q = 1063 L/sec	b/d = 3.3	S = 1 in 5000

Sections with lesser depth may be adopted where sufficient driving head is not available in the parent channel.

DISTRIBUTARIES - STANDARDS - N.S. LEFT CANALS LATER REACHES

1. Side Slopes in cutting

a) Full cutting

i) In all softer soils 1.5 H to 1 V

ii) In harder soils below F & F when depth of cut in harder strata is F.S.D. OR Two times the difference in bed width of sections with 1:1 and 1.5 : 1 side slopes which ever is higher, 1 H to 1 V.

iii) In F&F and H.R. when depth of cut in harder strata is F.S.D. or difference in bed width of sections with 0.5 H to 1 and 1.5 H to 1 side slopes which ever is higher, 0.5 H to 1 V.

iv) Deep cut in H.R. Catch drains of size 0.3m X 0.3m with 1:1 side slopes at G.L. at the toe of spoil bank for discharges above 3 Cumecs.

b) Partial cutting in all soils bank 1.5H to 1V.

Any change in slope should be effected in full bay lengths.

2. Slopes in Banking

Inner slopes 1.5 H to 1 V.

Rear slopes up to 5 metre height 1.5 H to 1 V

Beyond 5 metre height depending upon nature of soil.

3. Top width of banks — **Free Boards**

Q range	LB - RB
15 to 5	3.0 - 1.8m
5 to 0.3	3.0 - 1.2m
Below 0.3	1.0 - 1.0m

Q Range	F.B.
15 to 4	1.0m
4 to 0.3	0.6m
Below 0.3	0.45m

4. Height of point bank, Limited to 3 meters.

5. Borrow pit Depth 1.0m, Side slopes 0.5 to 1, Distance between rear toe of bank and inner edge of borrow pit to be height of bank plus 1.5m.

Table B4

Table-4 . Values of Rugosity Coefficient (n) for

	Type of Canal	Value of n in normal Condition
A.	**Unlined Canals**	
1.	Earth, straight and uniform	
	a) Clean after weathering	0.022
	b) Gravel, uniform section, clean	0.025
	c) With short grass, few weeds	0.027
2.	Earth winding and sluggish	
	a) No vegetation	0.025
	b) Grass, some weeds	0.030
	c) Dense weeds or aquatic plants in deep channels	0.035
	d) Stony bottom and weedy banks	0.035
3.	Dragline excavated and no vegetation	0.028
4.	Channels not maintained, weeds and bush uncut dense weeds, high as flow depth	0.080
B.	**Lined Canals**	
5.	Concrete surface	
	a) Formed, no finish	0.013 to 0.017
	b) Float finish	0.013 to 0.015
	c) Gunite wavy section	0.018 to 0.022
6.	Concrete bottom float finished and sides as indicated below	
	a) Dressed stone in Mortor	0.015 to 0.017
	b) Random stone in mortor	0.017 to 0.020
	c) Cement Rubble masonry plastered	0.016 to 0.020
	d) Dry rubble (Rip-rap)	0.020 to 0.030
7.	Brick	0.014 to 0.017
8.	Asphalt smooth to rough	0.013 to 0.016
9.	Concrete lined excavated rock with	
	a) Good section	0.017 to 0.020
	b) Irregular Section	0.022 to 0.027

A small increase in n value may be made in channels of alignment other than straight.

TABLE - B5
Trapezoidal Canal Section

Side Slope = 0.5 H to 1V; n = 0.025
Bed Slope : 1 in S; V = cm/second
Q = litres / sec

d cm	b cm	b/d	A sqm	P m	R	$R^{2/3}$	S = 1 in 1000 V	Q	S = 1 in 2000 V	Q	S = 1 in 3000 V	Q
20	20	1.0	0.06	0.65	0.09	0.20	25.9	15.5	18.3	11.0	15.0	9.0
20	30	1.5	0.08	0.75	0.11	0.23	28.5	22.8	20.2	16.1	16.5	13.2
20	40	2.0	0.10	0.85	0.12	0.24	30.4	30.4	21.5	21.5	17.6	17.6
20	50	2.5	0.12	0.95	0.13	0.25	31.9	38.3	22.6	27.1	18.4	22.1
20	60	3.0	0.14	1.05	0.13	0.26	33.1	46.3	23.4	32.7	19.1	26.7
20	70	3.5	0.16	1.15	0.14	0.27	34.0	54.4	24.1	38.5	19.6	31.4
22	50	2.3	0.13	0.99	0.14	0.26	33.3	44.7	23.6	31.6	19.2	25.8
22	60	2.7	0.16	1.09	0.14	0.27	34.6	54.0	24.5	38.2	20.0	31.2
24	50	2.1	0.15	1.04	0.14	0.27	34.7	51.6	24.5	36.5	20.0	29.8
24	60	2.5	0.17	1.14	0.15	0.28	36.0	62.3	25.5	44.0	20.8	35.9
25	25	1.0	0.09	0.81	0.12	0.24	30.1	28.2	21.3	19.9	17.4	16.3
25	30	1.2	0.11	0.86	0.12	0.25	31.4	33.4	22.2	23.6	18.1	19.3
25	40	1.6	0.13	0.96	0.14	0.27	33.6	44.1	23.8	31.2	19.4	25.5
25	50	2.0	0.16	1.06	0.15	0.28	35.3	55.2	25.0	39.0	20.4	31.9
25	60	2.4	0.18	1.16	0.16	0.29	36.7	66.5	26.0	47.1	21.2	38.4
25	70	2.8	0.21	1.26	0.16	0.30	37.9	78.1	26.8	55.2	21.9	45.1
26	60	2.3	0.19	1.18	0.16	0.30	37.4	71.0	26.4	50.2	21.6	41.0
26	70	2.7	0.22	1.28	0.17	0.30	38.6	83.2	27.3	58.9	22.3	48.1
28	60	2.1	0.21	1.23	0.17	0.31	38.7	80.1	27.3	56.6	22.3	46.3
28	70	2.5	0.24	1.33	0.18	0.32	39.9	93.9	28.2	66.4	23.1	54.2
30	30	1.0	0.14	0.97	0.14	0.27	34.0	45.8	24.0	32.4	19.6	26.5
30	40	1.3	0.17	1.07	0.15	0.29	36.4	60.0	25.7	42.4	21.0	34.6
30	50	1.7	0.20	1.17	0.17	0.30	38.3	74.7	27.1	52.8	22.1	43.1
30	60	2.0	0.23	1.27	0.18	0.32	39.9	89.7	28.2	63.5	23.0	51.8
30	70	2.3	0.26	1.37	0.19	0.33	41.2	105.1	29.1	74.3	23.8	60.7
30	80	2.7	0.29	1.47	0.19	0.33	42.4	120.7	30.0	85.4	24.5	69.7
32	70	2.2	0.28	1.42	0.19	0.34	42.4	116.8	30.0	82.6	24.5	67.4

32	80	2.5	0.31	1.52	0.20	0.35	43.6	134.1	30.9	94.8	25.2	77.4
34	70	2.1	0.30	1.46	0.20	0.34	43.6	129.1	30.9	91.3	25.2	74.5
34	80	2.4	0.33	1.56	0.21	0.35	44.9	148.0	31.7	104.7	25.9	85.5
35	50	1.4	0.24	1.28	0.18	0.32	40.9	96.7	29.0	68.4	23.6	55.9
35	60	1.7	0.27	1.38	0.20	0.34	42.7	115.8	30.2	81.9	24.7	66.9
35	70	2.0	0.31	1.48	0.21	0.35	44.2	135.4	31.3	95.7	25.5	78.2
35	80	2.3	0.34	1.58	0.22	0.36	45.5	155.2	32.2	109.8	26.3	89.6
35	90	2.6	0.38	1.68	0.22	0.37	46.6	175.3	33.0	124.0	26.9	101.2
35	100	2.9	0.41	1.78	0.23	0.38	47.6	195.7	33.6	138.4	27.5	113.0
36	90	2.5	0.39	1.70	0.23	0.37	47.2	183.6	33.4	129.8	27.3	106.0
36	100	2.8	0.42	1.80	0.24	0.38	48.2	204.8	34.1	144.8	27.8	118.3
38	90	2.4	0.41	1.75	0.24	0.38	48.4	200.5	34.2	141.8	27.9	115.8
38	100	2.6	0.45	1.85	0.24	0.39	49.5	223.6	35.0	158.1	28.6	129.1

TABLE - B6
Trapezoidal Canal Section

Side Slope = 0.5 H to 1V; n = 0.025
Bed Slope : 1 in S; V = cm/second
Q = litres / sec

d cm	b cm	b/d	A sqm	P m	R	$R^{(2/3)}$	S = 1 in 2000 V	Q	S = 1 in 3000 V	Q	S = 1 in 4000 V	Q
40	70	1.8	0.36	1.59	0.23	0.37	33.2	119.4	27.1	97.5	23.5	84.4
40	80	2.0	0.40	1.69	0.24	0.38	34.2	136.7	27.9	111.6	24.2	96.6
40	90	2.3	0.44	1.79	0.25	0.39	35.0	154.2	28.6	125.9	24.8	109.0
40	100	2.5	0.48	1.89	0.25	0.40	35.8	171.9	29.2	140.4	25.3	121.6
40	110	2.8	0.52	1.99	0.26	0.41	36.5	189.8	29.8	155.0	25.8	134.2
40	120	3.0	0.56	2.09	0.27	0.42	37.1	207.9	30.3	169.7	26.2	147.0
42	110	2.6	0.55	2.04	0.27	0.42	37.3	205.5	30.5	167.8	26.4	145.3
42	120	2.9	0.59	2.14	0.28	0.42	38.0	225.0	31.0	183.7	26.9	159.1
44	110	2.5	0.58	2.08	0.28	0.43	38.2	221.7	31.2	181.0	27.0	156.7
44	120	2.7	0.62	2.18	0.29	0.43	38.8	242.6	31.7	198.1	27.5	171.6
45	90	2.0	0.51	1.91	0.27	0.41	37.0	187.1	30.2	152.8	26.1	132.3
45	100	2.2	0.55	2.01	0.27	0.42	37.8	208.4	30.9	170.1	26.7	147.4
45	110	2.4	0.60	2.11	0.28	0.43	38.6	229.9	31.5	187.7	27.3	162.6

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45	120	2.7	0.64	2.21	0.29	0.44	39.2	251.7	32.0	205.5	27.8	178.0
45	130	2.9	0.69	2.31	0.30	0.45	39.9	273.6	32.6	223.4	28.2	193.4
45	140	3.1	0.73	2.41	0.30	0.45	40.4	295.6	33.0	241.4	28.6	209.0
46	120	2.6	0.66	2.23	0.30	0.44	39.7	260.8	32.4	213.0	28.0	184.4
46	130	2.8	0.70	2.33	0.30	0.45	40.3	283.5	32.9	231.5	28.5	200.5
48	120	2.5	0.69	2.27	0.30	0.45	40.4	279.5	33.0	228.2	28.6	197.7
48	130	2.7	0.74	2.37	0.31	0.46	41.1	303.8	33.6	248.0	29.1	214.8
50	110	2.2	0.68	2.22	0.30	0.45	40.5	273.2	33.0	223.0	28.6	193.1
50	120	2.4	0.73	2.32	0.31	0.46	41.2	298.8	33.6	244.0	29.1	211.3
50	130	2.6	0.78	2.42	0.32	0.47	41.9	324.6	34.2	265.1	29.6	229.6
50	140	2.8	0.83	2.52	0.33	0.48	42.5	350.7	34.7	286.3	30.1	248.0
50	150	3.0	0.88	2.62	0.33	0.48	43.1	376.9	35.2	307.7	30.5	266.5
50	160	3.2	0.93	2.72	0.34	0.49	43.6	403.3	35.6	329.3	30.8	285.2
52	130	2.5	0.81	2.46	0.33	0.48	42.7	346.1	34.8	282.6	30.2	244.7
52	140	2.7	0.86	2.56	0.34	0.48	43.3	373.8	35.4	305.2	30.6	264.3
54	130	2.4	0.85	2.51	0.34	0.49	43.4	368.0	35.4	300.5	30.7	260.2
54	140	2.6	0.90	2.61	0.35	0.49	44.1	397.4	36.0	324.5	31.2	281.0
55	120	2.2	0.81	2.43	0.33	0.48	43.0	349.2	35.1	285.1	30.4	246.9
55	130	2.4	0.87	2.53	0.34	0.49	43.8	379.2	35.7	309.6	31.0	268.1
55	140	2.5	0.92	2.63	0.35	0.50	44.4	409.5	36.3	334.3	31.4	289.5
55	150	2.7	0.98	2.73	0.36	0.50	45.1	439.9	36.8	359.2	31.9	311.1
55	160	2.9	1.03	2.83	0.36	0.51	45.6	470.6	37.3	384.2	32.3	332.8
55	170	3.1	1.09	2.93	0.37	0.52	46.2	501.4	37.7	409.4	32.6	354.6
56	150	2.7	1.00	2.75	0.36	0.51	45.4	453.0	37.1	369.9	32.1	320.3
56	160	2.9	1.05	2.85	0.37	0.51	46.0	484.5	37.6	395.6	32.5	342.6
58	150	2.6	1.04	2.80	0.37	0.52	46.2	479.6	37.7	391.6	32.7	339.1
58	160	2.8	1.10	2.90	0.38	0.52	46.8	512.9	38.2	418.8	33.1	362.7

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TABLE - B7
Trapezoidal Canal Section

Side Slope = 0.5 H to 1V;
Bed Slope : 1 in S;

n = 0.025
V = cm/second
Q = litres / sec

							S = 1 in 2500		S = 1 in 3000		S = 1 in 4000	
d	b	b/d	A	P	R	$R^{(2/3)}$	V	Q	V	Q	V	Q
cm	cm		sqm	m								
60	150	2.5	1.08	2.84	0.38	0.52	42.0	453.3	38.3	413.8	33.2	358.4
60	160	2.7	1.14	2.94	0.39	0.53	42.5	484.8	38.8	442.5	33.6	383.2
60	170	2.8	1.20	3.04	0.39	0.54	43.0	516.4	39.3	471.4	34.0	408.3
60	180	3.0	1.26	3.14	0.40	0.54	43.5	548.2	39.7	500.4	34.4	433.4
60	190	3.2	1.32	3.24	0.41	0.55	44.0	580.1	40.1	529.6	34.7	458.6
60	200	3.3	1.38	3.34	0.41	0.55	44.4	612.2	40.5	558.9	35.1	484.0
62	180	2.9	1.31	3.19	0.41	0.55	44.2	578.1	40.3	527.7	34.9	457.0
62	190	3.1	1.37	3.29	0.42	0.56	44.6	611.8	40.8	558.5	35.3	483.6
64	180	2.8	1.36	3.23	0.42	0.56	44.9	608.7	41.0	555.6	35.5	481.2
64	190	3.0	1.42	3.33	0.43	0.57	45.3	644.1	41.4	587.9	35.8	509.2
65	170	2.6	1.32	3.15	0.42	0.56	44.7	588.1	40.8	536.9	35.3	464.9
65	180	2.8	1.38	3.25	0.42	0.56	45.2	624.2	41.3	569.8	35.7	493.5
65	190	2.9	1.45	3.35	0.43	0.57	45.7	660.4	41.7	602.9	36.1	522.1
65	200	3.1	1.51	3.45	0.44	0.58	46.1	696.9	42.1	636.1	36.5	550.9
65	210	3.2	1.58	3.55	0.44	0.58	46.5	733.4	42.5	669.5	36.8	579.8
65	220	3.4	1.64	3.65	0.45	0.59	46.9	770.2	42.8	703.1	37.1	608.9
66	190	2.9	1.47	3.38	0.44	0.57	46.0	677.0	42.0	618.0	36.4	535.2
66	200	3.0	1.54	3.48	0.44	0.58	46.5	714.3	42.4	652.1	36.7	564.7
68	190	2.8	1.52	3.42	0.45	0.58	46.7	710.6	42.6	648.7	36.9	561.8
68	200	2.9	1.59	3.52	0.45	0.59	47.1	749.7	43.0	684.4	37.2	592.7
70	200	2.9	1.65	3.57	0.46	0.60	47.8	785.8	43.6	717.3	37.8	621.2
70	210	3.0	1.72	3.67	0.47	0.60	48.2	826.9	44.0	754.9	38.1	653.7
70	220	3.1	1.79	3.77	0.47	0.61	48.6	868.2	44.4	792.6	38.5	686.4
70	230	3.3	1.86	3.87	0.48	0.61	49.0	909.7	44.8	830.4	38.8	719.1
70	240	3.4	1.93	3.97	0.49	0.62	49.4	951.2	45.1	868.4	39.1	752.0
70	250	3.6	2.00	4.07	0.49	0.62	49.8	993.0	45.4	906.5	39.3	785.0
72	220	3.1	1.84	3.81	0.48	0.62	49.3	908.7	45.0	829.5	39.0	718.4
72	240	3.3	1.99	4.01	0.50	0.63	50.1	995.6	45.7	908.8	39.6	787.1

74	220	3.0	1.90	3.85	0.49	0.62	50.0	950.0	45.6	867.2	39.5	751.0
74	240	3.2	2.05	4.05	0.51	0.63	50.8	1040.6	46.3	950.0	40.1	822.7

TABLE - B8

Trapezoidal Canal Section

Side Slope = 0.5 H to 1V;
Bed Slope : 1 in S;

n = 0.025
V = cm/second
Q = litres / sec

d	b	b/d	A	P	R	R(2/3)	S = 1 in 3000		S = 1 in 4000		S =1 in 5000	
							V	Q	V	Q	V	Q
cm	cm		sqm	m								
75	220	2.9	1.93	3.88	0.50	0.63	45.9	886.3	39.7	767.5	35.5	686.5
75	240	3.2	2.08	4.08	0.51	0.64	46.6	970.8	40.4	840.8	36.1	752.0
75	250	3.3	2.16	4.18	0.52	0.64	47.0	1013.3	40.7	877.6	36.4	784.9
75	260	3.5	2.23	4.28	0.52	0.65	47.3	1056.0	41.0	914.5	36.7	817.9
75	270	3.6	2.31	4.38	0.53	0.65	47.6	1098.7	41.3	951.5	36.9	851.1
75	280	3.7	2.38	4.48	0.53	0.66	47.9	1141.6	41.5	988.7	37.1	884.3
76	240	3.2	2.11	4.10	0.52	0.64	46.9	991.9	40.7	859.0	36.4	768.3
76	250	3.3	2.19	4.20	0.52	0.65	47.3	1035.3	41.0	896.6	36.6	801.9
78	250	3.2	2.25	4.24	0.53	0.66	47.9	1079.7	41.5	935.0	37.1	836.3
78	260	3.3	2.33	4.34	0.54	0.66	48.2	1125.1	41.8	974.3	37.4	871.5
80	280	3.5	2.56	4.59	0.56	0.68	49.5	1267.0	42.9	1097.2	38.3	981.4
80	300	3.8	2.72	4.79	0.57	0.69	50.1	1362.4	43.4	1179.8	38.8	1055.3
80	320	4.0	2.88	4.99	0.58	0.69	50.6	1458.2	43.8	1262.8	39.2	1129.5
80	340	4.3	3.04	5.19	0.59	0.70	51.1	1554.4	44.3	1346.2	39.6	1204.1
80	368	4.6	3.26	5.47	0.60	0.71	51.8	1689.7	44.8	1463.4	40.1	1308.9
80	38	0.5	0.62	2.17	0.29	0.44	31.8	198.6	27.6	172.0	24.7	153.8
82	280	3.4	2.63	4.63	0.57	0.69	50.1	1318.5	43.4	1141.9	38.8	1021.3
82	300	3.7	2.80	4.83	0.58	0.69	50.7	1417.8	43.9	1227.8	39.3	1098.2
84	280	3.3	2.70	4.68	0.58	0.69	50.7	1370.9	43.9	1187.2	39.3	1061.9
84	300	3.6	2.87	4.88	0.59	0.70	51.3	1474.0	44.4	1276.5	39.7	1141.8
85	280	3.3	2.74	4.70	0.58	0.70	51.0	1397.4	44.1	1210.1	39.5	1082.4
85	300	3.5	2.91	4.90	0.59	0.71	51.6	1502.4	44.7	1301.1	40.0	1163.8
85	320	3.8	3.08	5.10	0.60	0.71	52.2	1608.0	45.2	1392.6	40.4	1245.6
85	340	4.0	3.25	5.30	0.61	0.72	52.7	1714.1	45.7	1484.4	40.8	1327.7
85	360	4.2	3.42	5.50	0.62	0.73	53.2	1820.5	46.1	1576.6	41.2	1410.2

85	380	4.5	3.59	5.70	0.63	0.73	53.7	1927.3	46.5	1669.1	41.6	149
86	300	3.5	2.95	4.92	0.60	0.71	51.9	1531.1	45.0	1326.0	40.2	11[illegible]
86	320	3.7	3.12	5.12	0.61	0.72	52.5	1638.7	45.5	1419.1	40.7	12[illegible]
86	380	4.4	3.64	5.72	0.64	0.74	54.0	1964.0	46.8	1700.9	41.8	15[illegible]
88	300	3.4	3.03	4.97	0.61	0.72	52.5	1589.0	45.5	1376.1	40.7	12[illegible]
88	320	3.6	3.20	5.17	0.62	0.73	53.1	1700.6	46.0	1472.8	41.1	13[illegible]

TABLE - B9

Trapezoidal Canal Section

Side Slope = 0.5 H to 1V;
Bed Slope : 1 in S;

n = 0.025
V = cm/second
Q = litres / sec

d	b	b/d	A	P	R	R(2/3)	S = 1 in 4000		S = 1 in 5000		S =1 in 60[illegible]	
							V	Q	V	Q	V	Q
cm	cm		sqm	m								
90	280	3.1	2.93	4.81	0.61	0.72	45.4	1327.4	40.6	1187.2	37.1	10[illegible]
90	300	3.3	3.11	5.01	0.62	0.73	46.0	1427.0	41.1	1276.4	37.5	11[illegible]
90	320	3.6	3.29	5.21	0.63	0.74	46.5	1527.2	41.6	1366.0	38.0	12[illegible]
90	340	3.8	3.47	5.41	0.64	0.74	47.0	1627.8	42.0	1456.0	38.4	13[illegible]
90	360	4.0	3.65	5.61	0.65	0.75	47.4	1728.8	42.4	1546.3	38.7	14[illegible]
90	380	4.2	3.83	5.81	0.66	0.76	47.8	1830.2	42.8	1637.0	39.1	14[illegible]
92	300	3.3	3.18	5.06	0.63	0.73	46.5	1478.6	41.5	1322.5	37.9	1[illegible]
92	340	3.7	3.55	5.46	0.65	0.75	47.5	1686.6	42.5	1508.5	38.8	1[illegible]
94	340	3.6	3.64	5.50	0.66	0.76	48.0	1746.2	42.9	1561.8	39.2	1[illegible]
94	360	3.8	3.83	5.70	0.67	0.77	48.5	1854.5	43.4	1658.7	39.6	1[illegible]
95	300	3.2	3.30	5.12	0.64	0.75	47.2	1557.4	42.2	1393.0	38.5	1[illegible]
95	320	3.4	3.49	5.32	0.66	0.75	47.7	1666.6	42.7	1490.6	39.0	1[illegible]
95	340	3.6	3.68	5.52	0.67	0.76	48.3	1776.3	43.2	1588.7	39.4	1[illegible]
95	360	3.8	3.87	5.72	0.68	0.77	48.7	1886.4	43.6	1687.3	39.8	1[illegible]
95	380	4.0	4.06	5.92	0.69	0.78	49.2	1997.0	44.0	1786.2	40.1	1[illegible]
95	400	4.2	4.25	6.12	0.69	0.78	49.6	2107.9	44.3	1885.4	40.5	1[illegible]
96	360	3.8	3.92	5.75	0.68	0.77	49.0	1918.6	43.8	1716.0	40.0	1[illegible]
96	380	4.0	4.11	5.95	0.69	0.78	49.4	2031.0	44.2	1816.6	40.4	1[illegible]
98	360	3.7	4.01	5.79	0.69	0.78	49.5	1983.5	44.3	1774.1	40.4	1[illegible]
98	380	3.9	4.20	5.99	0.70	0.79	49.9	2099.7	44.7	1878.0	40.8	1[illegible]

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TABLE - B10
Trapezoidal Canal Section

Side Slope = 0.5 H to 1V;
Bed Slope : 1 in S;

n = 0.025
V = m/second
Q = Cubic metre per second

d	b	b/d	A	P	R	$R^{(2/3)}$	S = 1 in 4000 V	Q	S = 1 in 5000 V	Q	S = 1 in 6000 V	Q
1	3.4	3.4	3.90	5.64	0.69	0.78	0.5	1.9	0.4	1.7	0.4	1.6
1	3.6	3.6	4.10	5.84	0.70	0.79	0.5	2.0	0.4	1.8	0.4	1.7
1	3.8	3.8	4.30	6.04	0.71	0.80	0.5	2.2	0.5	1.9	0.4	1.8
1	4.0	4.0	4.50	6.24	0.72	0.80	0.5	2.3	0.5	2.0	0.4	1.9
1	4.2	4.2	4.70	6.44	0.73	0.81	0.5	2.4	0.5	2.2	0.4	2.0
1	4.4	4.4	4.90	6.64	0.74	0.82	0.5	2.5	0.5	2.3	0.4	2.1
1.1	4.0	3.6	5.01	6.46	0.77	0.84	0.5	2.7	0.5	2.4	0.4	2.2
1.1	4.2	3.8	5.23	6.66	0.78	0.85	0.5	2.8	0.5	2.5	0.4	2.3
1.1	4.4	4.0	5.45	6.86	0.79	0.86	0.5	3.0	0.5	2.6	0.4	2.4
1.1	4.6	4.2	5.67	7.06	0.80	0.86	0.5	3.1	0.5	2.8	0.4	2.5
1.1	4.8	4.4	5.89	7.26	0.81	0.87	0.5	3.2	0.5	2.9	0.4	2.6
1.1	5.0	4.5	6.11	7.46	0.82	0.87	0.6	3.4	0.5	3.0	0.5	2.8
1.2	4.8	4.0	6.48	7.48	0.87	0.91	0.6	3.7	0.5	3.3	0.5	3.0
1.2	5.0	4.2	6.72	7.68	0.87	0.91	0.6	3.9	0.5	3.5	0.5	3.2
1.2	5.2	4.3	6.96	7.88	0.88	0.92	0.6	4.1	0.5	3.6	0.5	3.3
1.2	5.4	4.5	7.20	8.08	0.89	0.93	0.6	4.2	0.5	3.8	0.5	3.4
1.2	5.6	4.7	7.44	8.28	0.90	0.93	0.6	4.4	0.5	3.9	0.5	3.6
1.2	5.8	4.8	7.68	8.48	0.91	0.94	0.6	4.5	0.5	4.1	0.5	3.7
1.3	5.5	4.2	8.00	8.41	0.95	0.97	0.6	4.9	0.5	4.4	0.5	4.0
1.3	6.0	4.6	8.65	8.91	0.97	0.98	0.6	5.4	0.6	4.8	0.5	4.4
1.3	6.2	4.8	8.91	9.11	0.98	0.99	0.6	5.5	0.6	5.0	0.5	4.5
1.3	6.4	4.9	9.17	9.31	0.98	0.99	0.6	5.7	0.6	5.1	0.5	4.7
1.3	6.8	5.2	9.69	9.71	1.00	1.00	0.6	6.1	0.6	5.5	0.5	5.0
1.3	7.0	5.4	9.95	9.91	1.00	1.00	0.6	6.3	0.6	5.6	0.5	5.1
1.4	6.0	4.3	9.38	9.13	1.03	1.02	0.6	6.0	0.6	5.4	0.5	4.9
1.4	6.5	4.6	10.08	9.63	1.05	1.03	0.7	6.6	0.6	5.9	0.5	5.4
1.4	7.0	5.0	10.78	10.13	1.06	1.04	0.7	7.1	0.6	6.4	0.5	5.8

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1.4	7.2	5.1	11.06	10.33	1.07	1.05	0.7	7.3	0.6	6.5	0.5	6.0
1.4	7.4	5.3	11.34	10.53	1.08	1.05	0.7	7.5	0.6	6.7	0.5	6.2
1.4	8.0	5.7	12.18	11.13	1.09	1.06	0.7	8.2	0.6	7.3	0.5	6.7
1.5	6.0	4.0	10.13	9.35	1.08	1.05	0.7	6.8	0.6	6.0	0.5	5.5
1.5	6.5	4.3	10.88	9.85	1.10	1.07	0.7	7.3	0.6	6.6	0.6	6.0
1.5	7.0	4.7	11.63	10.35	1.12	1.08	0.7	7.9	0.6	7.1	0.6	6.5
1.5	7.5	5.0	12.38	10.85	1.14	1.09	0.7	8.5	0.6	7.6	0.6	7.0
1.5	8.0	5.3	13.13	11.35	1.16	1.10	0.7	9.1	0.6	8.2	0.6	7.5
1.5	8.5	5.7	13.88	11.85	1.17	1.11	0.7	9.7	0.6	8.7	0.6	8.0

TABLE - B 11
Trapezoidal Canal Section

Side Slope = 0.5 H to 1V;
Bed Slope : 1 in S;

n = 0.025
V = m/second
Q = Cubic metre per second

d m	b m	b/d	A sqm	P m	R	$R^{(2/3)}$	S = 1 in 6000 V	Q	S = 1 in 8000 V	Q	S = 1 in 10000 V	Q
2.0	10.0	5.0	22.00	14.47	1.52	1.32	0.7	15.0	0.6	13.0	0.5	11.6
2.0	15.0	7.5	32.00	19.47	1.64	1.39	0.7	23.0	0.6	19.9	0.6	17.8
2.0	20.0	10.0	42.00	24.47	1.72	1.43	0.7	31.1	0.6	26.9	0.6	24.1
2.5	15.0	6.0	40.63	20.59	1.97	1.57	0.8	33.0	0.7	28.6	0.6	25.6
2.5	20.0	8.0	53.13	25.59	2.08	1.63	0.8	44.6	0.7	38.7	0.7	34.6
2.5	25.0	10.0	65.63	30.59	2.15	1.66	0.9	56.4	0.7	48.8	0.7	43.7
3.0	20.0	6.7	64.50	26.71	2.41	1.80	0.9	60.0	0.8	51.9	0.7	46.4
3.0	25.0	8.3	79.50	31.71	2.51	1.85	1.0	75.8	0.8	65.6	0.7	58.7
3.0	30.0	10.0	94.50	36.71	2.57	1.88	1.0	91.7	0.8	79.4	0.8	71.0
3.5	25.0	7.1	93.63	32.83	2.85	2.01	1.0	97.2	0.9	84.2	0.8	75.3
3.5	30.0	8.6	111.13	37.83	2.94	2.05	1.1	117.7	0.9	101.9	0.8	91.2
3.5	40.0	11.4	146.13	47.83	3.06	2.11	1.1	158.9	0.9	137.6	0.8	123.1
4.0	30.0	7.5	128.00	38.94	3.29	2.21	1.1	146.1	1.0	126.5	0.9	113.2
4.0	40.0	10.0	168.00	48.94	3.43	2.28	1.2	197.4	1.0	171.0	0.9	152.9
4.0	50.0	12.5	208.00	58.94	3.53	2.32	1.2	249.0	1.0	215.6	0.9	192.8
5.0	40.0	8.0	212.50	51.18	4.15	2.58	1.3	283.5	1.2	245.5	1.0	219.6
5.0	50.0	10.0	262.50	61.18	4.29	2.64	1.4	357.9	1.2	310.0	1.1	277.2
5.0	60.0	12.0	312.50	71.18	4.39	2.68	1.4	432.7	1.2	374.7	1.1	335.1

SECTION - C

5.3.0 Design of Open channels on the basis of maximum permissible velocity

5.3.1. Open channels which don't present silting problems are designed on principles of maximum permissible velocity:
maximum permissible velocity is the maximum velocity that will not cause erosion of the channel.
Maximum permissible velocity depends on the depth of flow in the channel and soil characteristics.

5.3.2 The limiting velocity (upper limit) corresponding to depth of flow in the channel is arrived at adopting kennedy's expression.

$V = Cd^{0.5}$ (for clear water)
where, V = velocity in meter per second
C = 0.47 for B.C; siltyand softer soils.
C = 0.60 for murram, gravelly and hard soils.
d = depth of flow in the channel in meters.

5.3.3. Maximum allowable velocity.

Under normal conditions velocities may be limited as under.

In large earthern channels, 1.1 meter per second.

In small earthern channels., 0.75 metre per second.

However in B.C soils the maximum velocity may be limited to 0.5 meters per second.

Table - C -1
Allowable Velocity $V = Cd^{0.5}$

Depth 'd' in metres	Velocity in meters per second	
	In BC and soft soil C = 0.47	In hard soil C = 0.6
0.2	0.21	0.27
0.4	0.3[illegible]	0.38
0.6	0.33	0.42
0.8	0.42	0.54
1.0	0.47	0.60
1.2	0.51	0.66
1.4		0.71
1.6		0.76
1.8		0.80
2.0		0.85
2.5		0.95
3.0		1.04
3.5		1.12
4.0		

5.3.4 Depth of flow d.

Depth of flow d in a channel is a function of discharge. The formula suggested to arrive at the first approximation of 'd' is

$d = K Q^{1/3}$
d = depth of flow in meters
K = Constant, 0.7
Q = Discharge in cumecs

The depth arrived at is for general guidance and values can be varied according to field conditions.

Table C-2 ($d = K Q^{1/3}$)

Q in Litres per second	d in Centimeters	Q in Cumecs	d in meters
20	20	4	1.1
80	30	5	1.2
200	40	6	13
400	50	8	1.4
600	60	10	1.5
1000	70	23	2.0
1500	80	80	3.0
2100	90	190	4.0
2900	100	370	5.0

5.3.5 Bed width to Depth ratios

Bed width to depth ratio in a channel depends upon many factors like cost of excavation, seepage characteristics of canal prism etc. Therefore bed width to depth ratios cannot be fixed rigidly in the unlined channels. Following b and d ratios are suggested for general guidance. These values are approximate and can be varied according to individual needs.

Bed width b may be arrived at taking into consideration the depth of flow d assumed and b to d ratio corresponding to the discharge of the channel.

Table - C-3

Bed width to depth of flow ratios

Discharge range	b/d
Upto 50 litres per second (0.05 cumes)	1 to 1.5
50 to 500 litres per second (0.05 to 0.5 cumes)	1.5 to 2.0
500 to 5000 litres per second (0.5 to 5 cumecs)	2 to 3.5
5 to 50 cumecs	35 to 6
50 to 200 cumecs	6 to 8
200 to 280 cumecs	8 to 10

5.3.6 Side Slopes of Canal The side slopes of a canal depend upon the stability of materials with which they are formed.

5.3.7 Water Side slope (Inner side slope/Upstream slope)
If the bed filling is proposed, wherever required, '1.5 H to 1 V side slopes on waterside are adequate for all soil groups upto full supply depth of 0.5 meters.

In the channels where most of the reaches are either in partial cut and fill or bed filling the complication in mark out arising out of the adoption of different slopes in cutting and embankment will be avoided by confirming the entire canal section to a uniform inner side slope of 1.5 to 1. The slope need not be altered due to change in classification of soils, if the depth of cut is 3 meters or less. However in deep cutting reaches the inside slopes should be decided depending upon the strata met with.

5.3.8 Outer side slopes (Away from water side slope / Down - stream slope)
1.5 H to IV side slope is adequate for soil groups for the heights as indicated below.

i) 5 metre height in GM, SM, CL, and CH of soils.
ii) 9 metre height in SC type
iii) 12 metre height in GC type

5.3.9 Rugosity Coefficient n
The value of rugosity coefficient varies according to physical roughness of the sides and bottom of the channel.

The value of n to be adopted in design depends upon the channel condition expected in future and the channel condition depends upon the standards of maintenance. Keeping this aspect in view it-is desirable that earthen channels are designed for n value of not less than 0.025.

Smaller channels (less than 0.3 cumec) may be designed with 'n' value equal to **0.0275.**

5.3.10 Bed fall in the channel
If the general lay of the country is flat, the bed fall in the channel may be adopted equal to the fall of the country.
If the country has steep fall suitable bed slope may be adopted such that the velocity generated should be less than the allowable velocity.

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5.3.11 Flow formula

Manning's formula which is derived from Chezy's fundamental equation is generally used in design of open channels.

$V = 1/n.\ R^{2/3}\ .\ S^{0.5}$

Where V = Mean velocity of flow in metres per second
R = Hydraulic mean depth
(area over perimetre A/P)
S = Bed fall I in S

Discharge Q = A X V Cumecs.

5.3.12 Design example

Design of a channel of one cumec carrying capacity having no silting problems. in BC Soils/Murrum soils.
(refer tables indicated - if necessary adopt interpolation)

1 st Step : Depth of flow from table 2 corresponding to Q
1 st approximation d = 0.7 metre.

2 nd Step : Allowable velocity from table 1 corresponding to 0.7m depth.
V = 39 Cm/sec BC soils

3 rd Step : Bed width from table 3
(0.7 X ratio corresponding to 1 cumec)
say b = 2.0 metres

4 th step : Inner side slopes as per para 7.
1.5 H to IV

5 th step : Rugosity Coefficient n as per para 9
n = 0.025

6 th Step : Choose typical sections from tables which are capable of carrying one cumec flow
a) section suitable in B.C., soils
i) d = 70 cm. S 1 in 4000
b = 280 cm.
V = 40.2 cm./Second
Q = 1082 litres/Sec.

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b) Section suitable in Hard soils

ii) d = 70 cm. S 1 in 3000
b = 220 cm.
V = 44.9 cm./Second
Q = 1021 litres/Sec.

iii) d = 72 cm. S 1 in 4000
b = 200 cm.
V = 44.9 cm./Second
Q = 996 litres/Sec.

iv)d = 74 cm. S 1 in 3000
b = 200 cm.
V = 45.6 cm./Second
Q = 1048 litres/Sec.

v) d = 75 cm. S 1 in 4000
b = 220 cm.
V = 40.0 cm./Second
Q = 1005 litres/Sec.

Section with lesser depth may be adopted where sufficient driving head is not available in the parent canal.

Standards adopted in Sriram Sagar Project for distributaries

1. a) Velocity $V = 1/n\ R^{2/3}\ S^{0.5}$
b) Velocity limitation $V = C\ d^{0.5}$
C = 0.8 for BC soil and maximum limit of V is 0.45 m/s
C = 1.1 for murrum soils and maximum limit of V is 0.76 m/s

2. Rugosity coefficient 'n'
Channels of above 0.28 cumecs n = 0.025
Channels of below 0.28 cumecs n = 0.0275

3. Inner side slopes for design and construction.
In B.C. soils 2H to 1 side slopes in cutting and embankment

In all other soils;	in cutting	1.5 H to 1 V
	in embankment	2 H to 1 V
	In all soils	2 H to 1 V

4. Outer side slopes — in all soils — 2 H to 1 V

5. Berms (in side). No berms are proposed in side the canal prism upto a depth of F.S.D + F.B.

6. Free Board

Above 4.25 cumec canals	0.90 metres
4.25 to 0.7 cumec canals	0.60 metres
0.70 to 0,03 cumec canals	0.45 metres

7. Top width of Banks.

Above 28.3 cumec canals	4.5/3.6 metres
8.5 to 28.3 cumec canals	4.5/1.8 metres
2.8 to 8.5 cumec canals	3.6/1.8 metres
1.4 to 2.8 cumec canals	2.7/1.8 metres
0.28 to 1.4 cumec canals	1.8/1.2 metres
below 0.28 cumec canals	0.9/0.9 metres

(larger width to be on the ayacut side of canal)

8. Bed width to depth ratios. proposed in accordance with U.S.B.R. practice.

discharge	depth in metres	b/d
1 cumec	0.7	1.8
2 cumec	0.9	2.2
10 cumec	1.6	3.1
50 cumec	2.8	5.8

9. Depth of dry cover saturated gradient.

Saturated gradient assumed	4 : 1 in clayey soils
	5 : 1 in murrum soils
Above 8.5 cumecs canals	Cover 1.2 metres
4.3 to 8.5 cumecs canals	cover 0.6 metres
Less than 4.3 cumecs canals	Cover 0.3 metres

10 Width of Land Acquisition clear of banks.

a) When the canal is in more than balancing depth of cutting.
Half the height of bank subject to a minimum of 1.5 m and total width rounded to nearest multiples of 3.3 m (11 ft)

b) When the canal is in less than B.D.C.

Full height of bank plus 1.5 m and the total width rounded to the nearest multiple of 3.3 (11 ft).

11. Duty. Distributary to be designed for following peak duties

i) Paddy 571 Hectares/cumec (40 acres per cusec)
ii) Irrigated dry 1429 Hectares/cumec (100 acres per cusec)
iii) Paddy and Irrigated dry

Determine the maximum discharge requirements during the over lap period from 1/11 to 10/11 taking paddy duty at
714 Hectares/cumec (50 acres/cusec) and
I.D. crop at 1643 Hectares/cumec (115 acres/cusec)

The highest of the above three governs the design of channels depending upon the localised area.

12. Seepage losses :

i) 2.44 cumecs per millions square metre of welted area in gravelly soils (8 cusecs per million sq.feet).

ii) 1.52 cumecs per million sq. metres (5 cusecs per million sq. feet) of wetted area in clay, clay loam light clay and clay looms.

The carrying capacity to be computed working from the tailend of each channel upwards to the head duly making allowances for seepage loss so as to ensure required water at each offtake.

13. Supply condition in parent canal.

a) Field channel sluices (00 4 cumeca or 1.5 cusecs & less)	Full supply condition in parent canal
b) In all other cases	Half supply condition in parent canal

14. Height of sill of Sluices

The driving head should not be less than 0.15 metre and 0.075 metre in case of distributaries and direct pipes respectively. If in any case it is not possible to get the driving head prescribed, the height of the sill should be reduced suitably to arrive at the minimum driving head.

TABLE - C4

Side Slope = 1.5 d and b in cms P in metres
n = 0.025 A in Sq.m V in cm/sec Q in Lit/sec.

d	b	b/d	A	P	R	$R^{(2/3)}$	S = 1 in 1000 V	S = 1 in 1000 Q	S = 1 in 2000 V	S = 1 in 2000 Q	S = 1 in 3000 V	S = 1 in 3000 Q
20	20	1.0	0.10	0.92	0.11	0.23	28.8	28.8	20.4	20.4	16.6	16.6
20	30	1.5	0.12	1.02	0.12	0.24	30.3	36.4	21.5	25.8	17.5	21.0
20	40	2.0	0.14	1.12	0.12	0.25	31.6	44.2	22.3	31.3	18.2	25.5
20	50	2.5	0.16	1.22	0.13	0.26	32.6	52.2	23.1	36.9	18.8	30.1
20	60	3.0	0.18	1.32	0.14	0.26	33.5	60.3	23.7	42.6	19.3	34.8
20	70	3.5	0.20	1.42	0.14	0.27	34.2	68.4	24.2	48.4	19.8	39.5
22	30	1.4	0.14	1.09	0.13	0.25	31.9	44.2	22.6	31.3	18.4	25.5
22	40	1.8	0.16	1.19	0.13	0.26	33.2	53.4	23.5	37.7	19.2	30.8
24	40	1.7	0.18	1.27	0.14	0.27	34.8	63.4	24.6	44.9	20.1	36.6
24	50	2.1	0.21	1.37	0.15	0.28	35.9	74.1	25.4	52.4	20.7	42.8
25	40	1.6	0.19	1.30	0.15	0.28	35.5	68.8	25.1	48.7	20.5	39.7
25	50	2.0	0.22	1.40	0.16	0.29	36.7	80.2	25.8	56.7	21.2	46.3

25	60	2.4	0.24	1.50	0.16	0.30	37.6	91.8	26.6	64.9	21.7	53.0
25	70	2.8	0.27	1.60	0.17	0.30	38.5	103.4	27.2	73.1	22.2	59.7
25	80	3.2	0.29	1.70	0.17	0.31	39.2	115.2	27.7	81.5	22.6	66.5
25	90	3.6	0.32	1.80	0.18	0.32	39.9	127.1	28.2	89.9	23.0	73.4
26	40	1.5	0.21	1.34	0.15	0.29	36.3	74.5	25.7	52.7	20.9	43.0
26	50	1.9	0.23	1.44	0.16	0.30	37.4	86.6	26.5	61.2	21.6	50.0
28	50	1.8	0.26	1.51	0.17	0.31	38.9	100.2	27.5	70.9	22.5	57.9
28	60	2.1	0.29	1.61	0.18	0.32	39.9	114.1	28.2	80.7	23.1	65.9
30	50	1.7	0.29	1.58	0.18	0.32	40.4	115.0	28.5	81.3	23.3	66.4
30	60	2.0	0.32	1.68	0.19	0.33	41.4	130.4	29.3	92.2	23.9	75.3
30	70	2.3	0.35	1.78	0.19	0.33	42.3	146.1	29.9	103.3	24.4	84.3
30	80	2.7	0.38	1.88	0.20	0.34	43.2	161.8	30.5	114.4	24.9	93.4
30	90	3.0	0.41	1.98	0.20	0.35	43.9	177.7	31.0	125.7	25.3	102.6
30	100	3.3	0.44	2.08	0.21	0.35	44.5	193.8	31.5	137.0	25.7	111.9
32	50	1.6	0.31	1.65	0.19	0.33	41.8	130.9	29.5	92.6	24.1	75.6
32	60	1.9	0.35	1.75	0.20	0.34	42.8	148.0	30.3	104.7	24.7	85.5
34	60	1.8	0.38	1.83	0.21	0.35	44.2	166.9	31.3	118.0	25.5	96.4
34	70	2.1	0.41	1.93	0.21	0.36	45.2	186.0	32.0	131.5	26.1	107.4
35	60	1.7	0.39	1.86	0.21	0.35	44.9	176.8	31.7	125.0	25.9	102.1
35	70	2.0	0.43	1.96	0.22	0.36	45.9	196.8	32.5	139.1	26.5	113.6
35	80	2.3	0.46	2.06	0.22	0.37	46.8	216.9	33.1	153.4	27.0	125.3
35	90	2.6	0.50	2.16	0.23	0.38	47.6	237.3	33.6	167.8	27.5	137.0
35	100	2.9	0.53	2.26	0.24	0.38	48.3	257.8	34.2	182.3	27.9	148.8
35	110	3.1	0.57	2.36	0.24	0.39	49.0	278.5	34.6	196.9	28.3	160.8
36	60	1.7	0.41	1.90	0.22	0.36	45.6	187.0	32.2	132.2	26.3	108.0
36	70	1.9	0.45	2.00	0.22	0.37	46.6	207.9	32.9	147.0	26.9	120.0
38	60	1.6	0.44	1.97	0.23	0.37	46.9	208.5	33.2	147.4	27.1	120.4
38	70	1.8	0.48	2.07	0.23	0.38	47.9	231.2	33.9	163.5	27.7	133.5
40	70	1.8	0.52	2.14	0.24	0.39	49.2	256.0	34.8	181.0	28.4	147.8
40	80	2.0	0.56	2.24	0.25	0.40	50.2	280.9	35.5	198.6	29.0	162.2
40	90	2.3	0.60	2.34	0.26	0.40	51.0	306.1	36.1	216.5	29.5	176.7
40	100	2.5	0.64	2.44	0.26	0.41	51.8	331.5	36.6	234.4	29.9	191.4
40	110	2.8	0.68	2.54	0.27	0.42	52.5	357.1	37.1	252.5	30.3	206.2
40	120	3.0	0.72	2.64	0.27	0.42	53.2	382.8	37.6	270.7	30.7	221.0
42	70	1.7	0.56	2.21	0.25	0.40	50.5	282.1	35.7	199.5	29.2	162.9
42	80	1.9	0.60	2.31	0.26	0.41	51.5	309.1	36.4	218.6	29.7	178.5
44	70	1.6	0.60	2.29	0.26	0.41	51.8	309.7	36.6	219.0	29.9	178.8
44	80	1.8	0.64	2.39	0.27	0.42	52.7	338.8	37.3	239.5	30.4	195.6

45	70	1.6	0.62	2.32	0.27	0.41	52.4	324.1	37.0	229.1	30.2	187.1
45	80	1.8	0.66	2.42	0.27	0.42	53.4	354.2	37.7	250.4	30.8	204.5
45	90	2.0	0.71	2.52	0.28	0.43	54.3	384.6	38.4	271.9	31.3	222.0
45	100	2.2	0.75	2.62	0.29	0.44	55.1	415.2	39.0	293.6	31.8	239.7
45	110	2.4	0.80	2.72	0.29	0.44	55.8	446.1	39.5	315.4	32.2	257.6
45	120	2.7	0.84	2.82	0.30	0.45	56.6	477.2	40.0	337.4	32.7	275.5
46	80	1.7	0.69	2.46	0.28	0.43	54.0	370.0	38.2	261.6	31.2	213.6
46	90	2.0	0.73	2.56	0.29	0.43	54.9	401.5	38.8	283.9	31.7	231.8
48	80	1.7	0.73	2.53	0.29	0.44	55.2	402.8	39.0	284.8	31.9	232.5
48	90	1.9	0.78	2.63	0.30	0.44	56.1	436.5	39.7	308.6	32.4	252.0

TABLE - C5

Side Slope = 1.5   d and b in cms   P in metres   Q in Lit/sec
n = 0.025   A in Sq.m   V in cm/sec

							S = 1 in 2000		S = 1 in 3000		S = 1 in 4000	
d	b	b/d	A	P	R	$R^{(2/3)}$	V	Q	V	Q	V	Q
50	90	1.8	0.83	2.70	0.31	0.45	40.5	334.5	33.1	273.1	28.7	236.5
50	100	2.0	0.88	2.80	0.31	0.46	41.2	360.2	33.6	294.1	29.1	254.7
50	120	2.4	0.98	3.00	0.32	0.47	42.3	412.0	34.5	336.4	29.9	291.3
50	140	2.8	1.08	3.20	0.34	0.48	43.2	464.4	35.3	379.2	30.5	328.4
50	150	3.0	1.13	3.30	0.34	0.49	43.6	490.8	35.6	400.7	30.8	347.0
50	160	3.2	1.18	3.40	0.35	0.49	44.0	517.3	35.9	422.4	31.1	365.8
52	100	1.9	0.93	2.87	0.32	0.47	42.0	388.9	34.3	317.5	29.7	275.0
52	120	2.3	1.03	3.07	0.33	0.48	43.1	444.1	35.2	362.6	30.5	314.0
54	100	1.9	0.98	2.95	0.33	0.48	42.9	418.9	35.0	342.0	30.3	296.2
54	120	2.2	1.09	3.15	0.34	0.49	44.0	477.5	35.9	389.8	31.1	337.6
55	100	1.8	1.00	2.98	0.34	0.48	43.3	434.3	35.3	354.6	30.6	307.1
55	120	2.2	1.11	3.18	0.35	0.50	44.4	494.6	36.3	403.9	31.4	349.8
55	140	2.5	1.22	3.38	0.36	0.51	45.4	555.7	37.1	453.7	32.1	392.9
55	150	2.7	1.28	3.48	0.37	0.51	45.9	586.4	37.4	478.8	32.4	414.7
55	160	2.9	1.33	3.58	0.37	0.52	46.3	617.3	37.8	504.0	32.7	436.5
55	180	3.3	1.44	3.78	0.38	0.53	47.1	679.4	38.4	554.7	33.3	480.4
56	120	2.1	1.14	3.22	0.35	0.50	44.8	512.2	36.6	418.2	31.7	362.2
56	140	2.5	1.25	3.42	0.37	0.51	45.8	575.0	37.4	469.5	32.4	406.6

58	120	2.1	1.20	3.29	0.36	0.51	45.7	548.2	37.3	447.6	32.3	387.7
58	140	2.4	1.32	3.49	0.38	0.52	46.7	614.7	38.1	501.9	33.0	434.6
60	120	2.0	1.26	3.36	0.37	0.52	46.5	585.7	38.0	478.2	32.9	414.1
60	140	2.3	1.38	3.56	0.39	0.53	47.5	655.8	38.8	535.5	33.6	463.7
60	150	2.5	1.44	3.66	0.39	0.54	48.0	691.1	39.2	564.3	33.9	488.7
60	160	2.7	1.50	3.76	0.40	0.54	48.4	726.6	39.6	593.3	34.3	513.8
60	180	3.0	1.62	3.96	0.41	0.55	49.3	798.0	40.2	651.6	34.8	564.3
60	200	3.3	1.74	4.16	0.42	0.56	50.0	870.0	40.8	710.3	35.4	615.2
62	130	2.1	1.38	3.54	0.39	0.53	47.8	661.3	39.1	540.0	33.8	467.6
62	140	2.3	1.44	3.64	0.40	0.54	48.3	698.4	39.5	570.2	34.2	493.8
64	140	2.2	1.51	3.71	0.41	0.55	49.2	742.4	40.1	606.2	34.8	525.0
64	150	2.3	1.57	3.81	0.41	0.56	49.6	781.6	40.5	638.2	35.1	552.7

TABLE - C6

Side Slope = 1.5   d and b in cms   P in metres   Q in Lit/sec.
n = 0.025   A in Sq.m   V in cm/sec

							S = 1 in 3000		S = 1 in 4000		S = 1 in 5000	
d	b	b/d	A	P	R	$R^{(2/3)}$	V	Q	V	Q	V	Q
65	140	2.2	1.54	3.74	0.41	0.55	40.5	624.6	35.0	540.9	31.3	483.8
65	150	2.3	1.61	3.84	0.42	0.56	40.9	657.4	35.4	569.3	31.7	509.2
65	160	2.5	1.67	3.94	0.42	0.56	41.2	690.3	35.7	597.8	31.9	534.7
65	180	2.8	1.80	4.14	0.44	0.57	41.9	756.6	36.3	655.2	32.5	586.1
65	200	3.1	1.93	4.34	0.45	0.58	42.6	823.4	36.9	713.1	33.0	637.8
65	220	3.4	2.06	4.54	0.45	0.59	43.2	890.6	37.4	771.2	33.4	689.8
66	150	2.3	1.64	3.88	0.42	0.56	41.2	676.9	35.7	586.2	31.9	524.3
66	160	2.4	1.71	3.98	0.43	0.57	41.6	710.7	36.0	615.5	32.2	550.5
66	180	2.7	1.84	4.18	0.44	0.58	42.3	778.6	36.6	674.3	32.8	603.1
66	200	3.0	1.97	4.38	0.45	0.59	42.9	847.0	37.2	733.5	33.2	656.1
66	220	3.3	2.11	4.58	0.46	0.60	43.5	915.9	37.7	793.2	33.7	709.4
66	240	3.6	2.24	4.78	0.47	0.60	44.0	985.1	38.1	853.1	34.1	763.0
68	150	2.2	1.71	3.95	0.43	0.57	41.8	716.9	36.2	620.9	32.4	555.3
68	160	2.4	1.78	4.05	0.44	0.58	42.2	752.3	36.6	651.6	32.7	582.8
68	180	2.6	1.92	4.25	0.45	0.59	42.9	823.6	37.2	713.3	33.3	638.0
68	200	2.9	2.05	4.45	0.46	0.60	43.6	895.4	37.8	775.4	33.8	693.6

68	220	3.2	2.19	4.65	0.47	0.61	44.2	967.3	38.3	838.0	34.2	749.5
68	240	3.5	2.33	4.85	0.48	0.61	44.7	1040.2	38.7	900.9	34.6	805.7
70	180	2.6	2.00	4.32	0.46	0.60	43.6	869.9	37.8	753.4	33.8	673.9
70	200	2.9	2.14	4.52	0.47	0.61	44.3	945.1	38.3	818.5	34.3	732.1
70	220	3.1	2.28	4.72	0.48	0.61	44.9	1020.8	38.9	884.0	34.8	790.7
70	240	3.4	2.42	4.92	0.49	0.62	45.4	1096.9	39.3	949.9	35.2	849.6
70	260	3.7	2.56	5.12	0.50	0.63	45.9	1173.3	39.8	1016.1	35.6	908.9
70	280	4.0	2.70	5.32	0.51	0.64	46.4	1250.1	40.2	1082.6	35.9	968.3
72	180	2.5	2.07	4.40	0.47	0.61	44.3	917.6	38.3	794.7	34.3	710.8
72	200	2.8	2.22	4.60	0.48	0.62	44.9	996.3	38.9	862.8	34.8	771.7
74	180	2.4	2.15	4.47	0.48	0.61	44.9	966.7	38.9	837.2	34.8	748.8
74	200	2.7	2.30	4.67	0.49	0.62	45.6	1048.9	39.5	908.4	35.3	812.5
75	180	2.4	2.19	4.50	0.49	0.62	45.2	991.7	39.2	858.9	35.0	768.2
75	200	2.7	2.34	4.70	0.50	0.63	45.9	1075.7	39.7	931.6	35.6	833.2
75	220	2.9	2.49	4.90	0.51	0.64	46.5	1160.2	40.3	1004.8	36.0	898.7
75	240	3.2	2.64	5.10	0.52	0.64	47.1	1245.2	40.8	1078.4	36.5	964.6
75	260	3.5	2.79	5.30	0.53	0.65	47.6	1330.7	41.2	1152.4	36.9	1030.7
75	280	3.7	2.94	5.50	0.53	0.66	48.1	1416.5	41.7	1226.7	37.3	1097.[illegible]
76	180	2.4	2.23	4.54	0.49	0.62	45.5	1017.1	39.4	880.9	35.3	787.9
76	200	2.6	2.39	4.74	0.50	0.63	46.2	1102.9	40.0	955.1	35.8	854.3
78	180	2.3	2.32	4.61	0.50	0.63	46.1	1069.0	40.0	925.8	35.7	828.0
78	200	2.6	2.47	4.81	0.51	0.64	46.8	1158.4	40.6	1003.2	36.3	897.3
80	200	2.5	2.56	4.88	0.52	0.65	47.5	1215.3	41.1	1052.5	36.8	941.4
80	250	3.1	2.96	5.38	0.55	0.67	49.0	1450.6	42.4	1256.3	38.0	1123.7
80	260	3.3	3.04	5.48	0.55	0.67	49.3	1498.1	42.7	1297.4	38.2	1160.4
80	280	3.5	3.20	5.68	0.56	0.68	49.8	1593.3	43.1	1379.8	38.6	1234.1
80	300	3.8	3.36	5.88	0.57	0.69	50.3	1688.9	43.5	1462.6	38.9	1308.2
80	320	4.0	3.52	6.08	0.58	0.69	50.7	1784.8	43.9	1545.7	39.3	1382.5
82	200	2.4	2.65	4.96	0.53	0.66	48.1	1273.7	41.6	1103.1	37.3	986.6
82	250	3.0	3.06	5.46	0.56	0.68	49.6	1518.5	43.0	1315.1	38.5	1176.3
84	200	2.4	2.74	5.03	0.54	0.67	48.7	1333.6	42.2	1154.9	37.7	1033.0
84	250	3.0	3.16	5.53	0.57	0.69	50.3	1588.1	43.5	1375.3	38.9	1230.1
85	200	2.4	2.78	5.06	0.55	0.67	49.0	1364.1	42.4	1181.3	38.0	1056.6
85	250	2.9	3.21	5.56	0.58	0.69	50.6	1623.4	43.8	1405.9	39.2	1257.5
85	260	3.1	3.29	5.66	0.58	0.70	50.9	1675.7	44.1	1451.2	39.4	1298.0
85	280	3.3	3.46	5.86	0.59	0.70	51.4	1780.7	44.5	1542.1	39.8	1379.3
85	300	3.5	3.63	6.06	0.60	0.71	51.9	1886.0	44.9	1633.4	40.2	1460.9
85	320	3.8	3.80	6.26	0.61	0.72	52.4	1991.8	45.3	1725.0	40.6	1542.9

86	200	2.3	2.83	5.10	0.55	0.68	49.3	1395.0	42.7	1208.1	38.2	1080.5
86	250	2.9	3.26	5.60	0.58	0.70	50.9	1659.2	44.1	1436.9	39.4	1285.2
88	200	2.3	2.92	5.17	0.56	0.68	49.9	1457.8	43.2	1262.5	38.7	1129.2
88	250	2.8	3.36	5.67	0.59	0.71	51.5	1732.0	44.6	1499.9	39.9	1341.6
90	240	2.7	3.38	5.64	0.60	0.71	51.8	1749.2	44.9	1514.9	40.1	1354.9
90	260	2.9	3.56	5.84	0.61	0.72	52.4	1863.7	45.4	1614.0	40.6	1443.6
90	280	3.1	3.74	6.04	0.62	0.73	53.0	1978.7	45.9	1713.6	41.0	1532.7
90	300	3.3	3.92	6.24	0.63	0.73	53.5	2094.3	46.3	1813.7	41.4	1622.2
90	320	3.6	4.10	6.44	0.64	0.74	54.0	2210.2	46.7	1914.1	41.8	1712.0
90	340	3.8	4.28	6.64	0.64	0.75	54.4	2326.6	47.1	2014.9	42.2	1802.2
92	240	2.6	3.48	5.72	0.61	0.72	52.4	1823.3	45.4	1579.0	40.6	1412.3
92	260	2.8	3.66	5.92	0.62	0.73	53.0	1941.8	45.9	1681.7	41.1	1504.1
94	240	2.6	3.58	5.79	0.62	0.73	53.0	1898.9	45.9	1644.5	41.1	1470.9
94	260	2.8	3.77	5.99	0.63	0.73	53.6	2021.7	46.4	1750.8	41.5	1566.0
95	260	2.7	3.82	6.03	0.63	0.74	53.9	2062.2	46.7	1785.9	41.8	1597.4
95	280	2.9	4.01	6.23	0.64	0.75	54.5	2187.6	47.2	1894.6	42.2	1694.5
95	300	3.2	4.20	6.43	0.65	0.75	55.0	2313.7	47.7	2003.7	42.6	1792.2
95	320	3.4	4.39	6.63	0.66	0.76	55.5	2440.2	48.1	2113.3	43.0	1890.2
95	340	3.6	4.58	6.83	0.67	0.77	56.0	2567.2	48.5	2223.2	43.4	1988.5
95	360	3.8	4.77	7.03	0.68	0.77	56.4	2694.6	48.9	2333.6	43.7	2087.2
96	260	2.7	3.88	6.06	0.64	0.74	54.2	2103.2	47.0	1821.4	42.0	1629.1
96	280	2.9	4.07	6.26	0.65	0.75	54.8	2230.7	47.5	1931.9	42.5	1727.9
98	260	2.7	3.99	6.13	0.65	0.75	54.8	2186.4	47.5	1893.5	42.5	1693.6
98	280	2.9	4.18	6.33	0.66	0.76	55.4	2318.3	48.0	2007.7	42.9	1795.7

TABLE - C6a

Side Slope = 1.5 d and b in m P in metres

n = 0.025 A in Sq.m V in m/sec Q in Cumecs

							S = 1 in 3000		S = 1 in 4000		S = 1 in 5000	
d	b	b/d	A	P	R	$R^{(2/3)}$	V	Q	V	Q	V	Q
1	2.6	2.6	4.10	6.21	0.66	0.76	0.6	2.3	0.5	2.0	0.4	1.8
1	2.8	2.8	4.30	6.41	0.67	0.77	0.6	2.4	0.5	2.1	0.4	1.9
1	3.0	3.0	4.50	6.61	0.68	0.77	0.6	2.5	0.5	2.2	0.4	2.0
1	3.2	3.2	4.70	6.81	0.69	0.78	0.6	2.7	0.5	2.3	0.4	2.1
1	3.4	3.4	4.90	7.01	0.70	0.79	0.6	2.8	0.5	2.4	0.4	2.2

1	3.6	3.6	5.10	7.21	0.71	0.79	0.6	3.0	0.5	2.6	0.4	2.3
1	3.8	3.8	5.30	7.41	0.72	0.80	0.6	3.1	0.5	2.7	0.5	2.4
1	4.0	4.0	5.50	7.61	0.72	0.81	0.6	3.2	0.5	2.8	0.5	2.5
1	4.2	4.2	5.70	7.81	0.73	0.81	0.6	3.4	0.5	2.9	0.5	2.6
1.1	2.8	2.5	4.90	6.77	0.72	0.81	0.6	2.9	0.5	2.5	0.5	2.2
1.1	3.0	2.7	5.12	6.97	0.73	0.81	0.6	3.0	0.5	2.6	0.5	2.4
1.1	3.2	2.9	5.34	7.17	0.74	0.82	0.6	3.2	0.5	2.8	0.5	2.5
1.1	3.4	3.1	5.56	7.37	0.75	0.83	0.6	3.4	0.5	2.9	0.5	2.6
1.1	3.6	3.3	5.78	7.57	0.76	0.84	0.6	3.5	0.5	3.1	0.5	2.7
1.1	3.8	3.5	6.00	7.77	0.77	0.84	0.6	3.7	0.5	3.2	0.5	2.9
1.1	4.0	3.6	6.22	7.97	0.78	0.85	0.6	3.8	0.5	3.3	0.5	3.0
1.1	4.2	3.8	6.44	8.17	0.79	0.85	0.6	4.0	0.5	3.5	0.5	3.1
1.1	4.4	4.0	6.66	8.37	0.80	0.86	0.6	4.2	0.5	3.6	0.5	3.2
1.2	3.0	2.5	5.76	7.33	0.79	0.85	0.6	3.6	0.5	3.1	0.5	2.8
1.2	3.6	3.0	6.48	7.93	0.82	0.87	0.6	4.1	0.6	3.6	0.5	3.2
1.2	4.0	3.3	6.96	8.33	0.84	0.89	0.6	4.5	0.6	3.9	0.5	3.5
1.2	4.2	3.5	7.20	8.53	0.84	0.89	0.7	4.7	0.6	4.1	0.5	3.6
1.2	4.4	3.7	7.44	8.73	0.85	0.90	0.7	4.9	0.6	4.2	0.5	3.8
1.2	4.6	3.8	7.68	8.93	0.86	0.90	0.7	5.1	0.6	4.4	0.5	3.9
1.2	4.8	4.0	7.92	9.13	0.87	0.91	0.7	5.3	0.6	4.6	0.5	4.1
1.2	5.0	4.2	8.16	9.33	0.87	0.91	0.7	5.5	0.6	4.7	0.5	4.2
1.2	5.2	4.3	8.40	9.53	0.88	0.92	0.7	5.6	0.6	4.9	0.5	4.4

TABLE - C7

Side Slope = 1.5 d and b in m P in metres
n = 0.025 A in Sq.m V in m/sec Q in Cumecs

d	b	b/d	A	P	R	$R^{(2/3)}$	S = 1 in 4000 V	Q	S = 1 in 5000 V	Q	S = 1 in 6000 V	Q
1.3	3.5	2.7	7.09	8.19	0.87	0.91	0.57	4.07	0.51	3.64	0.47	3.32
1.3	4.0	3.1	7.74	8.69	0.89	0.93	0.59	4.53	0.52	4.05	0.48	3.70
1.3	4.5	3.5	8.39	9.19	0.91	0.94	0.60	4.99	0.53	4.46	0.49	4.07
1.3	5.0	3.8	9.04	9.69	0.93	0.95	0.60	5.45	0.54	4.88	0.49	4.45
1.3	5.5	4.2	9.69	10.19	0.95	0.97	0.61	5.92	0.55	5.30	0.50	4.84
1.3	6.0	4.6	10.34	10.69	0.97	0.98	0.62	6.39	0.55	5.72	0.50	5.22
1.4	4.0	2.9	8.54	9.05	0.94	0.96	0.61	5.20	0.54	4.65	0.50	4.24

1.4	4.5	3.2	9.24	9.55	0.97	0.98	0.62	5.72	0.55	5.11	0.51	4.67
1.4	5.0	3.6	9.94	10.05	0.99	0.99	0.63	6.24	0.56	5.58	0.51	5.10
1.4	5.5	3.9	10.64	10.55	1.01	1.01	0.64	6.77	0.57	6.05	0.52	5.53
1.4	6.0	4.3	11.34	11.05	1.03	1.02	0.64	7.30	0.58	6.53	0.53	5.96
1.4	6.5	4.6	12.04	11.55	1.04	1.03	0.65	7.83	0.58	7.00	0.53	6.39
1.5	4.0	2.7	9.38	9.41	1.00	1.00	0.63	5.92	0.56	5.29	0.52	4.83
1.5	4.5	3.0	10.13	9.91	1.02	1.01	0.64	6.50	0.57	5.81	0.52	5.30
1.5	5.0	3.3	10.88	10.41	1.04	1.03	0.65	7.08	0.58	6.33	0.53	5.78
1.5	5.5	3.7	11.63	10.91	1.07	1.04	0.66	7.67	0.59	6.86	0.54	6.26
1.5	6.0	4.0	12.38	11.41	1.08	1.06	0.67	8.26	0.60	7.39	0.55	6.75
1.5	7.0	4.7	13.88	12.41	1.12	1.08	0.68	9.45	0.61	8.46	0.56	7.72
1.6	4.0	2.5	10.24	9.77	1.05	1.03	0.65	6.68	0.58	5.98	0.53	5.46
1.6	6.0	3.8	13.44	11.77	1.14	1.09	0.69	9.29	0.62	8.31	0.56	7.58
1.6	8.0	5.0	16.64	13.77	1.21	1.13	0.72	11.94	0.64	10.68	0.59	9.75
1.7	6.0	3.5	14.54	12.13	1.20	1.13	0.71	10.37	0.64	9.28	0.58	8.47
1.7	8.0	4.7	17.94	14.13	1.27	1.17	0.74	13.30	0.66	11.89	0.61	10.86
1.7	10.0	5.9	21.34	16.13	1.32	1.20	0.76	16.26	0.68	14.54	0.62	13.28
1.8	6.0	3.3	15.66	12.49	1.25	1.16	0.74	11.52	0.66	10.30	0.60	9.40
1.8	8.0	4.4	19.26	14.49	1.33	1.21	0.76	14.73	0.68	13.17	0.62	12.02
1.8	10.0	5.6	22.86	16.49	1.39	1.24	0.79	17.98	0.70	16.08	0.64	14.68
1.9	6.0	3.2	16.82	12.85	1.31	1.20	0.76	12.72	0.68	11.38	0.62	10.39
1.9	8.0	4.2	20.62	14.85	1.39	1.24	0.79	16.22	0.70	14.51	0.64	13.25
1.9	10.0	5.3	24.42	16.85	1.45	1.28	0.81	19.77	0.72	17.68	0.66	16.14
2	8.0	4.0	22.00	15.21	1.45	1.28	0.81	17.79	0.72	15.92	0.66	14.53
2	10.0	5.0	26.00	17.21	1.51	1.32	0.83	21.65	0.74	19.36	0.68	17.68
2	12.0	6.0	30.00	19.21	1.56	1.35	0.85	25.54	0.76	22.84	0.70	20.85
2.5	10.0	4.0	34.38	19.01	1.81	1.48	0.94	32.26	0.84	28.86	0.77	26.34
2.5	12.0	4.8	39.38	21.01	1.87	1.52	0.96	37.85	0.86	33.85	0.78	30.90
2.5	14.0	5.6	44.38	23.01	1.93	1.55	0.98	43.48	0.88	38.89	0.80	35.50

Courtesy : " Guide to Design of Irrigation Channels " by Ch. Venkata Ramaiah.

STANDARDS FOR CANALS AS PER IS STANDARDS

1. **Canal Discharge :**

a) Crop water requirement by Modified Penman Method.

b) Seepage Loses : Lines -0.6 Cumecs / Million sq.m. of wetted surface.

(Note in cl 2:1 (a) of IS : 10430 - 1982

Unlined - 1. 1.85 to 2.40 Cumescs / Million Sq.ms of wetted Surface. (20% Extra for Distributory System (Cl. 3.34.1 (h) of Civil Engineer's Hand Book).

2.44 Cumecs / Million Sq.ms of W.S. in gravelly soils

1.52 Cumecs / Million Sq.ms of W.S. in clay, clay loam;

(Cl. 12, 5-35 of Civil Engineer's Hand Book)

2. **C.W.C. Bed width to depth ratios as per draft manual on irrigation channels, C.W.C. Water-wing, New Delhi, 1960**

a) **b/d Ratio.**

S.No.	Discharge in Cum/second	B/D	S.No.	Discharge in Cum/second	B/D
1.	0.283	2.9	13.	11.327	5.7
2.	0.425	3.0	14.	14.158	6.0
3.	0.566	3.2	15.	28.317	7.4
4.	0.850	3.4	16.	33.980	7.9
5.	1.133	3.6	17.	39.644	8.2
6.	1.416	3.7	18.	45.307	8.6
7.	2.124	4.0	19.	50.634	9.0
8.	2.832	4.2	20.	56.634	9.5
9.	3.540	4.4	21.	70.792	10.1
10.	4.247	4.6	22.	84.950	11.0
11.	5.663	4.8	23.	283.170	19.2
12.	8.495	5.1			

b) **Critical velocity ratio v/vo :**

i) Diversions, silt carrying unlined canals 1.1 - 0.90 (Head - Tail end)

ii) Lined canals, v / vo > 1 is preferable.

$V_o = 0.39\ d^{0.55}$ For Godavari Delta System

$V_o = 0.53\ d^{0.53}$ For Krishna Delta System

c) **Coe. of Rugosity 'n' :**

Lined canal	-	0.018
Normal alluvial soils & Murum	-	0.0025
Rocky strata	-	0.03 - 0.035

d) **Allowable velocites : As per C.B.I.P. Publication No. 171**

I. Unlined canals - Recommended Velocities :

1.	All Soils	0.6 to 1.1m/sec
2.	Hard clay or grit	1.0 to 1.5m/sec
3.	Gravel and shingle	1.5 to 1.8m/sec
4.	Cemented gravel conglomite, hard pan	1.8 m/sec
5.	Soft Rock	1.4 m/sec.
6.	Hard rock	2.4 m/sec.
7.	Very Hard Rock	4.5 m/sec.

II. Lined canals - For CC lining max permissible velocity - 2.7 m/s.

e) **Side slopes :**

a) Canal in cutting - 1.5 : 1

b) Canal in banking - 2:1 (Inner / Outer)

c) Canal in rock - 0.5 : 1 / 0.25 : 1 (According to classification of Rocks)

f) **Free Board : (vide IS 7112 - 1973 and IS 10430 - 2000)**

For unlined canal	: > 10 cumes	- 0.75 m.
	< 10 cumecs	- 0.50
For lined canal	: > 10 cumecs	- 0.75 m + 0.15 m
	3 to 10 cumecs	- 0.60 + 0.15 m
	1 to 3 cumecs	- 0.50 + 0.15 m

g) **BERMS :**

1st berm - FSL + FB

2nd, 3rd Berm - a) around 5m height

b) upto 7.5 m height at the Ground Level.

Minimum Berm width - 3.0 m

h) **Dowels : on both the banks (vide IS 7112 - 1973 and IS 10430 - 2000).**

i) **Bank Top widths : (vide IS 10430 - 2000)**

Unlined and Lined canals :

Discharge (m3/sec)	Minimum Bank Top Width (m) Inspection bank / wider bank (including dowel)	Non Inspection bank / other bank
0.15 to 3.0	4.0	2.5
3.0 to 10.0	4.0+dowel	2.5
10.0 to 30.0	5.0+dowel	4
30.0 and above	6.0+dowel	5

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3. **Radii of curvature : IS 5968 - 1987**

RADII OF CURVES FOR CANALS

(Clause 6.4)

Unlined Canals		Lined Canals	
Discharge (1) m^3/s	Radius, Min (2) m	Discharge (3) m^3/s	Radius, Min (4) m
80 and above	1500	280 and above	900
Less than 80 to 30	1000	Less than 280 to 200	750
Less than 30 to 15	600	Less than 200 to 200	600
Less than 15 to 3	300	Less than 140 to 200	450
Less than 3 to 0.3	150	Less than 70 to 200	300
Less than 0.3	90	Less than 40 to 200	200
		Less than 10 to 200	150
		Less than 3 to 200	100
		Less than 0.3	50

NOTE : 1. The above radii are not applicable to unlined canals located in hilly re.. ,es and highly permeable soils.

NOTE : 1. On lined canals where the above radii may not be provided proper super-elevation shall be provided.

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Chapter - 6
Guide Lines for Pump Selection for Lift Irrigation Schemes

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PLANNING & DESIGN OF MAJOR LIFT IRRIGATION SCHEMES

1. Introduction

Rain is decentralized and so is the demand, but the supply has not been decentralized resulting in under development and drought conditions in some pockets. The prevailing situation is causing widening of socio-economic conditions between regions causing imbalance in development.

In the absence of dams and barrages due to their inherent problems such as submersion, R &R, environmental, inter state problems etc., Lift Irrigation Schemes (LIS) gained greater significance in the changed scenario. Further, there are regions situated at higher altitudes to which gravity is not possible or site conditions at source do not permit dam / barrage. Though LI Schemes are costly w.r.t. dams / barrages considering life period of project, the present circumstances demand more & more LI Schemes as provision of conventional irrigation structures is exhausted.

Though concept of LIS is centuries old, with the magnitude of head, discharge and capacity of pumps, the design of Major LI Scheme has become science with advanced technology in pumps and pipes. It is noticed that there are guidelines & codes for design of pumps and pipes but not much attention is given on planning & design of major LIS. The LIS need due attention on optimization w.r.t. pump capacity, pressure mains design & laying along with Operation & Maintenance.

2. Objectives of LI Schemes

- Diversion of flood water to upland areas which are starving for water
- Supplying water to needy regions located far away from source
- Feeding tanks for future needs
- Effective usage of water stored in reservoirs
- Optimum utilization of water by supplying designed quantity
- Interlinking of rivers
- Transfer of surplus water from reservoirs to the required regions.

3. Planning & Design of Major LI Schemes

It comprises finalisation of various aspects at different stages as given below :

I. Hydrology

II. Alignment

III. Hydraulic Particulars

IV. Pumps – type, number & capacity
V. Pressure mains / Water conductor system
VI. Surge protection system
VII. Design of Pump House
VIII. Intake Sump / Surge pool / Fore bay
IX. Delivery Cistern / Out fall structure
X. SCADA – Supervisory (Sequential) Control And Data Acquisition
XI. Canal networking system

I. *Hydrology*

The design discharge for operation shall be arrived at based on crop water requirement, seepage & evaporation losses and drinking water requirement. Operation period of the pumps is also one of the governing factor in arriving design discharge of LIS. For major LIS, the operation period shall be 24 hours as they will be provided with dedicated power lines. However for minor LIS, the design discharge may be for 16 hrs or 20 hrs operation period depending upon the power availability.

The pumping hours for major LI Schemes shall be carefully decided as it has the bearing on the pump discharge and compounding impact on cost of the scheme by increase in pump capacity, pressure main dia with increased surge pressure and power consumption.

Fixing of low water level (LWL i.e., water level below which pumping is not required), is also an important factor for optimization as well as efficient functioning of LI Scheme. Hence it is desirable to keep LWL :

- Above bed level of source to avoid siltation in approach canal.
- Above MDDL of reservoirs at river intake to avoid unnecessary extra pumping head & pump capacity, other wise it encroaches into dead storage of reservoir which results in increased duration for its filling causing delay in supply to the already committed ayacut
- Whenever pump house is far away from the source, LWL in the sump shall be taken after deducting conveyance losses in approach canal / tunnel from MDDL of source
- For optimization of scheme, it is desirable to fix duty point of pump correspond to level 2.0m (or average water level for pumping from tank) above LWL, however pump operation zone shall correspond to LWL & FRL.

II. Alignment

The alignment finalisation consists of :

- Fixing of Pump house location in the foreshore of river / reservoir
- Approach and gravity canal lengths
- Length of Pressure mains
- Utilization of tanks enroute the alignment
- Number of Lifts / Pump houses

<u>Fixing of Pump house location in the foreshore of river / reservoir</u>

The pump house location shall be located in such a way that it needs smaller length of approach canal and smaller length of approach bridge from TBL to the pump house as former needs periodical maintenance and the later has bearing on the cost.

<u>Approach canal and Gravity Canal</u>

Whenever the source is river, it is desirable to design the approach canal capacity 50% more than design requirement to absorb siltation etc,. Off take point of the approach canal shall not be silt accumulation region since approach canal acts as gateway of the scheme.

As much as possible, possibility of greater length of gravity canals has to be explored to achieve considerable economy. The gravity canals should be lined otherwise the design discharge will not be realized at the terminal points.

<u>Length of Pressure mains</u>

It is desirable that shorter length of pipe line shall be provided as not only it is costly but also increases pump capacity & surge protection devices and has compounding impact on the scheme cost.

<u>Utilization of tanks / Balancing Reservoirs (BR) enroute the alignment</u>

Whenever any tank is present enroute the alignment or possibility of formation, it has to be examined as it has the following advantages :

- Design discharge of pumps can be reduced by preserving water in tanks to meet the peak requirement of the crop period, which reduces the pump capacity, pipe dia and canal sizes
- Usually LI Schemes are adopted to lift flood waters which can be stored in the balancing reservoirs for future needs to suit the cropping pattern

- LIS with multiple stages lifts need proper synchronization of all and failure of any single lift leads to grinding halt of the system. Hence with the presence of Balancing Reservoir, the above problem minimizes.

Number of Lifts / Pump houses

The number of lifts / pump houses is dependent on the following parameters :

- Length of the alignment
- Total Pumping head required
- Presence of ayacut enroute the alignment
- Capacity and type of proposed pumps

III. Hydraulic Particulars

After freezing of the alignment with pump house locations, the length of canals, pressure mains are to be calculated. Hydraulic Particulars of the scheme shall be finalized w.r.t LWL and FRL of proposed sumps along with canal HPs and pipe alignment. The discharge at pump houses shall be worked out duly considering the water requirement at various locations of the alignment.

IV. Pumps – Number, Capacity and Type

Pumps act as heart of LI Scheme and play important role in the performance as well as efficiency of the LIS. A designer should have a comprehensive knowledge on availability of various types of pumps and their applications along with their limitations.

Any wrong judgement in selection of pumps may lead to procurement of unsuitable pump and the scheme always may face threat of repairs & maintenance along with non-functionality to the design requirement of the scheme. Higher capacity increases unnecessarily the capital cost as well as power consumption and on other hand lower capacity will not deliver design discharge.

Number of Pumps

The capacity and type of pumps not only decides number of pumps to be provided but also the number of pump houses to be adopted. It is desirable to provide minimum of 4 pumps to facilitate operational flexibility, since in major LIS, stand by pumps are not being provided. Single pump failure for schemes with less than 4 pumps reduces more than 1/3rd design discharge thereby not serving fully the purpose of the scheme. In major LIS, it is not desirable to have higher degree of risk. However, number of pumps more than 4 may be provided considering cost economics with larger pump house since it is more advantageous in view of the lower degree of risk and better operation flexibility.

Capacity of Pumps

Pump capacity can be calculated by the formula :

Pump capacity in KW = $9.81\,Q\,H / \eta$

Pump capacity in HP = $9.81\,Q\,H / 0.746\,\eta$

Where Q = Discharge in cumecs H = Hst + Hf + Hb = Total pumping head in m

And η = Efficiency of pump

Total pumping head (H) can be arrived at on summation of static head, frictional losses in pressure mains and exit & entry losses. The frictional losses in the pressure main may be calculated using Hazen-William's formula :

$$Hf = L\left(1.1778\,V / C\,R^{0.63} \right)^{1.852}$$

Where V = Velocity in pipe (m/s) R = Hydraulic Radius in m = D/4

C = Hazen Williams Coeff = 130 for PSC pipes

= 130 + 0.17 d (dia in inch) for MS pipes

It is desirable to limit the C value to 130 only considering surface deterioration.

Pumps are to be designed for operation range w.r.t. LWL as well as FRL and the same shall be verified by studying the family curves and pump characteristics of the type of pump offered by the manufacturer. It is always preferable to keep the duty point of pump in between the above range.

Motor Capacity

Motor capacity shall be 10% to 20% more than pump capacity. LIS with small head fluctuations may require motor capacity of 10% to 15% more than pump capacity, whereas motor capacity in LIS with considerable head fluctuations may need 20% more than pump capacity.

Type of Pumps

Mainly four types of pumps are adopted in the major LIS and they are :

1. Horizontal Centrifugal Pumps - Applicable for medium heads and discharges. It has the limitation of suction lift and hence may be better suited for LI schemes on canals or tanks with total suction lift less than 6.0m.
2. Vertical Turbine Pumps - Applicable for schemes with high heads and discharges. Best suited for the schemes where the suction lift is more than 6.0m and more applicable to schemes on rivers.

3. Concrete / Metallic Volute Pumps - Applicable for schemes with high heads and huge discharges.

4. Francis turbine Pumps - Applicable for very high heads and very huge discharges.

V. *Pressure mains / Water conductor system – type of material, dia & no. of rows*

Pressure mains function as nerves of LI scheme and they consume lions share of the project cost whenever pipe lengths are in Km. Type and dia of pipe lines shall be judiciously adopted whenever lengthy pipes are proposed as length of pipe has direct bearing on pumping head thereby on pump capacity & surge protection system.

Generally, MS pipes and PSC pipes are under more usage in irrigation schemes. But GRP pipes also entered into the market recently performance of which is yet to be established. While designing the pipe dia, it is desirable to limit the velocity in MS pipes to 2.0 m/s and 1.5 m/s in PSC pipes. Velocity more than 2.0 m/s in MS pipes may be considered for the schemes with shorter length of pipes duly examining the impact of pump capacity.

The MS pipe thickness may be calculated based on :

- *Deflection Criteria*
- *Stress Criteria* - Compressive Stress & Tensile Stress
- *Buckling*

Minimum thickness of MS pipe may be as per recommendations given in IS : 1916. However, as a thumb rule, the D/t ratio may be provided 185 for pipes with shorter length & medium heads and D/t ratio upto 150 for high heads with lengthy pipes, subject to satisfying the surge conditions.

VI. *Surge protection system*

Whenever power failure occurs, rapid changes in velocity and associated change in pressure results in the pipe line causing surge pressure. Power failure leads to movement of upsurge and down surge waves along the rising main and the waves travel with high speed developing low & high pressures all along the pipe line.

Down Surge - Related to pressure drop or minimum pressure. Pressure drop immediately after power failure at peak locations causes negative pressure, which may even go down to vapour pressure.

Up Surge - Related to pressure rise or maximum pressure. When separated water column rejoins, sudden pressure rise occurs

Surge analysis is a very complicated phenomenon and needs thorough analysis of the pipe line profile w.r.t surge heads to assess type and number of surge protection devices at appropriate locations. Due attention shall be given to the surge analysis of pipe lines for schemes with high heads and lengthy pipes.

The surge generated can be controlled by providing surge protection devices.

Varrious Surge protection devices (in ascending order of probable cost) :

- Air Valves / Air Cushion Valve
- Stand Pipe
- Surge relief Valves
- Zero velocity valves
- One Way Surge Tanks
- Air Vessels

VII. *Design of Pump House*

The pump house arrangement is not same for all the type of pumps. They can be classified as :

❶ Wet Pit Pump House - The pump / impeller will be submerged in the water. Substructure will be with water for full area. Ex : Pump house with Vertical turbine pumps.
The dimensions of VT pump house / sump can be calculated based on the guide lines given in BHRA / HIS / IS : 15310.

❷ Dry Pit Pump House - Access to all the components including pumps will be there. Substructure will be without water and in dry condition because of which maintenance is easy. Ex : Francis turbines and Volute Pumps

<u>Design of Stoplog :</u> The stoplog gates for VT pump house may be designed for LWL operation only, whereas stoplogs for dry pit pump houses shall be designed for FRL condition to facilitate maintenance of pumps above LWL also, as it cannot be waited for maintenance till the water in the source falls to LWL.

<u>Motor Floor Level :</u> Multiple lifts with lengthy approach canal preceding the pump house in the absence of Balancing Reservoir need proper drainage system / escape regulator to avoid inundation of successive pumping station during power failure. The motor floor level shall be kept above the possible inundation level.

VIII. Intake Sump / Surge pool / Fore bay

The objective of sump and approaches is to provide storage and good flow conditions to pumps. If the design is with poor geometric features, undesirable flow conditions develop in the sump which reduces the pump efficiency.

To develop uniform, steady and non-turbulent flow conditions in the sump, it is recommended to allow maximum velocity of 1.2 m/s at the entry of forebay and 0.30 m/s near the pumps. To achieve the above requirement, the forebay may be tapered with limiting enlargement angle in plan to 20 degrees and bed slope of forebay in elevation from entry limiting to 10 degrees. However, it is desirable to provide 15 degrees in plan and bed slope of 8 degrees.

It is desirable to have the sides of forebay on water side with vertical face only (atleast upto LWL) as slopes cause off sets near pump house resulting whirling effect generating vortices. Further, near the pump, jump formation shall be strictly avoided as it creates turbulence near pumps. Further, proposals of intake sump for major LIS shall be ascertained by physical sump model studies and satisfied for fine tuning of the flow conditions, before commencement of execution.

Intake sumps in the river foreshore shall be provided with controlling arrangement at entry of forebay also to facilitate maintenance of the sump, in addition to the gates provision in front of the pumps.

IX. Delivery Cistern / Out fall structure

Delivery cistern will be provided to dissipate the energy of water falling freely from the pressure mains and delivers into the canals. To have better energy dissipating arrangement, the bed level of the leading canal should always be kept above the bed level of the cistern. Measuring gauges may be provided in the cistern to assess the discharge and losses in the conveyance if any.

X. SCADA – Supervisory Control And Data Acquisition

Whenever Multiple pumping stations are involved in a LI Scheme, it needs proper monitoring and vigilance for better synchronization, for which SCADA installation is mandatory, which collects and detects data such as :

- Non-functioning of pump / pumps in any of the pumping stations
- Non performance of any of the surge protection devices such as air vessels / One way surge tanks (OWST) etc,.
- Records data during operation of the scheme
- Monitoring the inflow and outflow discharges of pumping station as well as pumps discharges
- Power requirements

SCADA will be controlled at one station and the total alignment can be monitored with it. The origin of failure of any component of the system enroute the alignment can be detected using SCADA, with the help of which operation of other pumping stations can be controlled.

4. Important aspects in design of major LI schemes

A) Influence of Velocity and number of rows on LI Scheme

<u>Influence of Velocity in Pressure main on Pump</u>

For every 0.50 m/s rise in Velocity of pipe, frictional loss rises by 75% to 100% with reduction of dia by 11% to 13% only. Smaller dia is economical during initial stage of construction but power consumption will be high . Higher dia needs less power but with high initial cost.

Hence, it is desirable to allow higher velocities in shorter length of pipes and lower velocities in lengthy pipes (particularly when the length of pipe is in KM) owing to the recurring power consumption annually.

<u>Advantages of minimum number of rows for the pipes</u>

More number of pipes leads more frictional losses as well as enhanced pumping heads / pumping capacities and more quantity of steel. More pipes with smaller dia causes more frictional losses and initial cost as well as recurring power cost over lesser no. of pipes with bigger dia with same velocity. Further, more number of rows need more land acquisition and CM & CD works.

Hence, it is desirable to provide bigger dia with less number of rows of pipes, particularly for the schemes with lengthy pressure mains.

B) Precautions to be taken in laying and design of Pressure mains

Pipe lines in Soft Soils

- Care shall be taken in designing and laying of pipes in soft soils / BC soils etc., Either pipe shall be designed for the soil condition or the refilling has to be done with selected soils as the soil modulus is also one of the parameter in design pipe thickness.
- As the soil modulus is also one of the major property influencing the pipe thickness, it is mandatory to have the refilled soil gets compacted to achieve minimum of 90% Proctor's density.
- Water logged areas causes settlement of pipes or uplifting of pipes.

Hence the above field conditions shall be conveyed to the designer wherever the pipes pass through such areas.

Clear distance between Pipes

Clear distance between pipes shall be minimum of 3.0m for the schemes with pipes with more than two rows for bigger dia pipes for the following reasons :

- When multiple number of rows of pipes are laid and some of the pipes are only in operation, then pipes without water may create instability among the combined trench or when pipes are closely placed
- There will be scour sluices / washouts with projections to flushout the water in pipes and which get overlapped whenever pipes are closely placed.
- It is desirable to have independent thrust blocks to avoid problems during O & M in the vicinity blocks particularly for lengthy pipes.
- If adequate clearance is not provided in water logged areas, whenever any one of the pipe is empty, imbalance condition develops which results in settlement or uplifting of pipe
- Handling of heavy / bigger dia pipes need crane for erection & maintenance which needs 3.0 m (min) clearance in between pipes
- Whenever next pipe is to be laid after first was already in operation, it needs excavation and disturbs existing pipe trench. Thus weakening the degree of compaction made to first one and resulting imbalance of earth pressure on existing pipe. If sufficient gap is provided, the effect can be minimized.

Connection of pump delivery pipe and pressure mains

- When multiple number of rows of delivery pipes are required to be connected to a pressure main, a manifold (cylindrical or WYE type) is required.
- Whenever stand by pump is provided, cylindrical manifold may be mandatory (twice the dia but not less than equivalent dia of pressure mains) as the rotation of stand by pump among the wye junctions is not possible to satisfy equal discharge in pressure mains.

5. Conclusions

As major L.I. schemes are going to play major role in coming days, due attention shall be given to the planning & design for better performance & efficiency of schemes.

The alignment shall be so chosen comprising shorter length of approach channel and the shorter length of pressure mains. As far as possible, greater length of gravity canal may be provided for economy in major LIS with lengthy alignment.

Pumps function as heart of LIS and hence attention shall be given in fixing the duty point of the pump. For optimization of the scheme, duty point shall be w.r.t water level above the LWL. Major LIS shall be designed for 24 hours pumping. Pumping discharge shall be designed for mean average of crop water requirement wherever intermediate balancing reservoirs are present with pumping stations. Importance shall be given in planning of proposed sump dimensions and arrangement finalization as any undesirable flow condition leads to damage of pump impeller resulting in reduction of efficiency as well as pump break down.

As the pressure mains act as nerves of LIS, care shall be taken for pipes when they are to be laid in BC soils, water logged area and at crossing of vagus/drains. Low velocity in the pipe would be economical for the schemes with very lengthy pressure mains, however higher velocity may be permitted for the schemes with shorter length. Larger dia with less number of rows may be economical w.r.t. installation cost as well running cost. Adequate clearance shall be maintained between pipes for stability as well as maintenance purpose. Due attention shall be given to surge parameters which are vital aspects for proper functioning of the pipe line.

LI Scheme comprising multiple pump houses shall be provided with SCADA for observing / monitoring system behavior as well as to take precautions against surge.

USAGE OF MILD STEEL PIPES FOR LIFT IRRIGATION PROJECTS

GUIDELINES FOR FABRICATION, LINING, COATTING, LOADING, TRANS-PORTATION, UNLOADING, TRENCHING, LAYING, ALIGNMENT, FULL WELDING, TESTING AND BACK FILLING

PREAMBLE:

1. The state governemnt of Andhra Pradesh has taken up large number of lift irrigation schemes to provide irrigation to drought prone and upland areas in order to give boost to agriculture sector and rural economy. In many of these schemes mild steel pipes with inner lining and outer coating are being utilized due to higher lift and other technical paramenters. These are capital-intensive schemes due to high cost of steel pipes and need very stringent quality control measures to be enforced during project implementation stage. The quality aspect has to be ensured at all stages starting from procurement of raw meterials mainly mild steel / steel plates, cement, welding electrodes and during processes of manufacture, transport, handling, laying and testing at various stages. This needs to be enforced to minimize subsequent maintenace problems, related to quality of pipes and their laying, over decades long life span of these projects.

2. There are various I.S. Codes prescribing the quality standards for aforementioned activities. However, during inspections of Guthpa L.I. Scheme and GLIS in Nizamabad and Warangal districts respectively, it has been observed that many of these codal provisions are not being adhered to with a view to educate and refresh the knowledge of the departmental engineers these guidelines consolidating various I.S. Code provisions are being issued for strict compliance to ensure quality in these projects.

3. These guidelines are only illustrative and engineers shall read all the relevant IS codes for ensuring the quality at their level of functioning and supervision.

I. FABRICATION OF PIPES : I.S. 800:1962

a) RAW MATERIALS :

i) Steel :

- The pipes shall be manufactured from steel plates conforming either to I.S. 226:1975 or to I.S. 2062 : 1984.
- Manuracturer shall test steel plates for quality. Test certificates shall be obtained from the manufacturer / steel producers along with the steel procurement.
- Test piece from steel plates shall be sent for testing for mechanical properties and chemical composition to engineering / metallurgical laboratory (ies)

ii) Welding Electrodes :

- Welding electrodes used for welding steel plates shall conform to I.S. 814 (Part 2) IS 814 (Part2) - 1974.
- For welding, Bureau of Indian standards approved welding electrodes of very reputed companies like, Advani-Orlikan, D&H, ESAB, IOL, Modi, L&T and Rock weld may be used.

iii) Portland cement : The cement used in the lining and coating of steel pipes shall conform to I.S. 8112-1989 (43 Grade).

iv) Sand : Sand shall consist of inert materials having hard, strong, durable uncoated grains conforming to the requirement of I.S. 2116.

v) Reinforcement : Reinforcement shall be with 50 X 100 mm. welded wire fabric. The wire shall conform to the requirements of I.S. 1566

b) PIPE MANUFACTURING PROCESS :

i) Cutting of steel plates :

- **Trimming:** If the steel plates received from Manufacturer are untrimmed, the untrimmed edge shall be cut. The steel plate cut shall be a true rectangle.
- Longirudinal Joint : The length of plate shall be selected in such a manner, that only one longitudinal joint is formed.
- Edge preparation : Straight edge preparation shall be done for steel plate for longitudinal and circumferential butt joints to be welded at the factory by submerged arc welding. Bevel (single 'V'-grove) edge preparation shall be done for circumferential butt joints to be welded at site manually, after laying of pipes.

ii) Welding of pipes :

- All welds shall be made down-hand by manual welding automatic shielded arc welding process.
- Prior to welding, the plates shall be fitted closely and during welding they shall be held firmly.
- Welding shall be done to ensure thorough fusion and complete penetration. The welding shall be as per I.S. 816 : 1969.

C) **TESTING OF BUTT WELDING :**

a) **Radiography & Ultrasound :**

- The butt welds of longitudinal and circumferential joints at the factory done by submerged arc welding shall be tested at the rate of 5% by radiography and balance 95% by ultrasound testing or as per terms of agreement.
- The circumferential butt-welding done manually in the trench at site shall be tested at the rate of 100% by ultrasound testing.
- The radiography films and ultrasound print records of quality check at site of manufacture shall be presented by contractor to the site in-charge engineer and / Quality Control Staff before dispatch of these to the field.

b) **Pressure testing of pipe (Hydro static pressure test) :**

- Each pipe shall be hydraulically tested at 2 times the working pressure at the manufacturing site before the pipe is lined and coated. The pipe shall be kept under pressure by pumping water for a period of not less than one minute. While under pressure, the pipe shall be moderately hammered with 1 kg., hammer throughout its length. The pipe shall withstand the pressure test without showing any leakage.

d) **IDENTIFICATION NUMBER OF PIPE AND MAINTENANCE OF RECORDS :**

- Each pipe shall be given an identification number and painted on each pipe. It shall be recorded for all quality tests like radiography / ultrasound, pressure testing etc and records shall be maintained.

II. LINING AND COATING OF PIPE - I.S. 3589:2001

i) **Lining :** For lining of pipe, the cement mortar shall be composed of cement, sand and water well mixed and of proper consistency to obtain a dense, homogenous lining that will adhere firmly to the pipe surface. It's prportion shall be 2 parts sand to 1 part cement by weight. Lining shall be done by spinning machine specifically designed and built for the purpose of rotating the pipe section and centrifugally applying cement mortar lining to the interior of steel pipe. Lining shall always be done at the manufacturing site.

ii) **Coating :** Cement mortar shall consist of not more than 3 parts sand to 1 part cement by weight. The water in the mixture shall be carefully controlled so that the mortar will not run, sag or segregate. Reinforcement shall be with 50 X 100 mm welded wire fabric. The outside pipe shall receive a reinforced cement mortar coating applied by mechanical placement or pneumatic placement. The coating shall always be done at the manufacturing site.

iii) **Curing :** Immediately after lining and coating pipe sections may be moved to curing area and water sprinkled by sprinklers. The curring should be done till lining and coating attains required strength.

III. LOADING,TRANSPORTATION, UN -LOADING AND STACKING OF PIPES -I.A.5822 : 1994

a) **Loading and unloading:**

- The Pipes shall be handled in such a manner as not to distort their circularity or cause any damage to their outer coating.
- Slings of canvas or equally non - abrasive material of suitable width or special attachment shaped to fit the pipe ends shall be used to lift and lower coated pipes, so as to eliminate the risk of damage to the coating.

b) **Transportation and Stacking :**

- The Pipe shall be transported to the site of laying and stacked along the route on timber skids. Padding shall be provided between coated pipes and timber skids to avoid damage to the coating.
- Suitable gaps in the pipes stacked should be left at intervals to permit access from one side to the other.

IV. TRENCHING - I.A. 5822; 1994

- For pipes larger than 1200 mm dia, in earth and murrum the curvature of the bottom of the trench should match the curvature of the pipe as far as possible, subtending an angle of about 12 degrees at the centre of the pipe.
- Where rock or boulders are encountered, the trench shall be trimmed to a depth of at least 100 mm below the level at which the bottom of the barrel of the pipe is to be laid and filled to a like depth with lean cement concrete or with non-compressible material like sand of adequate depth to give the curved seating.
- There shall be adequate space at bottom of the pipe for entry and sufficient movement of person doing welding task.

V. LAYING, ALLIGNING AND FULL WELDING OF PIPES - I.S. 5822 :1994

a) **Laying :**

- Slings of canvas or equally non-abrasive material of suitable width or special a ttachment shaped to fit the pipe ends shall be used to lower pipes into the excavated trench.

b) **Aligning of pipes :**

- While assembling, the pipes shall be brought close to have a uniform gap not exceeding 3 mm. The pipe faces shall be matched perfectly without any offset.
- The spiders from inside and tightening rings from outside or other suitable equipment should be used to keep the two faces in shape and position till at least one run of welding is carried out.

c) Full welding :

- The pipe faces shall first be tack-welded alternatively at one or more diametrically opposite paris of points. After completing out in welding, full welding shall be carried out in suitable runs following sequence of welding positions of segments diametrically opposite.
- For welding Bureau of Indian Standards approved welding electrodes (IS 814 (Part2): 1974 of very requted companies like Advani - Orlikan, D &H, ESAb< Iol, Modi, L &T and Rock weld may be used.

VI. Back Filling - I.S 5822 : 1994

- While back filling the trench under the weld Joint of two Pipes shall be filled first and consolidated.
- Back filling should closely follow the welding of Joints of the pipe, so that the protective coating should not be subsequently damaged.
- Material harmful to the pipeline shall not be used for back filling Refilling shall be done in layers not exceeding 300 mm.
- Each layer shall be consolidated by watering and ramming, care being taken to prevent damage to the pipeline.
- The filling on the two sides of the pipeline should be carried out simul taneously.

VII. Testing of pipe Line - I.S. 5822 : 1994

- The welded pipeline shall be tested both for its strength and leakage.
- Each section of the pipe shall be slowly filled with clean water and all air shall be expelled from the pipeline and specified test pressure shall be applied by means of pump connected to the pipeline. Under the test pressure, no leak or sweating shalll be visible at all section pipes and welded joints.
- Any defective pipes discovered in consequence of this pressure test shall be removed and replaced by sound material and the test shalll be repeated untill satisfactory results are obtained.

VIII. PIPES WHICH SHALL BE CONSIDERED SUB-STANDARD ON VISUAL INSPECTION AND REJECTED.

- Lamination of MS Plate.
- Inner lining having cracks exceeding 1.5 mm.
- Pipes having two longitudinal joints.

PUMPS

GUIDELINES FOR PUMP SELECTION FOR LIFT IRRIGATION SCHEMES

Introduction

<u>The following aspects are covered</u>

Planning of an L.I. Scheme consists of the following steps after identifying the ayacut that is to be provided with irrigation facility under the Scheme;

1. Selection of suitable site at source of water for locating the head work, Intake structure and delivery Cistern.
2. Assessment of water requirement for the proposed crops in the identified ayacut.
3. Determination of type of pumps and estimation of total HP of motors to be installed.
4. Fixing of dimensions of Civil works like; Sumpwell, Pump house, Cistern etc.,
5. Selection of diameter and the type of pipe required for pressure main.

Irrigation is practised in India from ancient times and experience and expertise is available on all items of works in irrigation system except the Pump Selection. Though certain good Books are available for Pump Selection, all the required information is not available in one compact Book to serve the needs. This booklet is prepared keeping that in view and after going through the Literature available on this subject and after collecting the required information from the different Books written by the renowned Authors. It deals in sufficient detail how to proceed with the selection of the Suction and Delivery Pipes, type of Pump and Motor suitable to the scheme and also the dimensions of Civil works like; Sump well, Pump House, Cistern etc.,

In addition to making use of the works done by different authorities on this subject, the following standards are referred to for laying down the procedure of pump selection and hydraulic design of intake (sump).

1. Indian Standard Specifications.
2. Standards of American Hydraulic Institute for Horizontal Centrifugal pumps.
3. Standards of American Water Works Association (AWWA) for vertical turbine pumps.
4. Standards of British Hydraulic Research Association (BHRA) for Hydraulic design of Intake (Sump).

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Since specifications and standards used, are of different countries, the units used are also different viz., metric, f.p.s. and American units in the following pages and Annexures.

In Annexures I to XI the following are presented.

a. Guide lines of permissible velocities in suction pipe, delivery pipe, column assembly and pressure main.

b. Schematic diagrams for two typical layouts.

c. Drawing indicating the relative positions of pumps and motors.

d. Efficiency and cushion to be adopted.

e. Graph for reading permissible suction lifts.

f. Field information required for selection of pumps and motors.

1. The following Data should be invariably made available for finalising the pump selection for a lift irrigation scheme;

   a. Schematic Diagram of the LI scheme giving the layout, location of pump house, suction, delivery, Pressure Mains and Delivery Cisterns.

   b. Hydraulic particulars in the Proforma given in Annexure-VIII

   c. Longitudinal section (LS) of the Pressure main to indicate whether it is in cutting or banking and whether there is any sudden drop or rise in the pressure main. In such cases, bends are to be provided for which friction losses are to be computed and added.

   d. A plan showing the alignment of the pressure main for the same purpose i.e. for providing 90 ° or 120 ° bends where ever there is a change in bearing of the pressure main.

   Specimen, Schematic diagrams of LI schemes with Horizontal and V.T.Pumps are given in Annexure X, XI respectively.

   The pump selection is done broadly as per the guidelines given below:

2. If the total suction lift including losses is less than 7.50 Mts., Horizontal centrifugal Pumps are generally suitable. Hence, in such cases, pump selections are to be worked out for Horizontal shaft pumps initially.

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3. If the total suction lift including losses is more than 7.50 Metres, We must go in for V.T.Pumps only. Horizontal shaft pumps are not suitable. Hence, pump selection is to be worked out for V.T.Pumps.

However in certain cases where even though the total suction lift is less than 7.50 Metres, we may have to go in for V.T.Pumps because of high specific speed.

Exact suction lift is determined on the graphs of IS standards or standards of American Hydraulic Institute after working out the specific speed of the pump and reading it against the total head of the pump. Except in cases of very low powered pumps of the range upto 15 HP, suction lift will generally be less than 7.5 Mts.

4. Initially (2+1) Nos. i.e. 2 Nos. working and one stand bye is worked out. Sometimes with this arrangements, it may so happen that there is little margin between total suction lift and permissible suction lift as per the Graph (Annexure-VII). This situation arises when the discharge of each pump is high which, when combined with the other parameters like; total head and speed and gives a very high specific speed. To reduce the specific speed, we will have to first try to reduce the speed from 1460 rpm (nominal) to 960 rpm (nominal). Since total head of the scheme is constant, in order to bring down the specific speed to the manageable limits either discharge is to be reduced by increasing the No. of pumps or the speed of the prime mover may be reduced. Since specific speed varies directly with Q, reduction will not be so significant as it is incase of reducing speed of the motor, say from 1460 to 960 (nominal). This will however increase the size of pump and motor and also the cost of the equipment. Even then, if it is not suitable, then we will have to increase the number of pumps to reduce the discharge and thereby reduce the specific speed i.e. pump selection is to be tried with (3+1) Nos. or (4+1) Nos.

5. System Resistance losses: when 2 or more pumps are run in parallel to feed a common pressure main, there will be a certain amount of reduction in total discharge. This is called system resistance loss. To take care of this, each pump has to be designed for a slightly higher discharge after taking into account the percentage losses as given below:

The exact quantity of reduction in discharge can be computed by drawing the system-Head curve, since it is not possible before fixing the supplying agency and the make and model of pump, the percentages given below are applied which give a fairly correct assessment of the loss of discharge from each pump. However in a major scheme say for an ayacut of 10,000 acres or so, the tenderers may be asked to furnish the system-head curves of parallel operation, along with their offer.

a. When there is no pressure main .. Nil

b. For (1+1) system of pumps .. Nil

c. For (2+1) system of pumps .. 5%

d. for (3+1) system of pumps .. 7.5%

e. For (4+1) system of pumps .. 10%

Pump Selection Procedure for horizontal centrifugal pumps

Discharge through each pump is to be arrived after considering system resistance losses.

Suction side

Static suction lift= Pump axis - LWL in sump well

Pump axis level = Platform level of pump + 0.3m to 0.5m

The dia. of suction pipe is to be arrived for the discharge, keeping the velocity through the suction pipe within the permissible velocities given in enclosed Annexure-III.

a. Entrance Losses = $KV^2 / 2g$

Where, K is constant = 0.5

V = Velocity in suction pipe

g = Gravitational force i.e. 32.2 ft/sec^2

b. Frictional Losses in suction pipe & fitting.

Approx length of suction pipe = 25' to 30' approx. or the actual length.

Equivalent 90º bend = 1 No.

Equivalent length of 1 No. of Bell mouth or Foot valve with strainer.

Total equivalent length= 25' + Equivalent length of 90º bend + Bell mouth or Foot valve.

Frictional losses in suction pipe can be obtained from the William and Hazen Formula or IS Standards.

$$\text{Losses as per the Formula} = \frac{4.69}{D^{4.87}} \times (Q/C)^{1.85} \times L$$

Where, D = Dia of suction pipe in ft.

Q = Discharge through each suction pipe in Cusecs.

L = Equivalent length of suction pipe and fittings

C = 120 for MS. or ERW pipe (Old or New)

Total suction lift including losses = Static suction lift + Entrance losses + Frictional losses in Suction pipe & fittings ••A

Delivery side:

Static delivery head = Delivery level at Cistern - Pump axis level.

The dia. of delivery pipe is to be arrived for the discharge, keeping the velocity through the delivery pipe within the permissible limits given in enclosed Annexure - IV (A).

c. Frictional losses in delivery pipe and fittings:

Approx length of delivery pipe = 25' approx. or as per actuals

Equivalent length of Reflux valve = 1 No.

Equivalent length of Sluice Valve = 1 No.

Equivalent length of 90º Bends = 2 Nos.

Total equivalent length =

Loss of head in delivery pipes and fittings can be obtained from the Formula;

$$\text{Losses as per the Formula} = \frac{4.69}{D^{4.87}} \times (Q/C)^{1.85} \times L$$

Where D = Delivery pipe dia. in ft.

Q	=	Discharge through delivery pipe in Cusecs
L	=	equivalent length on delivery side
C	=	120 for MS. or ERW pipe (New or Old)

Loss of head in the Manifold being small, it is ignored.

d. Loss of head in Pressure Main:

The dia.of pressure main is to be arrived for the discharge, keeping in view the maximum permissible velocities given in enclosed Annexure - IV (C). To arrive at the most economical dia. of pressure main, alternates may be worked out changing the dia of pressure main, calculating HP, of each pump set and the cost, water rates etc.,

Equivalent length of pressure main = Length of pressure main in ft. + equivalent length of 3 Nos.of 90° bends (minimum)

Total length can be arrived;

The loss of head in pressure main can be obtained from the

$$\text{Losses as per the Formula} = \frac{4.69}{D^{4.87}} \times (Q/C)^{1.85} \times L$$

Where	D	=	Dia of pressure main in ft.
	Q	=	Discharge in cusecs
	C	=	110 for pressure main of cc pipes or 130 for PVC pipes
	L	=	Total length of pressure main in ft.

e. Exit losses = $V^2 / 2g$

Where, V= Velocity in pressure main and g = 32.2 ft/sec2.

Total delivery head including losses = Static delivery head + losses in delivery pipe and fittings + loss of head in pressure main + Exit losses .... (B)

Total pumping head of each pump = Total suction lift including losses + Total Delivery head including losses + ½ Dia. of pressure main in ft.

Care may be taken that discharge occurs through the exit in the atmospheric conditions i.e. the pipe should never come under submerged condition.

The above total pumping head is used for finding the Ns (specific speed) ....h

Add 10% on losses for aging of pipes

This total pumping head is adopted for the purpose of finding out HP of the pump...H

$$\text{Specific speed at 1460 RPM (nom) } N_s = \frac{3.65 \times 1460 \times \sqrt{Q}}{(h)^{3/4}}$$

Where	Q	= Discharge in Cum/Sec
	h	= Total head including losses, in Metres

Note: For a Double suction pump Q/2 may be taken instead of Q or specific speed may be divided by $\sqrt{2}$

Exact suitability of the pump can be obtained from the Graphs as per IS-5120/1977 i.e. as per enclosed Annexure -VII. Alternatively if the graphs of American Hydraulic Institute are referred to, the formula is

$$N_s = \frac{N \times \sqrt{Q}}{(h)^{3/4}}$$

Where	N	=	speed in rpm
	Q	=	Discharge in American gallons per minute
	h	=	Total head in ft.

The advantages are

i) Classification with respect to impeller design of the pump is done more precisely.

ii) Suction lift for a double suction pump can be read directly.

The suction lift line on the Graph for the duty conditions (i.e. discharge and head) should be more than the total suction lift including losses. There must be minimum difference of 0.50 Metres.

If there is no such safe margin, the number of pumps may be increased. Even then if it is not within the limits, V.T. pumps have to be necessarily proposed.

H.P. Calculations

$$HP = \frac{62.45 \times Q \times H}{550 \times n}$$

Where Q = Discharge of each pump in Cusecs

H = Total pumping head in feet.

n = Overall efficiency of the pump as given in the enclosed Annexure-VI.

Cushioning is to be added as per enclosed Annexure-VI.

The HP is to be rounded off to the nearest range of HP.

Sizes of Sump-well, Pump house

Based on the sizes of pump, motor and as per the dimensions given in the drawing (i.e. Annexure-I) the sizes of sumpwell, pump house can be arrived.

Hydraulic Design of Approach channel and Intake

For obtaining the maximum degree of steamline flow, flow to the intake or sump is trained right from the point of entry in to the channel. Ideal conditions for the most satisfactory working of the pump is when it runs in still water. Since it is not possible to allow this condition, certain standards based on the model studies are adopted.

i) Velocity in the approach channel should be restricted to 1 to 1½ ft/sec or 0.3 to 0.45 m/sec.

ii) Angle of approach should be kept within the range of 45° to 75° with reference to the chord on which the pumps are located i.e. on the axis normal to the flow of water in the intake. The angle of approach should vary directly with the quantity of flow.

iii) Portion of the intake or sump from the location of pump upto the length of longitudinal separators should have zero slope i.e. it should be perfectly levelled.

iv) To minimise the effect of interference when two adjacent pumps are working, separators of specified lengths may be provided and raised to the height of max. or atleast

normal level of water in the sump.

v) Backing wall may be provided.

vi) If trash rack is provided in a major LI scheme, it should be located at the entry of water into the separate compartments.

vii) Enough margin in-depth may be kept for providing minimum submergence which depends on the cavitation, characteristic of the pump and which is furnished by the manufacturers.

Net positive suction Head calculations

Available NPSH at LWL = Pr - P - A

Where Pr = Atmospheric pressure - As shown in IX A or IX C

P = Vapour pressure As shown in IX B

A = Total suction lift including losses.

Max. Water Level = Platform level - 1.00 Metre.

Available NPSH at Max. WL = Available NPSH at Lwl + (Max. W.L.- LWL)

The manufacturer of the pump should be asked to specify the NPSH required for the pump he is offering. There should be a clear margin of 0.6 metres between NPSH available at LWL and NPSH required. The NPSH available should be more by at least 0.6 metres than the NPSH required. When this is not the case, cavitation will occur during running of the pumps and so the particular pump is not suitable. We will have to go in only for such of the pumps whose NPSH required is less than NPSH available at LWL by the prescribed margin.

Selection of type of impeller

Whereas in centrifugal pumps, reading the graph itself will indicate the type of impeller which is required for the duty conditions as it is directly related to the suction lift, it will not be the case in vertical turbine pumps as there is no question of suction lift. Hence it is absolutely necessary in selecting vertical turbine pump to check the specific speed and the duty condition with reference to the configuration of the impeller design.

In American units, the specific speeds for different types of impellers are given in the following tables.

Type	Centrifugal Double-suction	Mixed Flow Double-suction	Mixed Flow propeller	Axial Flow propeller
N_s	1250	2200	6500	13500
gpm	2400	2400	2400	2400
Head (ft)	70	48	33	20
rpm	870	1160	1750	2600
D_2 (in)	19	12	10	7
D_1/D_2	0.5	0.7	0.9	1.0

These specific speeds may be compared with the total pumping head and it may be seen whether they are matching. If they donot, change the speed of the prime mover according to the requirements.

Model pump selection for horizontal centrifugal pumps

Hydraulic particulars

Discharge	..	10 Cusecs
Lwl in sump well	..	88.545 Metres
Bottom finished level or sill level of sumpwell	...	86.395 Metres
Platform level of pumps	..	+ 93.50 Meters
Pressure main dia.	..	To be arrived
Length of pressure main	..	149.00 Ft. or 45.50 Metres
No. of Rows of pressure main	..	One
Delivery level at cistern	..	+ 101.80 Metres
Static suction lift	..	93.50 + 0.30 - 88.54 =5.255 Mts. or 17.24 feet

Static delivery head 101.80-(93.5+0.30) = 8 Metres or 26.25 feet

Since the static suction lift is less than 7.50 Metres, the suitability of Horizontal shaft pumps is checked hereunder:

No. of pumps proposed	=	(2+1) = 3 Nos.
Discharge	=	10.00 Cusecs
System resistance losses for (2+1) system at 5%	=	0.50 Cusecs
		10.50 Cusecs

Discharge of each pump = 10.5 / 2 = 5.25 cusecs or 0.1487 Cum or 148.68 Lps or 2357 Usgpm or 1964 Igpm

Selection of suction pipe, assuming 14" dia. suction pipe

$$Vs = \frac{5.25}{(\pi/4) \times (14/12)^2} = 4.91 \text{ ft/ sec}$$

The velocity in 12" dia. suction pipe is 6.68 ft/sec. against the permissible limit of 4.5 ft/ sec. Hence, 12" dia. suction pipe is not suitable. The velocity in 14" dia. suction pipe is 4.91 ft/sec. which is within the permissible limit of 5.0 ft/sec. Hence, 14" dia. (350 mm dia.) suction pipe is recommended.

Entrance losses = $0.5 \times (4.91)^2 / (2 \times 32.2) = 0.19$ ft ...... (a)

Frictional losses in 14" dia. suction pipe & fittings;

Total length = 25+36+30 = 91 ft

Loss of head (as per Table) = $(7.2/100) \times 91 = 0.66$ ft. ... (b)

$$\text{or } \frac{4.69}{(14/12)^{4.87}} \times (5.25/120)^{1.85} \times 91 = 0.62 \text{ ft.}$$

Total suction lift including loses = 17.24 + 0.19 + 0.62 = 18.05 ft. or 5.50 Metre. ...(A)

Selection of Delivery Pipe, assuming 12" dia (300mm dia) delivery pipe

$$Vd = \frac{5.25}{\Pi/4 \times (12/12)^2} = 6.68 \text{ ft/sec}$$

The velocity in 10" dia delivery pipe is 9.62 ft/sec. against the permissible limit of 6.5ft/sec. Hence, 10" dia delivery pipe is not suitable. The velocity in 12" dia (300mm dia) delivery pipe is 6.68 ft/sec. which is within the permissible limit of 8.50 ft/sec. Hence, 12" dia (300 mm dia)delivery pipe is proposed.

Frictional losses in 12" dia delivery pipe & Fittings

= Total length = 25 + 44 + 6.5 + 2x30 = 135.5 ft

Loss of Head (as per Table) = 15.3 x 135.5 / 1000 = 2.07 ft................(c)

or $4.69/(12/12)^{4.87} \times (5.25/120)^{1.85} \times 135.5 = 1.95$ ft.

Loss of head in the manifold being small, it is ignored.

Loss of head in pressure main, assuming 24" dia (600mm)

$$Vpr = \frac{10}{\Pi/4 \times (24/12)^2} = 3.18 \text{ ft/ sec}$$

The velocity in 20" dia pressure main is 4.58 ft/sec. against the permissible limit of 4 ft/sec. Hence, 20" dia pressure main is not suitable. The velocity in 24" dia (600mm dia) pressure main is 3.18 ft/sec. which is within the permissible limit of 4 ft/sec. Hence,

24" dia (600 mm dia) pressure main is recommended.

Total length = 149 + 3x63 = 338 feet

Loss of Head (as per Table) = 1.93 x 338 / 1000 = 0.65 ft...............(d)

or $4.69/(12/12)^{4.87} \times (10/110)^{1.85} \times 338 = 0.63$ ft.

Exit losses = $3.18^2 / (2 \times 13.2) = 0.16$ ft. ....(e)

Total delivery head including losses = 26.25 + 2.07 + 0.65 + 0.16 = 29.13 ft or 8.88 metres. ....(B)

Total pumping head of each pump = 18.05 + 29.13 + 1.00 = 48.18 ft or 14.68m for Ns.

Add 10% for ageing on losses = 48.18

0.29

48.47 ft or 14.77 m per HP.

$$\text{Specific speed at 1460 rpm Ns} = \frac{3.65 \times 1460 \times (0.1487)^{0.5}}{(14.68)^{3/4}} = 274$$

As per IS-5120/1977, the suction lift limit for single stage, double suction, radial flow pumps is about 6.20 Metres against our requirement of 5.50 Metres. Hence, Horizontal Shaft, single stage, double suction, radial flow pumps running at 1460 rpm. are recommended.

<u>HP Calculations</u>

$$HP = \frac{62.45 \times 5.25 \times 48.47}{550 \times 0.75} = 38.52$$

Add 10% towards Cushioning	=	3.85
Next range is 50 HP		42.37

Hence, (2+1) Nos, 50 HP, 1460 rpm (nominal) motors are recommended.

NPSH Calculations

Available NPSH at LWL = 33.9 - 3.0 -18.05 = 12.85 ft. or 3.92 m

(As shown in Annexure IXa, IXb and IXc)

Max. Water level	=	93.5 - 1.00	=	92.5 m
LWL			=	88.545 m
Difference			=	**3.955m or 12.98 ft.**

Available NPSH at Max. W.L. = 12.85 + 12.98 = 25.83 or 7.87 m.

Pump Selection procedure for Vertical Turbine Pumpsets

If the static suction lift is more than 7.50 Metres, V.T. Pumpsets are to be proposed.

Discharge of each pump is to be arrived after considering system resistance losses.

1. Selection of delivery pipe & Column assembly:

   The delivery pipe dia. can be arrived from V = Q/A and keeping in view the permissible velocities for delivery pipes and column pipe given in the enclosed Annexure : IV (B).

Losses in delivery pipe & fittings.

Approx. length	=	25
Reflux valve	=	1 No.
90° Bends	=	3 Nos.
Sluice valve	=	1 No.
Total equivalent length	=	

The losses can be arrived either by Table or from Formula as explained in case of Horizontal pump selection.

II. Selection of Column assembly, Line shaft:

$$\text{Approximate HP} = \frac{(62.45 \times \text{discharge} \times \text{Static pumping head})}{550 \times 0.75} \times (130/100)$$

$$\text{Dia. of line shaft in inches} = \sqrt[3]{\frac{\text{HP} \times 321000}{7000 \times 1460}}$$

where 1460 is speed of the Prime mover.

a. The losses in column assembly can be arrived from the Tables of Johnston Pump Company

b. Entry losses $= 0.5\, V^2 / 2g$

c. Losses in Strainer $= 0.05x\, V^2 / 2g$

III. Loss of head in pressure main may be worked out as explained in the case of Horizontal pump selection.

IV. Exit losses $= V^2 / 2g$

Static pumping head = Delivery level at cistern - LWL in Jack well

h = Total pumping head of each pump =

Static pumping head + losses in delivery pipe + losses in column assembly + Entry losses + losses in Strainer + loss of head in pressure main + Exit losses + ½ dia.. of Pressure main.

This h is for Ns purpose.

Total pumping head , H = h + 10% on losses towards ageing of pipe

H for HP purpose

$$\text{Ns Specific Speed @ 1460 rpm (Nom.)} = \frac{3.65 \times 1460 \sqrt{Q}}{(h)^{3/4}}$$

HP calculation = 62.45 x Q x H / (550 x n)

Add cushioning as per Annexure - VI

Minimum Clearance in between the Main Girders= = 2d + 150 mm (Approx)

Where d= dia. of column assembly in mm.

The inner dia. of Jack well-cum-pump house, length of chord etc., are to be calculated as per the standards, details shown in the enclosed Annexure - II.

Model pump selection for V.T. pumpsets

Hydraulic Particulars:

Discharge	..	15.20 cusecs.
Lwl in Jack well	..	+ 145.285 Metres.
MFL in River	..	+ 156.285 Metres.
Bottom finished level or		
Sill level of Jack well cum pump house	..	+143.335 Metres
Plat form level of pumps	..	+157.285 Metres.
Pressure main dia.	..	To be arrived.
Length of pressure main	..	490 Metres or 1608 ft.
No. of Rows of pressure main	..	one
Delivery level at cistern	..	+158.735 Metres.
Static suction lift	..	157.285-145.285
		12 Metre or 39.37 ft.

Since the static suction lift is more than 7.50 Metres, Horizontal centrifugal pumpsets are not suitable. Hence, V.T. pumps are proposed.

No. of pumpsets	..	(2+1) = 3 Nos.
Discharge of the scheme	..	15.20 cusecs.
System resistance losses at 5% for (2+1) system.	..	0.76 cusecs
TOTAL	..	15.96 cusecs
Discharge of each pump	..	15.96/2 = 7.98 cusecs

Or 0.2260 cum/sec. or 226 LPS or 3582 USGPM or 2985 IGPM.

Static pumping Head = 158.735 - 145.285 = 13.45 Metres or 44.13 ft..... I

Selection of delivery pipe, and column pipe assuming 12" delivery pipe.

$$Vd = \frac{7.98}{\Pi/4 \times (12/12)^2} = 10.16 \text{ ft/sec}$$

The velocity in 10" dia delivery pipe is 14.63 ft/sec. against the permissible limit of 11 ft/sec. Hence, 10" dia delivery pipe is not suitable. The velocity in 12" dia delivery pipe is 10.16 ft/sec. which is within the permissible limit of 11.50 ft/sec. Hence, 12" dia delivery pipe and column pipe are recommended.

Losses in delivery pipe of 300 mm dia and Pipe fittings.

Equivalent length = 25 + 30 + 2 x 30 + 6.5 + 44 (for Tee)

= 165.5 ft.

Losses as per tables = (33.0 x 165.6 / 1000) = 5.46 ft..........................(a)

Or losses can be arrived from the formula also.

Selection of column assembly, line shaft,

Approx. HP = (62.45 x 7.98 x 44.13) / (550x0.75) x (130/100) = 69.31.

Next range is 75 HP.

$$\text{Dia of line shaft} = \sqrt[3]{\frac{75 \times 321000}{7000 \times 1460}} = 1.331"$$

Assuming 1460 rpm speed

Next range is 1½" to $1\frac{11}{16}$"

The losses for 3600 US GPM with 12" dia column assembly and with line shaft dia of 1½" to 1 11/16 dia is 4 feet for every 100 feet length.

Losses in column assembly, assuming length of column assembly as 40 feet.

	=	4x40 /100 = 1.6 ft	..	b
Entry Losses	=	$0.5 \times (10.16)^2 / (2 \times 32.2) = 0.8$ ft	..	c
Losses in strainer	=	$0.05 \times (10.16)^2 / 2 \times 32.2 = 0.08$ft	..	d

Losses in column assembly and delivery pipe upto pressure main =

$$= 5.46 + 1.60 + 0.80 + 0.08 = 7.94 \text{ ft} \quad .. \quad \text{II}$$

Losses in pressure main, assuming 26" pressure main

$$V_{Pr} = \frac{15.20}{\Pi/4 \times (26/12)^2} = 4.12 \text{ ft/sec}$$

The velocity in 24" dia pressure main is 4.84 ft/sec. against the permissible limit of 4.5 ft/sec. Hence 24" dia pressure main is not suitable. The velocity in 26" dia pressure main is 4.12 ft/sec which is within the permissible limit of 4.5 ft/sec. Hence 26" dia pressure main is recommended.

Total length = 1608 + 3x68 = 1812 ft

$$\text{Loss of head} = \frac{4.69}{(26/12)^{4.87}} \times (15.2/110)^{1.85} \times 1812 = 4.98 \text{ ft} \quad ..\text{III.}$$

Exit losses = $(4.12)^2 / (2 \times 32.2) = 0.26$ ft. IV

Losses in manifold are negligible and hence taken as 'nil'.

Half the dia. of pressure main = 26/(2x12) = 1.08 ft V

Total pumping head including losses = 44.13+7.94+4.98+0.26+1.08

= 58.39 ft. or 17.80 metres - For Ns

Add 10% on losses towards ageing of pipes=58.39+1.32=59.71 ft.= or 18.20 m for HP.

Specific Sped @ 1460 rpm (Nominal)

$$Ns = \frac{3.65 \times 1460 \times (0.226)^{0.5}}{(17.80)^{3/4}} = 292.34 \text{ or say } 292.$$

$$\text{Specific speed in US Units} = \frac{1460 \times \sqrt{3582}}{(58.39)^{3/4}} = 4137 \text{ Francis / Mixed flow}$$

Hence (2+1) Nos., V.T. Pumps with Francis / mixed flow impellers running @ 1460 rpm (Nominal) are proposed.

<u>HP Calculations:</u>

HP = 62.45 x 7.98 x 59.71 / (550x 0.78) = 69.36

Add 10% towards cushioning = 6.94 **76.30 or say 80 HP**

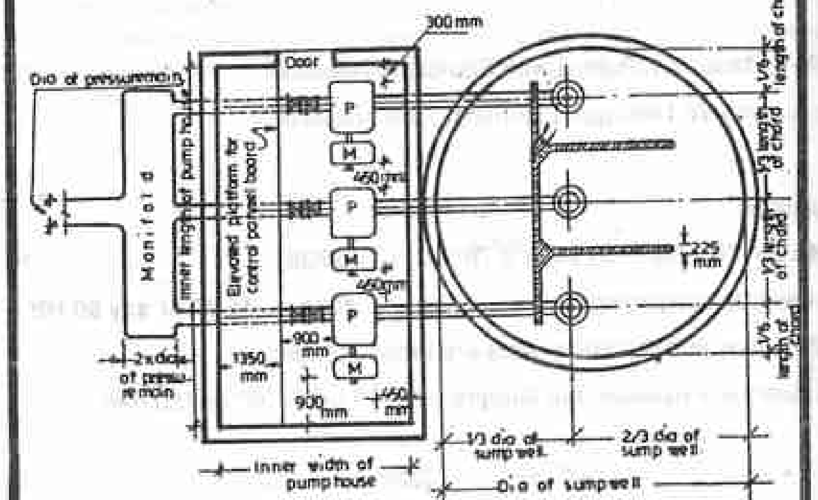
Hence, (2+1) Nos 80 HP, VHS motors are recommended.

Minimum clearance between the Girders = 2x12" + 6" = 30" or 750 mm.

***

ANNEXURE - I

SKETCH SHOWING THE APPROXIMATE DIMENSIONS AND LOCATION OF HORIZONTAL SHAFT PUMPS.



PUMP HOUSE SUMP WELL

Approx size of pump + motor = To be arrived from the chart enclosed vide annexure VIII & IX.

Length of chord = 3(450 + Length of pump & motor).

Inner dia of sumpwell = $\frac{2.1213}{2}$ x Length of chord.

Inner length of pumphouse = 3(Length of pump & motor) + 300 + 450 + 450 + 900

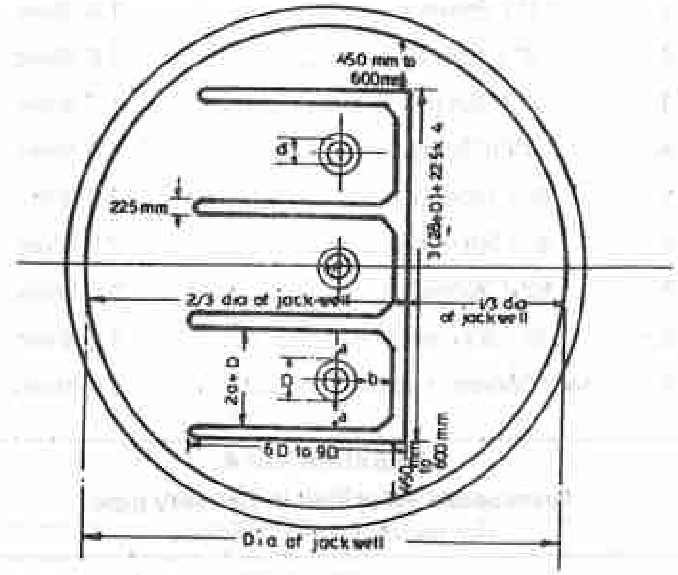
Inner width of pump house = 1350 + 900 + (Width of pump) + 450

Diameter of pressuremain = From pumpselection

Diameter of barrel type manifold = 2 x dia of pressuremain.

ANNEXURE - II

SKETCH SHOWING THE Approx. DIMENSIONS AND LOCATION OF V.T. PUMPS.



DETAILS OF JACKWELL CUM PUMP HOUSE, BAFFLE WALL, BACK WALL etc.

d = Dia of colum assembly

D = 1.5 to 1.8d.

a = 0.5 to 0.75 D.

b = 0.75 D to 1.0.

Bottom clearance = 0.6 D to 0.75 D.

Length of baffle wall = 6D to 9D.

Length of chord = 3(2a+D) + 225 x 4 + (450 to 600 mm) x 2.

Inner dia of jack well cum pump house = $\frac{2.1213}{2}$ x Length of chord.

Length of backing wall = 3(2a+D) + 225 x 4.

ANNEXURE - III

Permissible Velocities in Suction Pipe

Sl.No	Size	Permissible Velocity
1	1" (25mm)	1.5 ft/sec
2.	2" (50mm)	1.6 ft/sec
3.	3" (75mm)	1.7 ft/sec
4.	4"(100mm)	1.8 ft/sec
5.	6" (150mm)	2.0 ft/sec
6.	8" (200mm)	2.5 ft/sec
7.	10" (250mm)	3.0 ft/sec
8.	12" (300 mm)	4.5 ft/sec
9.	14" (350mm) above	5.0 ft/sec

ANNEXURE - IV A

Permissible Velocities in Delivery pipe
(for horizontal shaft pumps)

Sl.no.	Size	Permissible velocity
1.	1" (25 mm dia)	3.5 ft/sec
2.	2" ("50 mm dia)	3.6 ft/sec
3.	3"(75 mm dia)	3.8 ft/sec
4.	4"(100 mm dia)	4.0 ft/sec
5.	6" (150 mm dia)	4.7 ft/sec
6.	8"(200 mm dia)	5.5 ft/sec
7.	10"(250 mm dia)	6.5 ft/sec
8.	12"(300 mm dia)	8.5 ft/sec
9.	14"(350 mm dia) and above	10 ft/sec.

ANNEXURE -IV B

Permissible Velocities in column assembly & delivery pipe
(For V.T. Pumps)

Sl No.	Size	Permissible velocity
1.	1" (25 mm dia)	3.5 ft/sec
2.	2"(50 mmdia)	6.0ft/sec
3.	3"(75 mm dia)	7.0 ft/sec.
4.	4"(100 mm dia)	8.0 ft/sec
5.	6" (150 mm dia)	9.0 ft/sec
6.	8"(200 mm dia)	10.0 ft/sec
7.	10"(250 mm dia)	11 ft/sec.
8.	12"(300 mm dia)	11.50 ft/sec
9.	14"(350 mm dia)	11.75 ft/sec
10.	Above 14"(350 mm dia)	12 ft/sec.

ANNEXURE-IV C

Permissible Velocities in pressuremain
(For both Horizontal and V.T.pumps)

Sl No.	Discharge in each row	Permissible velocity
1.	Upto 5 cusecs	3.5 ft/sec
2.	Above 5 cusecs upto 15 cusecs	4.0 ft/sec
3.	Above 15 cusecs to 25 cusecs	4.5 ft/sec
4.	Above 25 cusecs	5.0 ft/sec

Note: The velocities indicated are for general guidence. It is necessary to estimate the HP requirements at least for three alternative diametre of pressuremains for selecting the pumps and motors which give least annual cost.

ANNEXURE-V

Equivalent length in feet of some common pipe fittings

Taken from the Circular issued by Chief Engineer, Minor Irrigation.

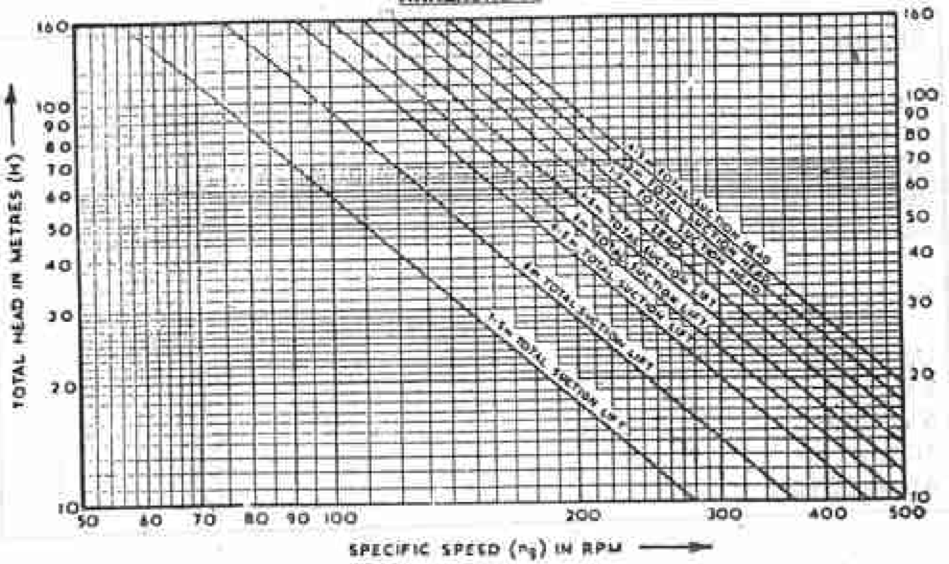
Pipe size	Std.90⁰ Elbow	Medium Bend	Gate Valve (Sluice Valve)
3/4"	2	1.7	0.5
1"	2.6	2.2	0.6
1¼"	3.5	3	0.8
1½"	4	3.5	1
2"	5	4.5	1.2
2½"	6	5.5	1.4
3"	8	6.5	1.8
4"	11	9	2.4
5"	13	12	2.9
6"	16	14	3.5
8"	20	18	4.5
10"	26	22	5.6
12"	30	26	6.5
14	36	33	8
16"	41	36	9
18"	43	40	10
20"	52	44	11
24"	63	54	14
30"	77	67	16
36"	92	76	20
42"	115	96	23
48"	130	110	26

Foot Valve with Strainer	..	44 ft.
Reflux Valve	..	44 ft.
Bellmouth with Strainer	..	30 ft.

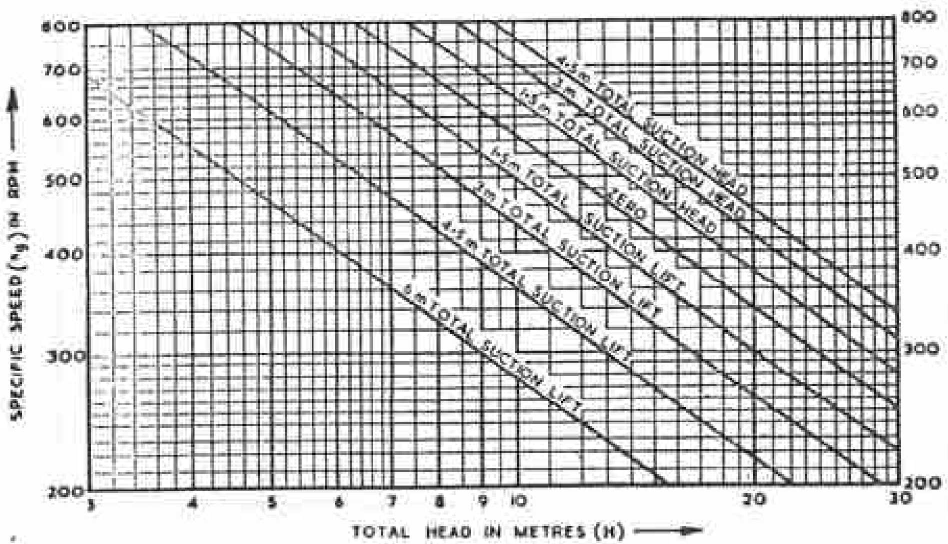
ANNEXURE-VI

Efficiency of Pump		
2 to 20 HP	-	60%
21 to 35 Hp	-	70%
36 to 75 HP	-	75%
76 to 200 HP	-	78%
Above 200 HP	-	80%
Cushioning		
Up to 2 Hp	-	50%
2 to 5 HP	-	30%
5 to 10HP	-	20%
10 to 20 HP	-	15%
Above 20 HP	-	10%

ANNEXURE-VII



SUCTION LIMIT CURVES FOR SINGLE STAGE, SINGLE AND DOUBLE SUCTION PUM



SUCTION LIMIT CURVES FOR SINGLE SUCTION MIXED FLOW PUMPS

ANNEXURE - VIII

Proforma of data sheet in which field Officer have to furnish details for Pumpselection :

Name of the Scheme with stsge No. :

I TECHNICAL DETAILS :

1. Total discharge :
2. LWL in sumpwell / Jackwell cum Pump House :
3. Bottom finished level of Jackwell / Sump well
4. Platform level of Pumps :
5. Length of pressure main in each row :
6. Dia and material of pressure main :
7. No. of rows of pressure main :
8. a. Delivery level at each cistern / each tapping

   b. Discharge required at each delivery cistern / each tapping :
9. Number of working pumps and number of stand bye pumps proposed :

Note : Schematic diagram showing all the levels and all constructional features from source of water upto delivery cistern shall be enclosed.

II General information required for procurement of Pumpsets etc.

1. Location of site of scheme with reference to Sub-division Head quarters / Division Head Quarters.
2. Complete postal address of consignee for receiving Pumps, Motors etc.
3. Name of Bank and Name of Branch for sending the despatch documents to Executive Engineer concerned.
4. Stage of procurement of pipes for pressure main.

5. Stage of completion of connected Civil works.

6. Programmed date of comissioning of the scheme.

7. Size of sumpwell, Pump House / Jack well cum Pump House as per actual execution.

NOTE : The purchase proposals are to be sent atleast nine(9) months in advance of programmed date of commissioning.

ANNEXURE - IX A

ATMOSPHERIC PRESSURE AT VARIOUS ALTITUDES

Altitude in Feet	Barometer Reading in Hg	Barometer Reading mm Hg	Atmospheric Pressure psia	Atmospheric Pressure ft. of water
-1000	31.0	788	15.2	35.2
Sealevel	29.9	760	14.7	33.9
+ 1000	28.9	734	14.2	32.8
2000	27.8	706	13.7	31.5
3000	26.8	681	13.2	30.4
4000	25.8	655	12.7	29.2
5000	24.9	633	12.2	28.2
6000	24.0	610	11.8	27.2
7000	23.1	587	11.3	26.2
8000	22.2	564	10.9	25.2
9000	21.4	544	10.5	24.3
10000	20.6	523	10.1	23.4

ANNEXURE -IX B
Properties of water

Temp. °F	Absolute Vapour Pressure pisa	Absolute Vapour Pressure ft. of water	Specific Weight lb/cu.ft	Specific Gravity*	Absolute Viscosity (Centipoises)
32	0.088	0.20	62.42	1.0016	1.79
40	0.122	0.28	62.43	1.0018	1.54
50	0.178	0.41	62.41	1.0015	1.31
60	0.256	0.59	62.37	1.0008	1.12
70	0.363	0.89	62.30	0.9998	0.98
80	0.507	1.2	62.22	0.9984	0.86
90	0.698	1.6	62.12	0.9968	0.81
100	0.949	2.2	62.00	0.9949	0.77
110	1.275	3.0	61.86	0.9927	0.62
120	1.693	3.9	61.71	0.9903	0.56
130	2.223	5.0	61.56	0.9878	0.51
140	2.889	6.8	61.38	0.9850	0.47
150	3.718	8.8	61.20	0.9821	0.43
160	4.741	11.2	61.01	0.9790	0.40
170	5.993	14.2	60.79	0.9755	0.37
180	7.511	17.8	60.57	0.9720	0.35
190	9.340	22.3	60.35	0.9684	0.32
200	11.526	27.6	60.13	0.9649	0.31
210	14.123	33.9	59.88	0.9609	0.29

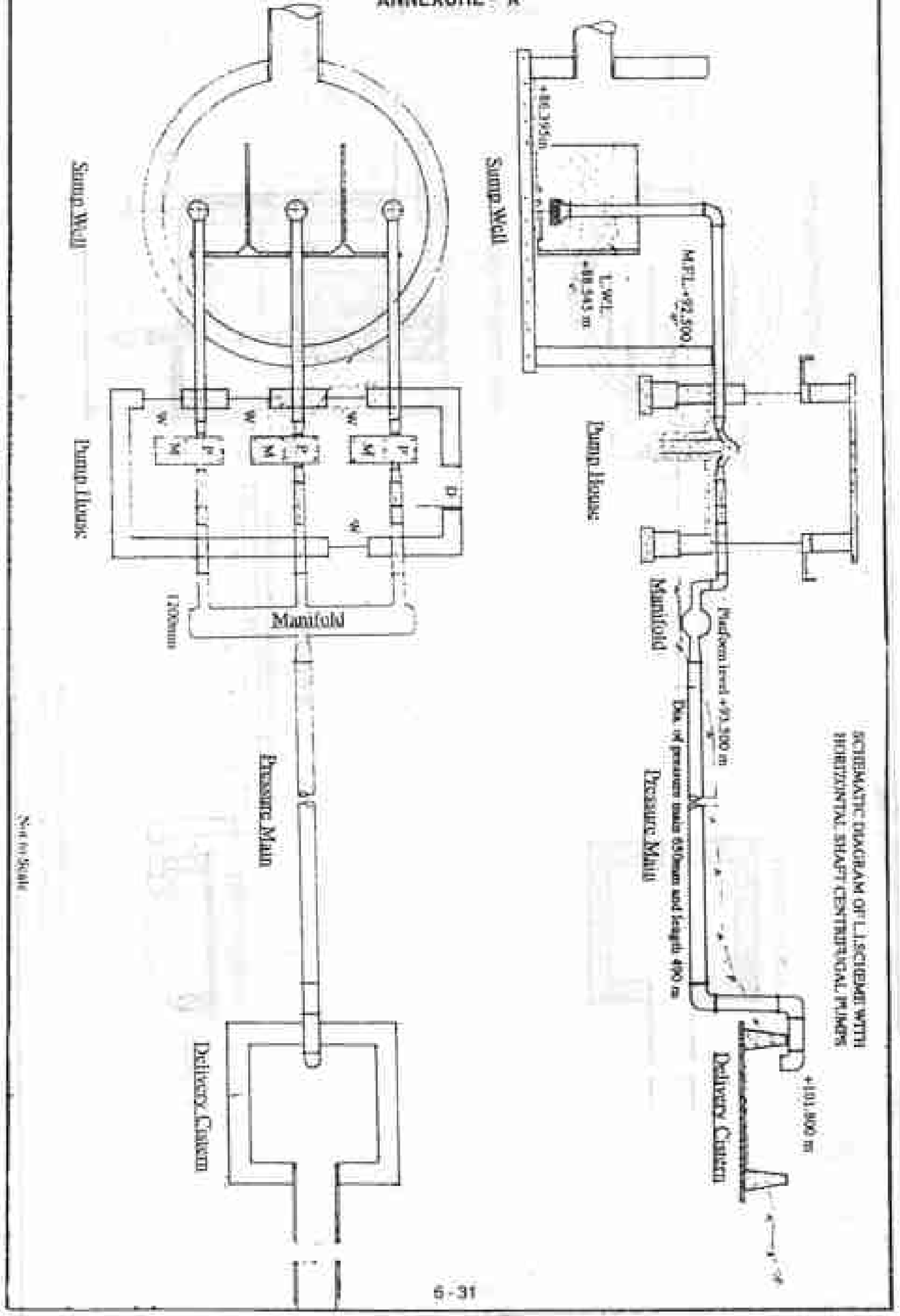
* Referes to water at 68°F, weighing 62.318 lb/cu ft. and have specific gravity of 1.0

ANNEXURE - IX C

As per the above table the water equivalent Column of Atmosphere pressure is shown here under, district wise.

Sl.No.	Name of the District	Pressure value in Feet
1.	Srikakulam District	33.90
2.	Vizianagaram District	33.90
3.	Visakhapatnam District	33.90
4.	East Godavari District	33.90
5.	West Godavari District	33.90
6.	Krishna District	33.90
7.	Guntur District	33.90
8.	Prakasham District	33.50
9.	Nellore District	33.90
10.	Kurnool District	33.00
11.	Cuddapah District	33.00
12.	Ananthapur District	33.00
13.	Chittoor District	33.00
14.	Khamman District	33.90
15.	Warangal District	33.90
16.	Karimnagar District	33.00
17.	Adilabad District	33,00
18.	Nalgonda District	33.50
19.	Mahaboobnagar District	33.00
20.	Medak District	33.00
21.	Nizamabad District	33.00
22.	Ranga Reddy District	33.00
23.	Hyderabad District	33.00

ANNEXURE - X



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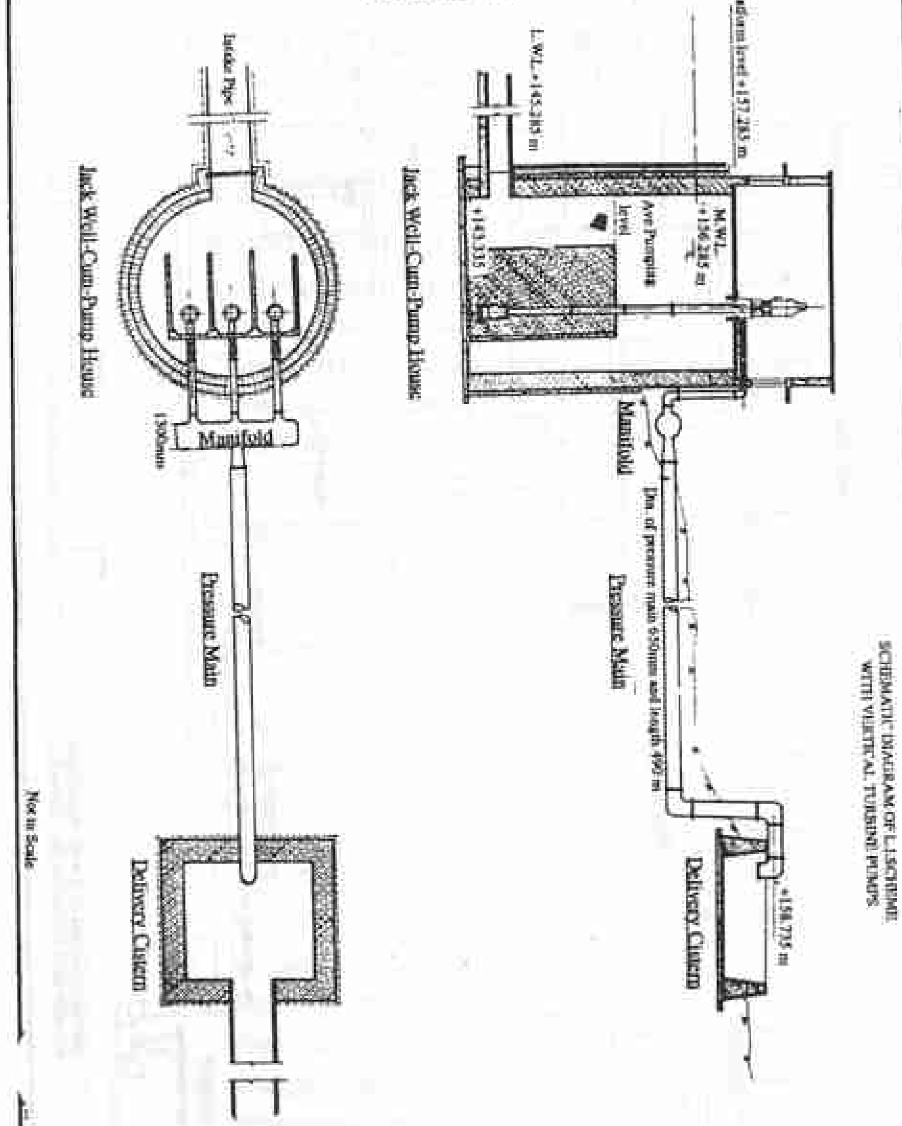
ANNEXURE - XI



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Chapter - VII

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B - SURVEYING TOTAL STATION & HIGHWAYS

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C - HIGHWAYS

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7A - SURVEYING

7.1. Corrections to be applied to measured length

7.1.1 <u>Correction for slope:</u> $C = L(1 - \cos\alpha)$

Let

h1	=	height of the instrument
h2	=	height of the target
θ	=	measured vertical angle
α	=	slope of the measured line
L	=	length of the measured line

$a = \theta + \delta\theta$, $\quad \sin\delta\theta = (h1 - h2)\cos\theta / L$

$\delta\theta = 206265\,(h1 - h2)\cos\theta / L$

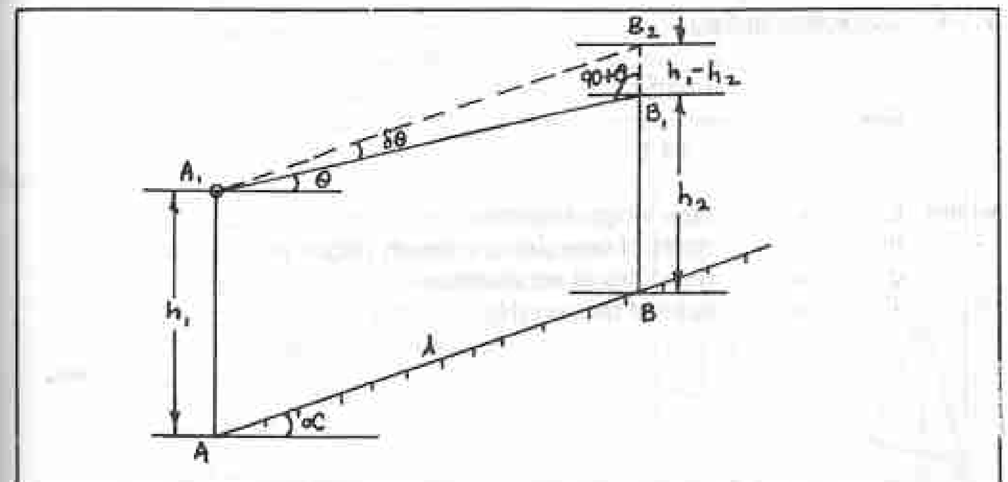
(1) If h1 = h2 $\delta\theta = 0$, $\alpha = 0$

(2) If h1 < h2 and θ is +ve, $\delta\theta$ is -ve

(3) If h1 > h2 and θ is -ve, $\delta\theta$ is +ve

if α is +ve, $\delta\theta$ is +ve

(4) If h1 < h2 and θ is -ve, $\delta\theta$ is -ve



7.1.2 Correction for temperature

$C = L.\ \alpha\ (tm - t_s)$

Where	L	=	the measured length
	α	=	Coefficient of linear expansion of the tape metal
		=	10.60 to 12.2 (x 10^{-6}) / °C for steel
	t_m	=	field temperature of the tape
	t_s	=	temperature at which measuring tape is standardised

7.1.3. Correction for Tension

$$C = \frac{L(T_m - T_s)}{A.E.}$$

Where	L	=	The measured length
	A	=	Cross sectional area of the tape
	E	=	Young's modulus of Elasticity (N/mm^2)
	T_m	=	Tension applied in the field
	T_s	=	Tension at which the tape is standardised

7.1.4. Correction for Sag

$$C = \frac{(mg)^2 L^3}{24 T^2}$$

where,	L	=	tape length recorded
	m	=	mass of tape per unit length (Kg/m)
	g	=	gravitational acceleration
	T	=	applied tension (N)

7.2 Adjustment of Dumply Level

7.2.1 Adjustment of the Perpendicularity between Vertical Axis and Level Tube Axis:

Test

1. Set up the instrument on firm ground, and level carefully in two positions at right angles to each other.
2. Swing the telescope through 180^0. If the bubble remains central, the adjustment is correct:

Adjustment - 1

. If not bring the bubble halfway back by the adjusting screws, connecting the bubble tube to the telescope

Adjustment - 2

2. Level up, and repeat the test and adjustment until correct.

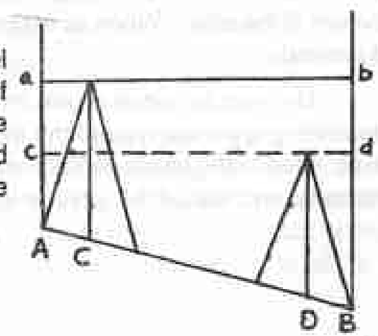
7.2.2. Adjustment of the Line of Collimation or the Collimation Adjustment

Test

1. Peg 2 points, A&B (see fig.) 90 to 120m apart on fairly level ground. Set the level between them at C, so that the eyepiece will almost touch the face of levelling staff held on A.
2. Level up carefully, and note the reading b of the staff held on B: then, by looking through the object glass, observe the reading 'a' of the staff on A.
3. Transfer the instrument to D near B, level up, and read the staff on both pegs as before, obtaining the readings 'c' and 'd'. If the difference of level between A and B, as deduced from readings 'a' and 'b' equals that given by 'c' and 'd', the result is their true difference of level, and the adjustment must be correct, since such equality could be secured only by a horizontal line of collimation.

Adjustment -1.

If not, compute the true difference of level between A and B. This is given by the mean of the two erroneous differences. If D be the true difference, of level, and 'e' the error introduced in reading the far staff,then, for the case illustrated,



b-a = D+e and c-d = D-e

D = [(b-a) + (c-d)] / 2

Adjustment-2

By applying D to the reading c, compute d, the reading on A at the same level as c. The instrument being still at D turn the top and bottom diaphragm adjusting screws until the line of collimation is directed to d, taking care that the bubble is in the centre of its run. The line of collimation is now horizontal.

Alternative Method

Test-1. Drive two pegs, A and B, as before. Set up the instrument at C, exactly midway between them, level up carefully, and read a staff on A and on B. The difference of the readings a and b is the true difference of level.

2. Set the instrument near one of the pegs and preferably beyond it, at D, level up, and note the readings c and d of the staff on the pegs. If the difference between c and d equals the true difference of levels, the adjustment is correct.

Adjustment : If not, by applying the true difference of level to the reading C, deduce d, the reading of A which would be observed by a horizontal sight from D, and by means of the diaphragm adjusting screws direct the line of collimation to d, taking care that the bubble is central.

7.2.3 Sensitiveness of a Level Tube

The sensitiveness or sensitivity of a level tube means its capability of showing small angular movements of the tube vertically. It depends upon the radius of curvature of the tube which may vary from 10 to 300m. The larger the radius of curvature, the greater the sensitiveness (or the longer the bubble, the more sen sitive it is). The sensitiveness is some times expressed in terms of the radius of curvature, but it is more usually expressed in terms of the angle through which the axis of the level tube must be tilted to cause the bubble to move through one division of the scale, (i.e., the angular value of one division), or in terms of the angle subtended at the centre by an arc of one division of the scale. When so expressed, the sensitiveness varies inversely as the number of seconds.

The angular value of one 2mm, division of the tube may vary from 8 to 45 seconds, depending upon the type of the instrument. For example, the sensitiveness of the spirit levels fitted to the levels varies from 20 to 30 seconds, that of the plate level from 40 to 45 seconds, and that of the altitude level from 8 to 20 seconds.

7.3. Adjustment of Prismatic compass

7.3.1 Source of Error in Compass Observations

The errors may be classified as (1) instrumental errors, (2) errors of manipulation and sighting, and (3) errors due to external influences.

(1) Instrumental Errors: (a) The needle not being perfectly straight, (b) the pivot being bent, i.e. not being at the centre of the graduated circle, (c) the needle being sluggish, i.e. the needle having lost its magnetism, (d) the pivot point being dull, (e) the needle neither moving quite horizontally nor moving freely on the pivot due to the dip of the needle, (f) the plane of sights not being vertical, (g) the graduated circle not being horizontal, (h) the line of sight not passing through the centre of the graduated ring, and (i) the vertical hair being too thick or loose.

(2) Errors of manipulation & sighting: (i) Inaccurate centering of the compass over the station occupied, (ii) Inaccurate levelling of the compass box (the compass box not being horizontal) when the instrument is set up. (iii) Imperfect bisection of the ranging rods at stations or other objects. (iv) Carelessness in reading the needle or in reading the graduated circle through the prism in a wrong direction. (v) Carelessness in recording the observed readings.

(3) Errors due to external influences: (i) Magnetic changes in the atmosphere on cloudy or stormy day, (ii) Irregular variations due to magnetic storms, earthquakes, sun spots, lunar perturbations etc., (iii) Variations in declination viz., solar annual and biannual, (iv) Local attraction due to proximity of steel structures, electric lines etc.

7.3.2 Testing the Compass

The compass (surveyor's or prismatic) should be tested in the following manner to see if it is trustworthy (i.e. in good working order).

1. To see if the needle is horizontal when the compass is properly levelled. If not, slide the rider (small coil of brass wire) towards the high end so as to make it perfectly horizontal. In the case of the prismatic compass, the dial should be balanced by correctly setting the rider.

2. To find if the needle is straight and the pivot is at the centre of the graduated circle. Read both ends of the needle. The difference in the readings pointed to by the North and South ends of the needle will be exactly 180^{0}., if the needle be straight and the pivot in the centre. If the difference is not equal to 180^{0} it may be due to pivot not being in the centre or to a bent needle or to both. If the difference between the two end readings for

different positions of the compass is constant, it may be taken that the pivot is in the centre and the error is due to the needle not being straight. To correct it, bend the needle straight. If the difference in the readings is not constant the correction is made by bending the pivot and straightening the needle.

3. To ascertain if the needle is sluggish. It will not be exactly on the magnetic meridian when it comes to rest. Sluggishness of the needle may be caused by the loss of magnetism or by the bluntness of the supporting pivot, but it is more often due to the dullness of the pivot rather than to the loss of magnetism. To detect it, sight any object and take the reading. Turn the sights away and immediately bring them back and resight then same object and observe the reading. The two readings will be same, provided the needle is sensitive and the pivot point sharp.

Alternatively, after the first reading is taken, displace the needle by means of a piece of steel, say, a key and again take the reading. The reading will be the same as before, if there is no friction on the pivot and the needle is not sluggish.

4. To find if the sights are vertical and fixed diametrically opposite to each other. Suspend a plumb line in front of the compass. Level the compass and sight the string. If the sights are vertical, the eye vane, object vane, and the string will be parallel and in the same line.

5. To see if the line of sight passes over the zeros on the circle. Stretch a fine horse hair between the slits and see whether it passes over the N and S marks(zeros).

6. To detect if there be any observational error or errors due to external influence. Take the fore and back bearings of a line. These will differ exactly by 180^0, if the working is correct and there are no external influences.

Limits of precision : In the case of the prismatic compass the least value that can be estimated is 15 minutes and, therefore, the permissible error per bearing should never exceed this amount. The angular error or closure or summation error in minutes should not exceed 15 N where N is the number of stations or sides of a traverse.

The relative error of closure should between 1 in 300 to 1 in 600.

7.4 Adjustment of theodolite

7.4.1 Temporary adjustments

1. Setting over the station
2. Levelling up
3. Elimination of parallax.

Setting up: This includes both centering of the instrument over the station by the plumb bob and its approximate levelling by manipulation of the tripod legs only. First, hold the instrument off the ground with the legs spread out and plumb bob hanging approximately over the station. On being lowered to the ground it will not be seriously out of centre, and the exact centering and approximate levelling may be completed by small radial and tangential motions of two legs and the pushing of all three into the ground. On a hill side, place one leg uphill.

Levelling up:

1. Turn the upper plate until one level tubes is parallel to any pair of screws. The other tube will be parallel to the line joining the third screw and the point midway between the first pair.

2. Manipulate any pair of screws to bring the bubble to the centre of its run in the tube to which they are parallel. In turning the screws, the thumbs move towards or away from each other and the left thumb must be moved in the direction in which the bubble is required to travel.

3. Level the other tube with the remaining screw by using one hand only.

Elimination of parallax:

1. Turn the focussing screw until no object can be distinguished in the field, or point the telescope to the sky. Adjust the eye-piece until the cross hairs appear in sharp focus.

2. Point the telescope towards the object, and keeping the eye on the cross-hair the focussing screw until the image appears in sharp focus. The image and hairs should then be in the same plane, but the eye should be moved about to test whether any relative movement between them can be observed. If necessary, adjust the focussing screw until the apparent movement is eliminated.

7.4.2 Permanent Adjustments of Theodolite

1. Adjustment of the Plate Level: Object: To set the axis of the plate level tubes perpendicular to the vertical axis.

Test- 1 Set up the instrument on firm ground. Fix the lower clamp, and level the plate bubble carefully.

2. Swing the upper plate through 180^0. If the bubble remains central, the adjustment is correct.

Adjustment-1: If not, bring the altitude level, attached to the telescope or the index arm, parallel to a pair of levelling screws. If this tube is mounted on the telescope, set it approximately level by hand, clamp the vertical circle, and complete the levelling by the vertical tangent screw or the levelling screws, if it is mounted on the vernier frame, use the levelling screws. Turn through 90^0 and centre the bubble by the levelling screws. Repeat until the bubble is central in these two positions.

2. Swing through 180^0 the bubble will leave the centre of its run. Bring it half-way back by the levelling screws and the remainder by the vertical circle tangent screw or the clip screws. Repeat until the bubble remains central in any position.

3. The vertical axis is now truly vertical. By means of the adjusting screws of the plate levels bring the bubble of each to the centre of its run.

7.4.3 <u>Adjustment of the line of sight, or the collimation adjustment</u>

Object: To make the line of sight coincide with the optical axis of the telescope.

Horizontal Hair: Test

1. Set up and level the instrument carefully, with all the clamps fixed, take a reading on a levelling staff held upon a firm point few hundred feet away. Note the reading and the vertical angle.

2. Unclamp, transit the telescope and swing through 180^0 set the vertical circle to the same angle as before.

3. Again read the staff. If the previous reading is obtained, the adjustment is correct.

Adjustment:

1. If not, move the diaphragm by the top and bottom capstans until the staff reading is the mean of those previously obtained.

2. Repeat until no error is perceptible on changing face.

Vertical Hair: Test:

1. Set the instrument on a fairly level stretch of ground and in such a position that a sight of about 90m may be obtained on either side. Level up.

2. Establish a point A at about 90m from the instrument by thrusting a chaining arrow into the ground, or otherwise, Sight A, and clamp the horizontal movement.

3. Transit, and mark B at about the same level as A and so that OB=OA approximately.

4. Unclamp, swing through 180^0, and again sight A and clamp.

5. If, on again transiting, the line of sight intersects B, it is perpendicular to the horizontal axis.

Adjustment:

1. If not, mark C in the line of sight. Measure out from C towards B a length CD= (1/4) CB, and mark D.

2. By means of the diaphragm side capstans bring the vertical hair to D.

3. Repeat until no error is perceptible on changing faces.

Adjustment of the Horizontal Axis:

Object: To make the horizontal axis perpendicular to the vertical axis, so that it is truly horizontal when the instrument is levelled up.

Test: 1. Set the instrument in such a position that a highly inclined sight may be obtained to a well-defined point A,e.g. the final of a steeple. Level up very carefully, and sight A.

2. With both clamps fixed, depress the telescope, and mark a point B on the ground in the line of sight.

3. Unclamp, transit the telescope, and swing through 180^0. Sight on B and with the horizontal movements clamped, elevate the telescope. If the line of sight again cuts A, the horizontal axis is truly horizontal, and is therefore perpendicular to the vertical axis.

Adjustments: 1. If not, let the hairs appear at C, opposite, A, by means of the adjusting screw at the trunnion support on one standard bring the line of sight from C to D, estimated midway towards A.

Adjustments: 2. Again perform the Test, making a new point 'B' on the ground and repeat the test and adjustment until no error is detected on changing face.

Adjustment of the Telescope Level: Object: To place the axis of the level tube attached to the telescope parallel to the line of sight.

Test and adjustment: The procedure is exactly the same as in the "two peg" adjustment of the dumpy level the adjustment being made by means of the screws attaching the level tube to the telescope.

Adjustment of the Vertical Index Frame

Object: To make the vertical circle read zero when the line of collimation is horizontal.

Test: 1. Level the instrument by plate levels and by means of the vertical circle clamp and tangent screw set the vertical circle to read zero.

2. Bring the bubble on the index arm exactly central by means of the clip screw at the standard. Observe a levelling staff held 60m or 90m away and note the reading.

3. Release the vertical circle clamp, transit the telescope, and again set the vertical circle to read zero. Swing through 180^{0}, re-level if necessary, and again read the staff held on the same point. If the reading is unchanged, the adjustment is correct.

Adjustment: 1. If the reading differ, bring the line of collimation on to the mean reading by turning by the clip screw.

2. Return the bubble of the altitude level to the central position by means of the adjusting screws attaching it to the index arm.

3. Repeat until no error is discovered in the test.

Considering now the case of an instrument in which the clip and tangent screws operate on separate arms, the elimination of index error and the adjustment of the altitude level are performed as follows, whether the level is mounted on the vernier arm of the telescope.

Test: As in the last case, if the level is on the vernier arm, but, if a telescope level only is fitted, set the bubble central by the levelling screws.

Adjustment: 1. If the readings differ, bring the line of collimation on to the mean reading by turning the vertical circle tangent screw.

2. Return the vernier index to zero by means of the clip screw.

3. Bring the bubble of the level central by means of the adjusting screws attaching it either to the vernier arm of the telescope.

4. Repeat until no error is discovered in the test.

CONCEPTS OF TOTAL STATION SURVEY

7. B SURVEYING TOTAL STATION

1.0 INTRODUCTION:

Surveying is the process of determining the earthen features and transferring its relative position on to the paper. Surveying principles can be applied in a variety of ways to accomplish the aim of position finding. The preferred survey method for both two and three dimensional position finding has changed through the years in response to the advances in technology. Different methods considered in modern surveying include precise leveling & traversing, Astronomical surveying, Photogramentry, EDM, Remote sensing, GIS, GPS, DGPS etc.,

2.0 BASIC MAPPING TERMS IN SURVEYING:

To be able to map irrigation applications and to operate the total station and data collector effectively, it is necessary to know some basic mapping concepts related to how locations are designated. It is essential that one is familiar with how to use a compass and how to read topographic maps. In fact, it is recommended that one know how to map a site using a compass and measuring tape or pacing before learning how to use the total station. If the cardinal directions and distances are understood well, then one will be able to know if he or she is using the total station correctly. The cardinal directions refer to north, east, south, and west, which are essential reference points for site mapping. Depending on the mapping project it may be more appropriate to record locations relative to true rather than magnetic north. It is important to know about azimuths and bearings, and how either of these readings along with distance can be used to plot the location of a point. One needs to know how to read and determine UTM (universal Transverse Mercator) coordinates. The total station records points by translating the azimuth and distance between the site datum and the point being recorded into UTM a coordinates. Therefore, it is crucial that one should understand the relationship between azimuths/bearings and distances, and UTM coordinates.

2.1 An azimuth is any direction read on a 360° circle, which relates to the cardinal directions. North is read as both 0° and 360°, east as 90° south as 180°, and west as 270°. When using a compass to map a site, points are plotted according to the azimuth between the site datum and the point, and the distance between them is recorded generally in meters (m).

2.2 Regardless, the site datum is the reference point for all locational data recorded at a site and, therefore, it is where all mapping activities must begin. The total station serves the same purpose as a compass in terms of keeping track of how azimuth designations relate to actual directions. However, it differs in that it can not determine true or magnetic north by itself: an external compass must be used to determine north, and then the direction is set on the total station.

2.3 When site mapping, engineers generally employ a combination of azimuths/beings and UTM (Universal Transverse Mercator) coordinates. Among various reasons, what map systems are used depends on the nature of the mapping project, the location on earth, and what cartographic conventions are used in that region.

3.0 EDMs And TOTAL STATION:

3.1 DISTANCE MEASUREMENT:

There are three methods of measuring distance between points 1) Direct distance measurement (DDM) such as the one by chaining or taping ii) Optical distance measurement (ODM) such as the one by tacheometry, horizontal subtense method or telemetric method using optical wedge attachments iii) Electromagnetic distance (EDM) such as the one by Geodimeter and, Distomat.

The method of direct distance measurement in unsuitable in difficult distance measurement is unsuitable in difficult terrain and sometimes impossible when obstructions occur. The problem was overcome after the development of optical distance measuring methods but in ODM method also the range is limited to 150 m and the accuracy obtained is 1 in 1000 to 1 in 10000. Electromagnetic Distance Measurement enables the accuracy up to 1 in 100000 over ranges up to 100 km.

3.2 ELECTRONIC DISTANCE MEASUREMENT:

Electromagnetic distance measurement (EDM) equipment consists of an aiming head/receiver unit set at one end of the line to be measured and pointed towards a reflective glass prism set up at the other end. An electromagnetic beam is emitted by aiming head, projected towards the prism, reflected back and analyzed to determine the distance.

3.2.1TYPE OF EDM INSTRUMENTS

Depending upon the type of carrier wave employed, EDM Instruments can be classified under the following three heads.

i) Micro Wave Instruments
ii) Visible light Instruments
iii) Infra red Instruments

Distomat is a very small compact EDM particularly useful in building construction, Civil Engineering Construction, Cadastral and detail survey, particularly in populated areas where 99% of distance measurements are less than 500m.

4.0 ADVANTAGES OF MODERN SURVEYING INSTRUMENTS AND METHODS:

i) Traditional survey methods are laborious and time consuming
ii) Familiar manual procedures have been integrated and automated
iii) Fully automatic electronic measurement
iv) Digital display of staff reading and distance
v) Single measurement or averaged repeat measurements.
vi) Data storage in Instruments possible
vii) Direct transfer to Personal computer of data stored in Instruments
viii) Online operation through integrated interface to computer.

5.0 TOTAL STATION:

Instrument of choice for the modern surveyor integrates an electronic digital theodolite, a electronic distance measuring instrument and a computer in to single unit. The resulting hybrid instrument is called a 'Total Station' A total station automatically measures and displays distance and direction data (both horizontal and vertical angles) and results to its computer.

Hence the total station instrument is a combination of

a. Distance measuring instrument (EDM)
b. Angle measuring instrument (Theodolite)
c. A simple micro processor

The total station emits a laser signal that bounces off the prism reflector at the top of the rod and then returns to the gun. The distance and slope at which the laser beam travels provides the basis on which the data collector calculates the azimuth to and elevation of the point on which the rod is placed.

The total station emits a laser signal that bounces off the prism reflector at the top of the rod and then returns to the gun. The distance and slope at which the laser beam travels provides the basis on which the data collector calculates the azimuth to and elevation of the point on which the rod is placed.

6.0 COMPONENTS OF TOTAL STATION:

6.1 GUN:

The total station is referred to generically as the gun. The process of sighting the rod with the gun, and then recording the locational data using the data collector, is referred to as "shooting" **points.**

6.1.1 OPTICAL PLUMB BOB:

There is the optical plummet telescope, or optical plumb bob that looks like another knob and is located immediately to the left of the data screen. The optical plumb bob can be viewed through to see the ground directly below the center of the gun. When using this plumb bob, the gun can not be screwed into the tripod, since it will obscure the view. The gun must be sitting on top of the tripod loosely, so that it can be shifted around until it is positioned directly over the marker. When looking through the knob, there are two black circles and one dot arranged in a "bulls-eye" fashion that allow the user to line up the gun precisely on a location.

6.1.2 EYEPIECE AND SIGHTING COLLIMATOR:

The easiest way to line up the crosshairs on the prism reflector is to rely on the sighting collimator, or sight, which is located on the top of the gun below the carrying handle. The sight can be used to get the gun into range with great accuracy without having to look through the eyepiece. The key is to line up the triangle viewed in the sight on the prism reflector. This should place the prism reflector close enough to the crosshairs such that the focus adjustment knobs can be locked down without having to look through the eyepiece. Once the knobs are locked down, the user can look through the eyepiece and make fine-grained adjustments to line up the prism reflector on the crosshairs perfectly. However, it will take some practice using the sight and knowing how the sight works in order to do this accurately every time.

There are two ways to focus the eyepiece such that both the prism reflector and the crosshairs can be viewed clearly. First, if the prism reflector is blurry, rotate the gray ring, or objective focusing knob, that encircles the eyepiece until it is clear. Second, if the crosshairs are blurry, adjust the small black knob that is part of the eyepiece, which is known as the reticle focus know.

6.1.3 BATTERY:

Battery is attached to the gun on the right hand side when facing the eyepiece and data screen. The battery locks into place with a locking lever at the top of the battery. It is essential that the battery be re-charged daily when conducting fieldwork. There should always be at least one fully charged spare battery in case the one being used needs to be replaced.

6.1.4 ENVIRONMENTAL BOX:

The data collector is stored in an environmental box to prevent the elements from penetrating into the equipment. In particular, the environmental box prevents heavy misting and light rain from affecting the performance of the data collector. However, one note of caution is that condensation can build up inside the environmental box in humid and sunny conditions. In humid environments, it is recommended that the data collector should be taken out of the environmental box when not in use and allowed to "breathe." Furthermore, it may be necessary to clean the contacts on the data collector regularly, even nightly, to prevent potential problems. Too much humidity (more than 90% relative humidity) can corrode the contacts such that data collector can not read information from the gun, and therefore can not record points. In general, the data collector should be stored in moderately cool, shady, and dry conditions as much as possible when it is not being used.

6.1.5 HP-48CX CALCULATOR AND TDS-48GX SURVEY CARD:

The data collector is composed calculator with TDS-48GX survey card that is inserted into the back of the calculator. The data collector referes to the calculator when the survey program is enabled.

6.1.6 BATTERIES OF CALCULATOR AND RAM CARD:

There are two parts of the data collector that use batteries: the calculator and the RAM card. It is essential that there are always spare batteries in the field. Whenever one is preparing to go into the field to conduct site mapping, make sure that extra batteries of both kinds are included in the bag in which the data collector is stored.

The calculator is operated using three AAA alkaline batteries. To change the batteries, open the environmental box by unscrewing the two spring- loaded screws located on the front of the box when viewing the calculator. Then, lift the calculator out carefully and turn it over. The batteries are located inside the calculator at the base of its backside.

The RAM card is located immediately below the survey card in the back of the calculator. The card uses one 3-volt lithium battery (CR2016) that should be replaced about once a year. When the battery is low, it will be indicated by a symbol displayed on the data collector screen

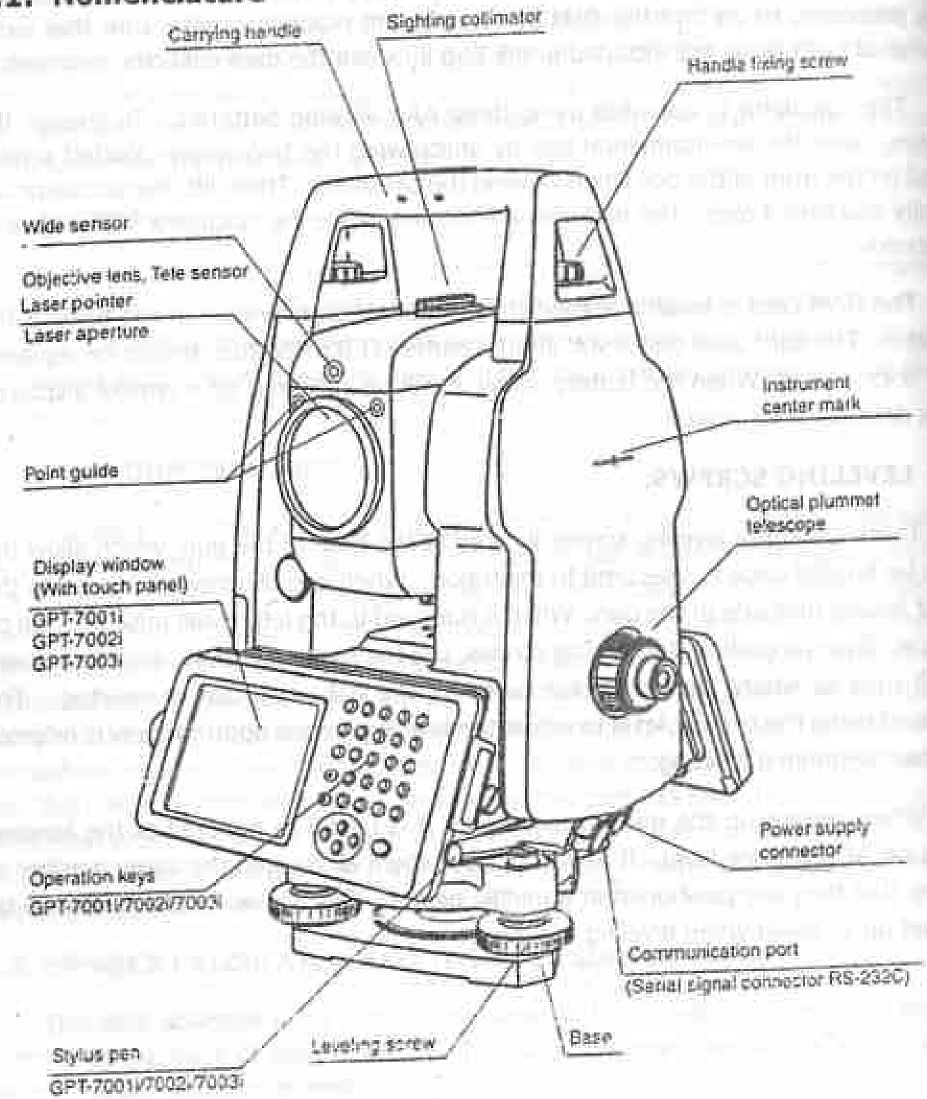
6.1.7 LEVELING SCREWS:

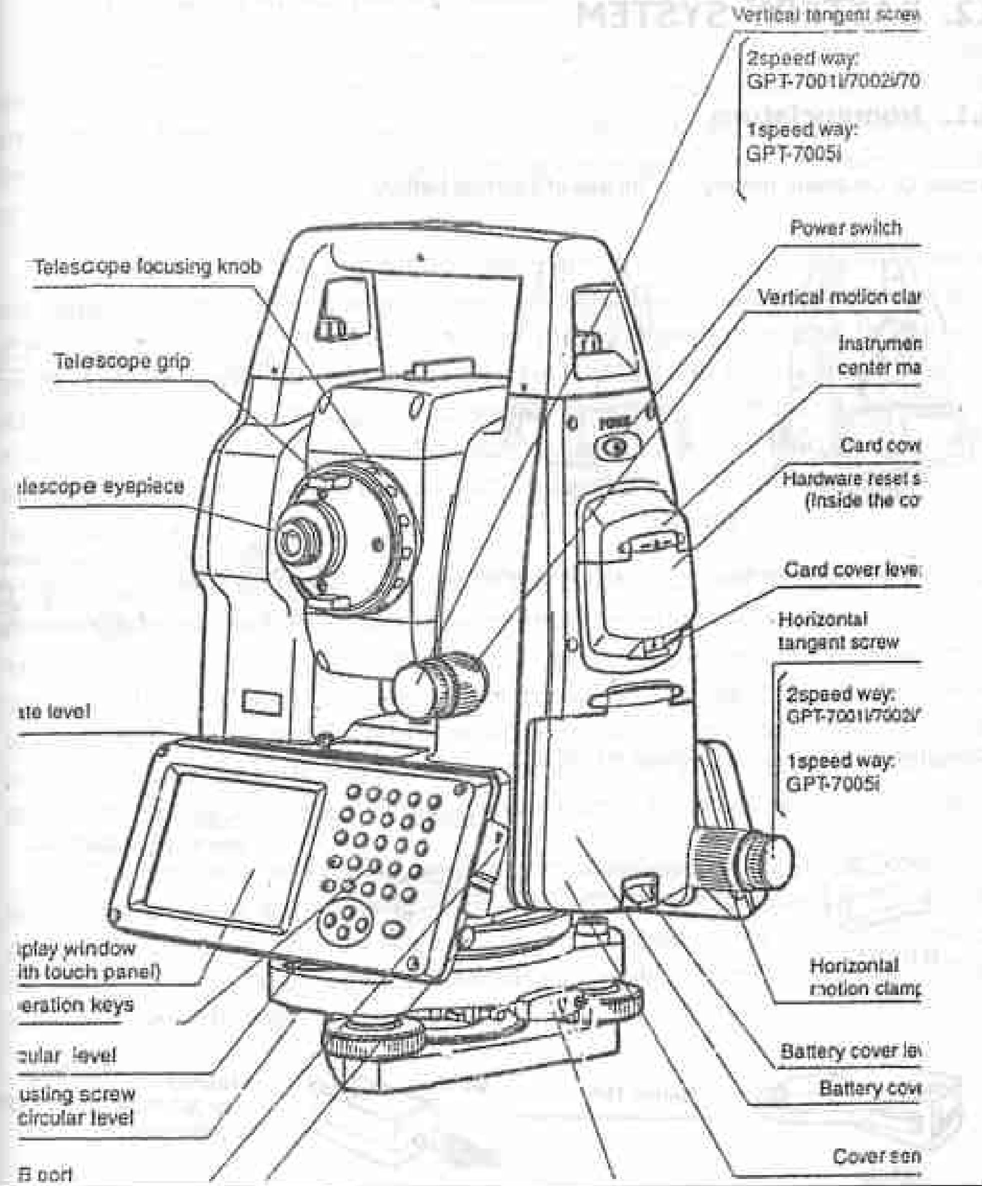
There are three leveling screws located at the base of the gun, which allow the gun to be leveled once it is secured to the tripod. When one of screws is turned to the right, it lowers that side of the gun. When it is moved to the left, it will raise the gun on that side. When adjusting the leveling screws, use the leveling bubble, or circular level, located next to where the connector cable for the data collector is inserted. The process of using the circular level to adjust the leveling screws appropriately is referred to as coarse-grained leveling.

When setting up the gun on the tripod, it is useful to have all of the leveling screws set at the same level. It is best to have them all rotated the same number of turns so that they are positioned at a middle height. This allows for the screws to be adjusted up or down when leveling the gun.

1. NOMENCLATURE AND FUNCTIONS

1.1. Nomenclature



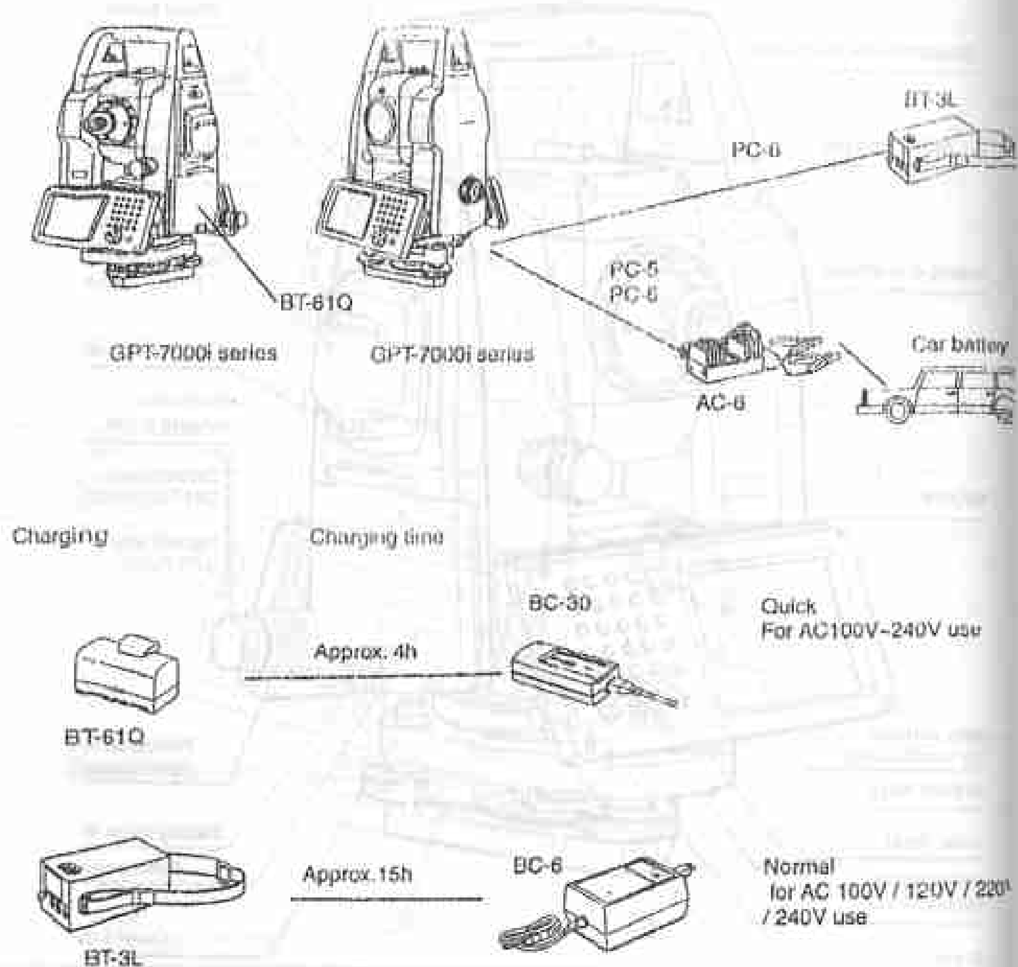


*1) The position of vertical motion clamp and tangent screw will differ depend on the markets.

12. BATTERY SYSTEM

1.1. Nomenclature

In case Of On-board battery   Incase of External battery



6.1.8 FOCUS ADJUSTMENT KNOBS:

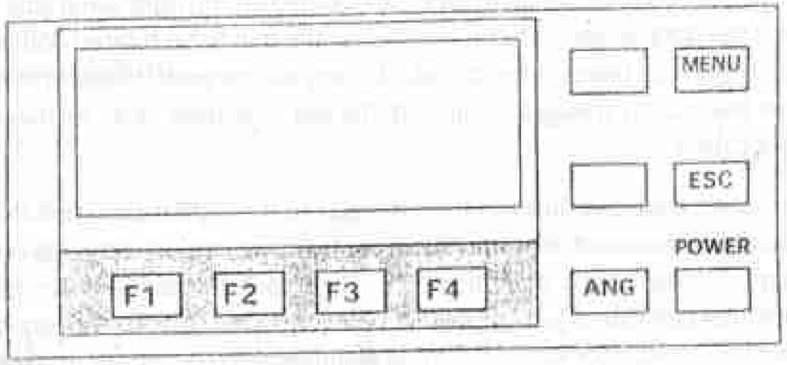
There are two focus adjustment knobs located on the right-hand side of the gun when facing the data screen. These knobs allow the gun to be rotated 360 horizontally and vertically from its base on the tripod. As long as the prism reflector is visible, the knobs allow the user to line up the gun with the rod regardless of its relative horizontal or vertical location.

The lower knob that just out from the gun to the right is the knob that controls the horizontal orientation of the gun. There are two separate adjustments on the knob, of which one is closer to gun than the other. The closer part of the knob is referred to as the horizontal lock clamp and the further part is referred to as the horizontal tangent screw, when horizontal lock clamp is turned all the way to the left, or towards the gun, it is totally loose and the gun can be rotated in any horizontal direction. When it is turned all the way to the right the gun is locked in that position and can not be moved horizontally.

The second knob is located above the lower knob and just out in a forward direction from the gun. The upper know controls the vertical orientation of the gun, of which the parts are referred to as the vertical lock clamp and vertical tangent screw. In contrast to the horizontal lock clamp, when the vertical lock clamp is turned all the way to the right, it is loose and the eyepiece can be rotated up or down. The gun is locked in position when the vertical lock clamp is turned all the way to the left. Whenever a point is being shot, it is essential that the lock clamps be locked down in this manner.

The horizontal and vertical tangent screws control the fine-grained adjustment once the lock clamps are locked down. This means that the lock clamps can be locked down when the prism reflector is close to being centered on the crosshairs viewed in the eyepiece. Then, fine-grained adjustments can be made such that the prism reflector is lined up perfectly with the crosshairs.

6.1.9 DATA SCREENS:



6.2 PRISM REFLECTOR:

The Prism reflector is an updated version of the stadiarod that is used in conjunction with conventional optical transits. The Prism reflector itself is the small square. Shaped piece that contains a circular mirror. This piece is attached to the metal rod by screwing its base into the top of the rod.

The total station emits a laser signal that bounces off the prism reflector at the top of the rod and then returns to the gun. The distance and slope at which the laser beam travels provides the basis on which the data collector calculates the azimuth to and elevation of the point on which the rod is placed.

There are two black spring-loaded grips on the rod used to adjust the height. The rod can be extended or shortened when the grips are depressed by wrapping the palm of the hand around them. The rod measures 1.65 m in height at its shortest, and can be extended to a maximum of 3.78 m using both of the grips.

The most important aspect of holding the rod when recording points is to maintain as much accuracy as possible. First the base of the rod need to be placed precisely at the location that is to be recorded. If the rod can not be placed directly on the point, then position it as close as possible and record the difference in the field notes.

It is important that the rod is level to ensure that the elevation is recorded correctly. The rod has a built-in leveling bubble for this purpose. The person holding the rod should stand in a stable position (it is best to have your legs spread apart somewhat) and then firmly hold the rod near where the leveling bubble is located. Once the person is in position and the rod is leveled, the point is ready to be recorded. While the point is being shot, the person should keep his/her eyes on the leveling bubble to make sure that the rod doesn't move.

The taller the rod is, the more difficult it is to keep it level in windy and inhospitable conditions. Consequently, points should be recorded at the lowest rod height possible to make it earier on the person holding the rod. The rod height can be changed at any time and as many times as necessary. However, it is critical that the person using the total station is notified each time the rod height is changed so that it can be adjusted on the data collector.

If the rod holder forgets to tell the person using the total station that the rod height has been changed, then the error can be written down in the field notes and corrected later.

6.3 TRIPOD:

Several types of tripods that can be used to support the gun, most of which being simple in their design and use. The primary differences between tripods include material, maximum height, and weight. Materials include wood aluminum, and wood/fiberglass, of which aluminum tripods are the lightest (13-14lbs.) and wood/fiberglass ones (16-18 lbs.) are the heaviest and most durable. The kind of tripod used depends on the circumstances of the mapping, including whether the tripod has to be carried for long distances, if conditions are windy, and other factors.

For complex mapping projects, such as ones with multiple mapping stations, more than one tripod is recommended. Having several tripods to work with will allow for more efficiency when occupying multiple mapping stations and using multiple back sight points. The tripods, along with tripod bracket (for leveling purposes), can be set up such that the location of the gun and prism are easily interchangeable.

6.4 MECHANICAL PLUMB BOB:

There is the mechanical plumb bob that can be used for the same purpose. The four components of the mechanical plumb bob are stored along with the gun. One, there is a small metal bar that is shaped like a V with a hook extending from the base of the V. which allows for the plumb bob to be attached to the tripod. The ends of the top of the V are inserted into the base of the tripod screw. Two there is the string that extends between the metal bar and the plumb bob. The string is attached to the hook at the base of the V. Three, there is a metal strip that has two holes in it and is curved on both ends. The string is threaded through the metal strip such that it can be used to adjust the length of the string. Four, there is the actual plumb bob that is attached to the other end of the string and should hang immediately above the marker.

7.0 OPERATIONS OF TOTAL STATION

Setting an Instrument over a Point and Leveling the Instrument

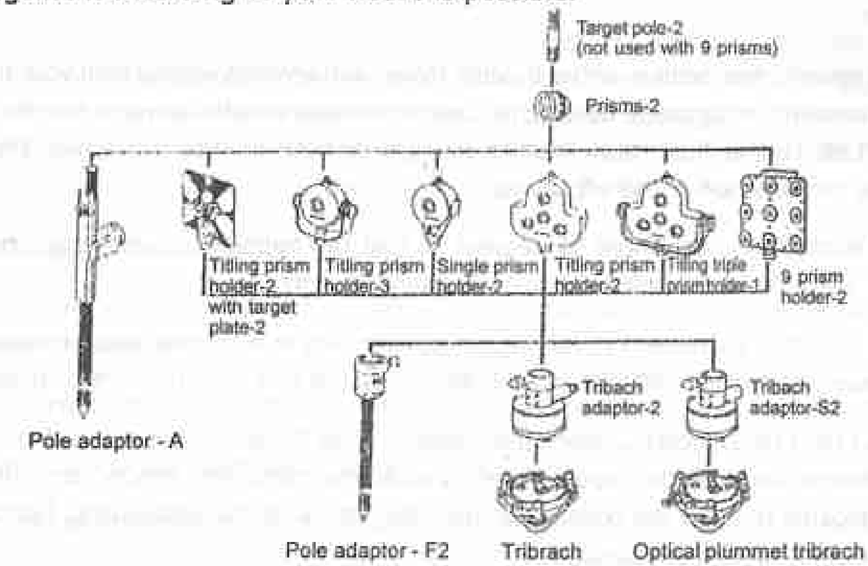
7.1 Roughly set the tripod over the point by holding two legs and set the third past the point. Move the two legs that you are holding until the top of the tripod is approximately over the point. Make sure that the top of the tripod is roughly level . Press the legs of the tripod firmly into the ground. Screw the instrument onto the tripod. Make sure that the leveling screws are even. If they are not even, screw each one of the screws so that it is in the middle of its leg.

7.2 The plumb bob can be used to position the instrument over the point. The optical plummet is quicker and, easier to use than a measured plumb bob. Look through the optical plummet and determine its location relative to the point (you sometimes have to place your foot next to the point to determine its location). Adjust the location of the tripod (if necessary) to find the point in the optical plummet by lifting two legs and pivoting he instrument on the third leg while looking through the optical plummet. Centre the optical plummet exactly on the point.

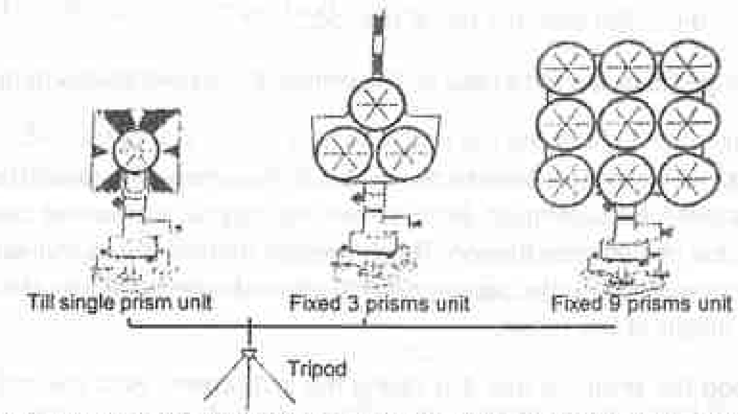
7.3 Level the tripod base. Carefully adjust the tripod legs (one at a time) up or down to centre the circular level on the instrument. Check that the optical plummet is still on the point.

13. PRISM SYSTEM

Arrangement according to your needs is possible.



It is possible to change the combination according prupose.



Use the above prism after setting them at the same height as the instruments. To adjust the height of prism set, change the position of 4 fixing screws.

7.4 Level the instrument plate bubble using the plate level.

7.4.1 Align the level so that its axis is parallel to the line between two of the leveling screws.

7.4.2 observe the location of the bubble. Move both leveling screws with your thumbs equal amounts in opposite directions. Centre the level exactly between the two large lines.*Left Thumb Rule: Both thumbs move in or both thumbs move out. The level follows the direction of the left thumb.

7.4.3 Rotate the instrument 90 degrees so that the bubble is aligned over the third screw. Use that screw only to centre the bubble exactly.

7.4.4 Turn the instrument to its original position 9as in a.). Level again if necessary. Continue this process until the bubble stays centered as the instrument is rotated.

7.4.5 Check the optical plummet. If it is slightly off of the point, loosen the instrument's attachment screw on the tripod. Carefully slide the instrument exactly over the point while looking through the optical plummet. Do not rotate the instrument. Tighten the clamp. Then re-level the plate level.

7.5 Assembling and Holding the Prism (Prism Boy)

7.5.1 Attach the prism onto the top of the rod.

7.5.2 Place the tip of the rod's base on the centre of the point that is to be measured.

7.5.3 Usually the height of the rod should stay at 1.5 m. This is the distance from the tip of the rod that is on the point to the centre of the prism. The lower the rod is, the more accurate the measurement. If the person running the instrument cannot see the prism, then you should raise the rod. Tell the person running the instrument the height of the rod or target (HT). the person running the instrument will use the HOT key to change the height of the target.

7.5.4 Position the prism so that it is facing the instrument. Hold the rod so that the circle bubble is centered.

7.5.5 Keep the prism in position until the person running the instrument says "Okay" or "Good."

Sighting a Point

Unlock the vertical and upper clamps. Sight in the reflector so that the crosshairs on the telescope are in the centre of the prism. To do this, use the optical sight(Finder). Align the optical sight arrow with the prism rod or prism. Lock the upper clamp. Look through the telescope and find the prism. Lock the vertical clamp. Use the fine-tuning knobs on the ends of the clamps to centre the crosshairs on the centre of the prism.

7.6 Using the Total Station

The following is a step-by step procedure of how to use the total station after the instrument is leveled.

7.6.1 Turn on the Total station. Press the [power] button. The person assigned to instrument setup will perform this step.

7.6.2 Create the Job. Press the [menu] key. Press [1] Job. Press the [menu] key. Press [1] create. Input Job name (for example: wal-group =5409). Press the [enter] key.

7.6.3 Set the Horizontal Angle to 0.sight in a point along the N-S baseline looking north. Sight the crosshairs on the bottom of the rod. Lock the horizontal angle (use the upper clamp0. Press the [ANG] KEY. Press [1] key for Horizontal Angle 0-set. The horizontal angle is now 0.

7.6.4.Instrument Station Setup From a known Point. A point is assigned at the site that the total station will be set. The coordinates of this point will be known in relation to the site datum. The site datum's coordinates are x:1000 m, Y:1000 m, Z:100 m. The instrument station must be set up before any data points (shots) can be recorded.

Press the [STN] key to display the menu screen.

Press the [1] key for "known" coordinates. A screen is displayed for input of the station point information (ST:). The station point is the point that the instrument is over. Input "1"for the station name and the coordinates of the point. If the point number/ name has already been recorded its coordinates are displayed. If the input point is new, the screen waits for the input of coordinates of the point that the instrument station is set on in relation to the site datum. We will know these coordinates . Input the height of the instrument (HI:). Measure (in m) the height of the instrument from the ground to the center of the eyepiece. Input the code (CD:) "ST" for the station point. Press the [ENT] key. The screen changes to the menu display for methods of setting the back sight point.

The screen menu still displays methods of setting the back sight point. Press the [1] key to select "coordinates ." Input the back sight point name (PT:). The back sight point name for the point that we use for the excavation's back sight point will always be the next point after the station point (or 2). Input the height of the target (HT). The height of the target is usually 1.5 m, but this might change. Input the code (CD:). The code is always "BS" for back sight. Enter the coordinates of the back sight point. Press the [ENT] key. Set the prism on the back sight point and sight in the prism. Press the [ENT] key.

The station is recorded and the HA (horizontal azimuth) is calculated between the two points from the input coordinates. The BMS(Basic Measurement Screen) is displayed but no coordinates are displayed. Now all of the points measured will be referenced to the site datum.

7.7 Perform the Back sight Check. To make sure that the total station is on the grid system, take a measurement of a known point (see#7 for measurements).If the measurement is correct then record the shot as the next consecutive point number after the back sight and the code BAC. If the measurement is wrong then the total station should be re-set up.

7.8 Shoot the Laser Level. The laser level will be tied to the site datum (we will calculate the distance below the datum) The X, Y, Z will be recorded relative to the datum. The total station setup will measure and record the laser level before any of the measurements (or shots) are taken and before the laser level is turned on. The Z

of the laser level is set to 1001.13. This is the height of ;the laser level relative to the height of the site datum. The Prism Boy places the center of the prism exactly on the location what the laser is emitted. The height of the target is now O, so the person running the instrument will use the HOT key to change the height of the target. The laser level must be set to 1001.13, so the Prism Boy must move the laser level up or down then wait for the person running the instrument to measure and read the Z coordinates. The rod person adjusts the laser level, and the person running the instrument will measure the elevation (Z) unit it is 1001.13. The coordinates of the laser level are then recorded. The person assigned to the total station setup performs this task. The code for the laser level is LL. The total station is now ready to take measurements.

7.9 Taking Measurements (Shooting a point, taking a shot, recording a shot/ point)

Take the Measurement: Make sure that the X,Y,Z are displayed on the BMS menu. If they are not displayed then press the (DSP) key until they are displayed. Have the Prism Boy position the prism on the point. Sight in the center of the prism on the crosshairs. If the height of the target is not 1.5m, press the (HOT) key. Select HT and input the height of the target that the rod person reads to you from the prism rod. Press the (MSR) key.

Record the Measurement: Write the X,Y,Z coordinates on the EDM form. Press the (REC) key to record the measurement into the specified job's database. Raw and coordinate (XYZ) data are recorded. The point name input screen is displayed. Enter the point information for the point name (PT), such as "8" Enter the height of the target (HT:) (a.k.a. height of the rod that the prism is on, usually 1.5m). Enter the code (CD:). Use the code list to identify what the point item is. Enter the code for that item this information onto the EDM form. Write any comments about the shot on the EDM form that might help you when mapping. **Be sure to press ENTER.**

7.10 Putting the Instrument to Sleep or Turning the Instrument Off

Sleep: After taking a measurement, and if no one is waiting to use the instrument, put the instrument to sleep. Press the PWR) key, then press the (4) key to put the station point information (ST:). The station point is the point that the instrument is over. Input "1"for the station name and the coordinates of the point. If the point number/ name has already been recorded its coordinates are displayed. If the input point is new, the screen waits for the input of coordinates of the point that the instrument station is set on in relation to the site datum. We will know these coordinates . Input the height of the instrument (HI:). Measure (in m) the height of the instrument from the ground to the center of the eyepiece. Input the code (CD:) "ST" for the station point. Press the [ENT] key. The screen changes to the menu display for methods of setting the back sight point.

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The station is recorded and the HA (horizontal azimuth) is calculated between the two points from the input coordinates. The BMS(Basic Measurement Screen) is displayed but no coordinates are displayed. Now all of the points measured will be referenced to the site datum.

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7.10 Putting the Instrument to Sleep or Turning the Instrument Off

Sleep: After taking a measurement, and if no one is waiting to use the instrument, put the instrument to sleep. Press the PWR) key, then press the (4) key to put the station to sleep. Make sure that the upper and vertical clamps are unlocked and the lens cap is on the eyepiece.

Off: To turn the instrument off, press the (PWR) key, then press the (ENT) key. This will same all of the instrument settings.

8.0 FUNCTIONS OF TOTAL STATION:

a. It simultaneously measures the angle, distance and record with the help of EDM, Theodolite, and micro processor respectively. Recording facility saves lot of time and creates facility to store the data for long time.

b. Correcting distances with the following factors instantaneously:

i. Prism constant: It is the distance between the centre of prism rod on vertical axis and the point of reflection of laser beam on the cubiodal prism. Most of the prisms having prism constant from 1cm to 1.3cms and depends on the manufactures.

ii. Atmospheric and temperature constant: The atmospheric pressure and temperature at the instrument and at the place of prism may not be same when the survey is at hill ranges. In case, both are fed to total station before shooting the point, it will automatically correct forth coming distances with the help of pre programme fed to micro processor. Similarly, correction required due to curvature of earth and refraction.

c) Computing the point elevation: With the help of the trigonometrical equations and data collected through EDM, Total Station can find elevation of any point (in the same vertical plane) like towers, pillars, building heights etc.,

d) computing the coordinates at every point: Total Station can generate Northing, Easting and elevation of every point where the prism reflector is placed with reference to the known point coordinates and datum.

e) Remote Elevation Measurement (REM): The prism reflector is set directly below the place to be measured and by measuring the prism height, the height of the target can be found out. This makes us easy to determine the height of electrical power lines, suspension bridge cables and other large items used in construction.

f) Remote Distance Measurement (RDM): Horizontal distance, slope distance, difference in height and percentage of slope between the reference point and the observations point are measured. In a particular traverse, a missing line measurement can also the made with this function.

g) It has the Data transfer facility from the total station instrument to the computer software (compatible) and vise versa . The point whose coordinates are known but the location at field is not know, then this **stakeout** technology in Total Station will be useful to identify the point location at field. This will help the irrigation Engineers to set a curve in canal alignment within reasonable time. By way of traditional procedure it takes longertime.

h) Conversion of units: With the help of micro processor, the units can be changed (from MKS to FPS or fromEPS to MKS) without much effort.

i) Recection: when the coordinates of occupied point is not know and occasionally there are two points whose coordinates are available near by, then Total Station can give coordinates of such occupied point.

9.0 ACCURACY AND RANGE TOTAL STATION:

Angular accuracy of Total Station =0.5 sec to 7 sec (for the recent instrument models) otherwise it is 1 to 20 sec.

Linear accuracy of Total Station varies from 1mm to 10mm per kms.

Range of total station with	Single Prism is upto 2.50 kms
	Two Prisms – up to 5 to 7 kms
	Three prisms – up to 10 to 12 kms
	Nine Prisms - up to 100 km

10.0 COMPUTER PACKAGES:

The following post processing computer software packages are in use for various Engineering applications

1. Arc Pad 2. Arc view 3.Arc info 4. Micro station, 5 Erdas 6. Surfer 7. Auto plotter 8. Civil CAD 3D 9. Phrhagutas 10. Survey aid etc.,

11.0 LIMITATIONS OF TOTAL STATION:

a. It is not a Rugged instrument: It is a sensitive electronics operated machine and it shall be handled carefully and it is to be calibrated at every six months.

b. Prism verticality is questionable: The attitude of the prism boy plays a major role in surveying with Total station. Enough training shall be given and an awareness shall be created about the importance of prism verticality.

c. Visibility is must: There are so many apprehensions prevailing in the market that Total Station can do any thing. It can not measure the distance from the occupied point to an invisible point.

d. More expensive: The cost of the instrument varies from Rs.5.0 Lakhs to Rs.30.0 Lakhs. As the cost of instrument is r ore when compared to the salary of departmental Engineers, They have the fear of handling.

f. Amount of error is greater at short distances: Keeping in view of the constrains of timer placed in the total station, amount of error is greater at short distances. Experienced surveyors says that it is not advisable to measure the distance below 30m.

g. Height of instrument and height of prism is to fed manually: This one of the disadvantage with Total Station. It may lead to malpractices or wrong entries.

h. Awareness on battery maintenance shall be known to the persons handling the total station.

12.0 GENERAL HANDING PRECAUTIONS OF TOTAL STATION:

a. Before starting work or operation, be sure to check that the instrument is functioning correctly with normal performance.

b. Do not aim the instrument directly into the sun: Aiming the instrument directly into the sun can result in serious damage to the eyes. Damage to the instrument could also result from exposing the instrument's objective lens to direct sunlight. The use of a solar filter is suggested to alleviate this problem.

c. Setting the instrument on a tripod: When mounting the instrument on a tripod, use a wooden tripod when possible. The vibrations that may occur when using a metallic tripod can affect the measuring precision.

d. Installing the tribrach: If the tribrach is installed incorrectly, the measuring precision could be effected. Occasionally check the adjusting screws on the tribrach. Make sure the base fixing lever is locked and the base fixing screws are tightened.

e. Guarding the instrument against shocks: When transporting the instrument, provide some protection to minimize risk of shocks. Heavy shocks may cause the measurement to be faculty.

f. Carrying the instrument: Always carry the instrument by its handgrip.

g. Exposing the instrument to extreme heat: Do not leave the instrument in extreme heat for longer than necessary. It could adversely affect its performance.

h. Sudden changes of temperature: Any sudden change of temperature to the instrument or prism may result in a reduction of measuring distance range, i.e when taking the instrument out from heated vehicle. Let instrument acclimate itself to ambient temperature.

i. Battery level check: Confirm battery level remaining before operating.

j. Memory backup: The back-up battery built in the instrument needs to be charged approximately 24 hrs. before using it for the first time after purchase. Connect the fully charged battery to the instrument in order to change the back-up battery.

k. Taking the battery out: Leaving the instrument without the battery for more than an hour will cause the mermorize data to be lost, due to low voltage of the back-up battery. Connect the battery as soon as possible or execute RAM back-up.

l. No responsibility: of the seller or manufacturer for loss of data stored in the memory in case unexpected accidents.

m. Battery Cover: Completely close the battery cover before using. If the battery cover is not completely closed, instrument will not operate normally, regardless of whether the battery cover is opened while in operation, operation will automatically be us pended.

n. Power OFF: When turning off the Power, be sure to turn off the power switch.

o. Do not turn off the power by removing the battery.

p. Before removing the battery, press the power switch and confirm that the power is off. Then remove the battery.

q. While using the external power source, do not turn off with the switch on the external power source.

If the above-mentioned operating procedure is not followed, then the next time that power is turned on, it will be necessary to reboot the instrument.

13. **WHILE SUPERVISING THE TOTAL STATION WORK:**

Follow items mentioned below carefully.

a. Have keen observation on the prism boy's attitude. When his precision is required, use the prism tripod, to avoid human error.

b. The position of prism shall always be on hard surface instead of soft soil.

c. Focusing shall be exactly at the centre of prism, with the help of cross hairs and prism plate.

d. While measuring the instrument height and prism height, enough attention shall be diverted. Also, have keen observation while entering the above data.

e. Note the location and coordinates of station point and back sight so that specified intermediate points can be checked.

14. **SOURCES OF ERRORS IN TOTAL STATION:**

a. Instrumental errors: Error caused imperfection in instrument.

b. Natural errors: Errors due to changing condition in surrounding environment.

c. Human errors: Errors due to limitations in human senses

d. Random errors: Like Placement of instrument targeting the prism, placing prism tip, circle reading etc.,

7. C Highways

7.5 **ROADS (I.R.C. STANDARD)**

7.5.1. Classes of Highways:

1. National Highways (N.H.) 2. State Highways (S.H.)
3. Major District Road (M.D.R.) 4. Other District Roads (O.D.R.)
5. Village Roads (V.R.)

7.5.2 Design capacity and No. of Traffic Lanes:

Traffic Classification	Very light	Light	Medium A	Medium B	Heavy	Very Heavy A	Very Heavy B	Very Heavy C
Average Daily Tonnage	upto 200	201 to 500	501 to 750	751 to 1000	1001 to 1500	1501 to 2500	2501 to 5000	Over 5000
No. of Traffic Lanes		1	1	1	1	2* or lane divided	4* or Lane divided	6**

* In addition 9ft. wide Cycle track on each side where required.

** In addition 9ft. wide Cycle track and 9ft. wide foot path where required.

7.5.3 Width of carriageway and roadway for various classes of roads:

1. Width of carriageway

A-Roads	Single Lane	Two lanes without raised kerbs	Two lanes with raised kerbs	Multi lane pavements per lane
a. National and State Highways	3.75m	7.0m	7.5m	3.5m
b. Major District Roads	3.70m	-	-	-
c. Other District Roads	3.75m	-	-	-
d. Village Roads	3.75m	-	-	-

2. Width of roadway for roads other than in hilly terrain

a.	National and State Highways	12.00m
b	Major District Roads	9.00m
c.	Other District Roads	9.00m
d.	Village Roads	7.50.

i. For multi-lane carriageways, the width of roadway will be governed by factors such as central verge, footpaths, cycle tracks, etc., and width of roadway is to be designed according to the requirements in each case.

ii. In case of State Highways having a single lane carriageway, the width of roadway could be reduced in 9m provided that the possiblitity of widening the carriageway to 2 lane is very remote.

3. Clear width of carriageway on road bridges

a. Single-lane bridges 4.25m

b. Two-lane bridges 7.50m

c. Bridges of divided highways with four-lane dual carriageway (2 lanes on either side of the central dividing verge) 7.50m (for each two lane carriageway)

7.5.4 Road camber or cross fall:

a.	Water bound macadum road	1 in 48 1 in 36 in heavy rain fall regions.
b.	Black topped road	1 in 50 (2%)
c.	Concrete road	1 in 72 (1.4%)
d.	Shoulders	1 in 33 (3%)
e.	Earth roads	3 to 4 percent
f.	Gravel or wbm surface	2.5 to 3 percent
g.	Thin bituminous surfacing	2.5 to 2 percent
h.	High type bituminous surfacing or cement concrete surfacing	2.0 to 1.70 percent

7.5.5	Gradient (maximum):	Rulling	Limiting	Exceptional*
a.	Flat and Rolling terrain	1 in 30	1 in 20	1 in 15
b.	Hilly terrain upto 300m above MSL	1 in 20	1 in 16	1 in 14
c.	Steeps over 300m above MSL	1 in 20	1 in 15	1 in 12

* For short distance not exceeding 60 m in a km (300ft in a mile)

7.5.6 Sight Distance in Metres:

		Over taking	Safe stopping (for Minimum Design Speed	Safe stopping (for ruling Design Speed)
a)	Plain terrian	475 m	120 m	170 m
b)	Rolling terrian	310 m	90 m	120 m
c)	Hilly terrain (Mountainous)	-	50 m	60 m
d)	Hilly terrian steep	-	40 m	50 m

Superelevation:

Should normally be 1 in 225 R/ V^2 or $e = V^2/ 225\ R$

Where R-Radius in Metres, V-Design Speed in Kmph, e= Superelevation in m (with Maximum of : i) Plain & Rolling terrain 1 in 15 (6.7%) ii) Hilly terrain 1 in 10 (10%)

7.5.7 Transition: Minimum length of transition

Design Speed in Kmph	30	40	50	65	80
Min. Length of transition CUrve (m)	16	16	16	23	30

7.5.8 Radii for Horizontal Curves in Metres:

		Absolute (min)	Ruling (min)
a)	Plain terrain	250	370
b)	Rolling terrain	155	250
c)	Hilly terrain (Mountainous)	50	80
d)	Hilly terrain (steep)	30	50

MINIMUM CURVE RADII - HILL ROADS

Radius	Mountainous terrain		Steep terrain	
Highways Classification	Areas not Affected by snow	Snow bound Areas	Areas not affected by snow	Snow bound areas
a) N.H. and S.H.	50m	60m	30m	33m
b) Major District Road	30m	33m	14m	15m
c) Other District Road	20m	23m	14m	15m
d) Village Road	14m	15m	14m	15m

Note:The minimum radii have been calculated from the formulae: $e = v^2/225R$ or, $R=0.008V^2/(e \times f)$ Where R is the minimum radius in meters.

v= the design velocity in kilometres per hour : e= The roadway superelevation (0.07 for snow-bound areas and 0.10 for areas not affected by snow) and f= the lateral friction factor assumed as equal to 0.15.

7.5.9 Extra width of carriage way in Curves:

Radius of Curves in Metres Upto	60	61-150	151-300	Over 300
Extra width in Metres	0.9	0.6	0.3	Nil

7.6 Design Speed

The following design speeds should be adopted for various categories of roads:

Terrain Classification	Design speed and classification	Speed in kms. per hour				
		NH	SH	Mdr	Odr	vr
1)Plain terrain	a) Rulling design speed	100	100	80	65	50
	b) Minimum design speed	80	80	65	50	40
2)Rolling terrain	a) Ruling design speed	80	80	65	50	40
	b) Minimum design speed	65	65	50	40	35
3)Mountainous	a) Ruling design speed	50	50	40	30	25
	b) Minimum design speed	40	40	30	25	20
4)Steep	a) Ruling designspeed	40	40	30	25	25
	b) Minimum design speed	30	30	20	20	20

7.7 HAIR-PIN BENDS

i) A hair-pin bend may be designed as a circular curve with transition curves at each end. Alternatively, compound circular curves may be provided.

ii) The following design criteria should be adopted for the planning of hairpin bends

a) Max. design speed — 20 km.p.h.

b) Min. width at apex
- i) National and State highway — Double lane 11.5m, Single lane 9.0m
- ii) Major district roads & other district roads — 7.5m
- iii) Village roads — 6.5m

c) Minimum radius for the inner curve — 14.0m

d) Minimum length of the transition — 15.0m

e) Gradient — Max 1 in 40; Min 1 in 200

f) Superelevation — 1 in 10

Minimum vertical clearance = 5 metres from the highest point of carriage way.

7.8. HORIZONTAL CURVES:

(1) Length of transition curve (should be **higher** of the following two values).

a) By rate of change of centrifugal acceleration

L = $0.0215\ V^3/CR$

where L = Length of transition curve in metres

V = Design speed in kmph

R = Radius of circular curve in metres

C = Allowable rate of centrifugal acceleration in M/sec.2

c = 80/75-v (subject to maximum of 0.8 and minimum of 0.5)

b) By rate of change of Superelevation.

L = $2.7V^2/R$ For plain and Rolling Terrain

L = $1.0V^2/R$ For Mountainous and steep terrain

(2) Extra width of pavement at Horizontal curves

Radius of Curve(m)	upto 20	21 to 40	41 to 60	61 to 100	101 to 200	above 300
Extra width(m)						
Two-lane	1.5	1.5	1.2	0.9	0.6	nil
Single lane	0.9	0.6	0.6	nil	nil	nil

(3) Shift of the transit curve : $S = L^2 / 24R$

(4) Total tangent length : $T = (R + S)\tan\theta + L/2$
Where θ is deviation angle.

(5) Total curve length : $K = (R\theta / 180) + L$

(6) Length of circular curve : $c = K - 2L$

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7.8.1 WIDENING AT CURVES

Extra width of carriageway required at curves is given by the following formula :

$W_c = (n \times 18/RC) + 0.1\,V/\sqrt{Rc}$

Where Wc = Extra width in metres V= design speed in km.per hour

Rc= design radius in metres n= number of lanes

Considering the design speeds and minimum radii for various classes of roads, it is recommended that the pavement should be widened at curves by the following amounts:

a) Single lane roads:

Radius of curve	Extra width
14 to 20m	1.5m
+20 to 30m	1.2m
+30 to 60m	0.9m
+60 to 150m	0.6m
above 150m	No widening to be done

b) Two lane roads:

Radius of curve	Extra width
+ 30 to 40m	1.5m
+ 40 to 60m	1.2m
+ 60 to 100m	0.9m
+ 100 to 150m	0.6m
above 150m	No widening to be done.

7.9 VERTICAL CURVES

(a) Summit curve (1) Equation of Summit curve :

Summit $Y = X^2/e$ where

Y = Intercept between the grade the origin and the curve in m

X = Horizontal difference from the origin.

e = 2L/N

Where L = Length of summit curve in m

N = Deviation angle equal to algebraic difference between the two grades.

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7.9.1 MINIMUM LENGTH OF VERTICAL CURVES

Design speed	Maximum grade change (% not recuring a vertical curve)	Minimum length of Vertical Curve
30	1.5	15
50	1.0	30
60	0.8	40
80	0.6	50

7.10.0 BRIDGES

7.10.1 Loading

i) Single lane of IRC class "AA" loading checked for 2 lanes of IRC Class 'A' loading (within cetain municipal limits in certain existing OR contemplated industrial areas along certain specified highways.)

ii) I.R.C Class 'A' loading for permanentBridges and **culverts** in other localities.

iii) Temporary Bridges IRC Class 'B'

Axle Load = 8165 Kg

Maximum Permitted wheel load = 4082 Kg

7.10.2 DESIGN PRINCIPLES:

1. A high-level bridge is designed to allow all floods to pass through its vents.

2. A submersible bridge is designed to be over topped in floods but the interruption to traffic during floods greater than or equal to 2 days at a time and greater than or equal to 6 times in a year.

Depth of foundation (inerodible strata) =1.330.1+ proper grip.

where D = max. scour depth.

Depth below scour 2m, for piers & abutments with arches.

Less than or equal to 1.2 m - do- supporting other types of superstructure.

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7.11 Transition curves

Spiral curve should be used for the transitions. The length of the transition required may be calculated from the following formula

LS = 0.0215 V^8/CRc

Where V= design speed in km/hr
Rc = radius of circular curves in metres, and c+ 80/(75+v)
Subject to value of 0.46 for speed above 100 km.p.h. and a value of 0.76 for speeds less than 30 km.p.h.
The minimum length of transition provided shall be as under:

a) 15m. for design speeds upto 40 km./hr.
b) 20m for design speeds 40 to 50 km/hr
Where the deflection angle is small, it may be advantageous to provide a transitional curve all through omitting the small circular arc in between.

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Chapter - 8

GENERAL

I.S. CODES

LIST OF I.S. CODES FOR DESIGNS OF IRRIGATION COMPONENTS

I S CODES FOR SPILLWAY, NON-OVERFLOW DAMS

S.No.	COMPONENT	I.S.CODE No.
1	Reservoir capacity-Maximum flood discharge SPF/PMF/5OO/1OOO/year flood and fixing of spillway capacity and considering gate inoperative condition.	5477-1971 11223-1985
2	Site Selection	6966-1989
3	Permeability in rock below Gravity Dams	11216-1985
4	In-situ Permeability test	5529-1985
5	Selection of Spill ways and dissipaters	10137-1982
6	Foundation treatment –Consolidation Grouting-Pressure grouting	6066-1994
7	Design Criteria of solid Gravity Dams – stability analysis, Load combinations, Loads Factors of safety against sliding Fetch and free board computations	6512-1984
8	Hydraulic design of High Ogee Overflow Spillways Ogee Profile, U/S and D/S Quadrants, computation of Coefficient of discharge, provision of Breast wall etc.,	6934-1998
9	Hydraulic Design of Bucket type Energy dissipater	7365-1985
	(a) Solid Roller (b)Slotted Roller	
	(c)Trajectory Bucket	
10	Hydraulic jump type stilling basins	4997-1968
11	Structural arrangement of energy dissipater for spillways	11527-1985

12	Drainage arrangements of Energy Dissipators,Spacing of contraction joints, size and spacing of half round pipes-details of no fine concrete blocks.	11772-1986
13	Water stops at transverse contraction joints in Spillways, OF dams and at junctions of galleries	12200-2001
14	Construction of Spill ways-preparation of foundations and anchorage, maximum size of aggregates for different components, blocks lengths etc.,	11155-1994
15	Galleries and other openings in dams –drainage gallery, inspection gallery, instrumentation gallery ,adits ,sump well and pump chambers ,stair wells, lift etc., general and structural design	12966-1990 & 1992 parts 1&2
16	Curtain Grouting from galleries –spacing ,depth of grouting ,size and inclination of grout holes-pressure of grouts etc.,	11293-1993 Part-2
17	Shear parameters at interface of foundations of Dam with Rock.In-situ shear test on rocks	7746-1991
18	General construction of plain and R.C.C.for dams	457-1957
19	Aggregates ,Fine and Coarse for concrete	383-1997 (reaffirmed)
20	HYSD bars and wires for concrete Reinforcement	1786-1990 (Reaffirmed)
21	Drainage system for gravity dams, their foundations and abutments	10135-1985
	(a) spacing ,size and depth of drainage holes	
	(b) size of porous concrete and formed drains	
22	Earth Quake resistant design of structures seismic zone, and computations, hydrodynamic forces and values at different elevations	1893 -1984 part-1-2000

23	Dewatering during construction	9759-1981 (Reaffirmed-1998)
24	Extreme weather concreting (1) Hot weather (2) Cold weather	7861-1975 (Reaffirmed-2002)
25	Temperature control of mass concrete for dams	14591-1999
26	Computation of seepage –provision of sump well and pump chamber-size of sump etc.,	4721-2000
27	Radial gates –radius, trunion level etc.,	4623-2000
28 (a)	Instrumentation Location of measuring instruments in concrete & masonry dams.	7436(part-ii) -1997
(b)	Selection, splicing,installationand providing protection to the open ends of cables used for connecting type measuring devices in concrete and masonry dams.	10334-1982
©	Pore pressure measuring devices Electrical Resistance type cell, Vibrating Wire type cell.	8282-1976,1996 (parts-1&2) (1998-Reaffirmed) (2001-Reaffirmed)
(d)	Deformation measuring devices in concrete and masonry dams: part-1 resistance type joint meters.	10434-1982 (part-1)

(e)	Seismic instrumentation for river valley projects	4967-1968 (2000-RA)
(f)	Instruments for temperature measurement inside dams –	6524-1972
	Resistance type thermometers	(1998-RA)
	STRUCTURAL DESIGN	
1	Pier, Crest	13551-1992
2	Bucket(roller/trajectory)	7365-1985 11772-1986
3	Galleries,Adits,Sump well and pump chamber Stair well and Lifts	12966-1990(part-2)
4	Stilling basin	11772-1986
5	Training walls and divide walls	12720-1993
6	Air vent pipes	
7	Walkway bridge-steel Concrete	800-1984 (1998-Reaffirme)- 456-2000
8	Spillway bridge	IRC codes

BIS CODES FOR EARTH DAMS

1. BIS Code no 875-(Part III) 1987and 1893-1984(Reaffirmed 2002 Design Loads other than Earh Quake for Buildings and Structures and Earth quake analysis of the structures

2. BIS Code no 10635-1993/Reaffirmed 1998-Guide Line for Free-board requirements in Embankment dams

3. BIS Code 1498-1970/ Reaffirmed 2002/ Classification and Identification of soils for General Engineering purpose

4. BIS Code no 7894-1975 (Reaffirmed 1997)- Code of practice for stability analysis of earth dams

5. BIS Code 8414-1977 (Reaffirmed 1999) Guide Lines for Design of Under seepage control measures for earth and rock fill dams

6. BIS Code 9429-1999. Code of practice for Drainage system for Erath and Rock fill dams

7. BIS Code no 8237-1985/ (Reaffirmed 1999) Code of practice for Protection of Slope for Reservoir Embankments

8. BIS Code no 7436(Part I)-1993/ (Reaffirmed 1998) Guide for Types of measurements for structures in River Valley projects and criteria for choice and location measuring instruments Part I – For Erath and Rock fill dams

9. BIS Code 7356(Part I) -2002/Reaffirmed 1998- Code of Practice Instrumentation maintenance and observations for pore pressure measurements in Earth dams and Rock fill dams Part I-Porous tube peizometers and Part II Twin tube hydrostatic peizometers

BIS Code no 7500-2000 – Code of Practice for installation and observation of cross drains for measurements of internal vertical movements in Earth dam

Reference Codes / Manuals for L.I.Schemes(Excluding Dry pit Type)

S.No	B.I.S Code Number	Year	Code of practice for
1			BHRA Manual on " Hydraulic design of Pump sumps and intakes"
2			IDC Manual on L.I.Schemes
			Manual on water supply and treatment prepared by the Expert committee, Govt. of India, Ministry of Urban development,
			New Delhi determining the acceptability of Hazen-Williams
3		1999	Coefficient (Value of "C")
4	783	1985	Laying of concrete pipes
5	5822	1994	Code of practice for Laying of electrically welded steel pipes for water supply
6	784	2001	Pre-stressed concrete pipes (Including fittings)
7	458	1988	Per-cast concrete pipes(with and without Reinforcement)
8	456	2000	Plain and Reinforced Concrete
9	1916	1989	Specifications for steel cylinder pipe with concrete lining and coating
10	3589	2001	Specifications for steel pipe for water and sewerage (168.3 to 2540 mm Outside Diameter)
11	11639 (p -1&2)	1986	Criteria for design of surface penstocks
12	800	1984	steel structures
13	822	1970	Code of procedure for inspection of Welds
14	4853	1982	Recommended practice for Radiographic inspection of fusion welded butt joints in steel pipes
15	1182	1983	Recommended practice for Radiographic inspection of fusion welded butt joints in steel plates
16	2062	1999	Steel for general structural purposes - specifications

17	5504	1997	Specifications for spiral welded pipes
18	5330	1984	Design of Anchor or thrust blocks
19	SP 16	1980	Design Aids for IS - 456
20	SP34		Hand Book on concrete reinforcement detailing
21	875(1 - 5)	1987	Dead Loads; Imposed Loads; Wind Loads Snow Loads; Spl and Combination Loads
22	1893(part -1)	2002	Earthquake Resistant Design of Structures -
			General provisions and Buildings
23	2911	1979	Design and construction of Pile Foundations
24	IRC - 78	2000	Standard specifications for road bridges -
			Foundations and Sub-structures
25	2950	1981	Design and construction of Raft foundations
26	6403	1981	Determination of Bearing Capacity of shallow foundations
27	2720(1-41)	1987	Methods of test for soils
28	2131	1981	Methods of SPT for soils
29	8009(p2)	1980	Calculation of settlements of foundations
30	10262	1982	Recommended guidelines for concrete mix design
31	383	1970	Coarse and fine aggregates from natural sources for concrete
32	1786	1985	HYSD Bars and Wires for concrete reinforcement
33	2974 (P -5)		Design and construction of Machine foundations
34	4326	1993	Earthquake Resistant Design and construction of buildings
35	13920	1993	Ductile detailing for R.C.C structures subjected to Seismic forces
36	11908	1988	Recommendations for cement mortar lining for cast iron , mild steel and ductile - iron pipes (Fittings for transportation of water)

LIST OF I.S.CODES ON CANAL ALIGNMENT & DESIGN OF CM & CD WORKS AND OTHER IMPORTANT REFERENCE BOOKS

S.No.	DESCRIPTION	CODE NO / REFERENCE
1	Guide for Planning and layout of Canal System for Irrigation	IS:5968-1987
2	Criteria for Design of Cross Section for Unlined	
	Canals in alluvial soils	IS:7112-1973
3	Code of practice for Design of Cross Section of Lined Canals	IS:4745-1968
4	Code of practice for laying Cement Concrete/ Stone Slab lining on canals	IS:3873-1993
5	Guidelines for Lining of canals in Expansive soils	IS:9451-1994
6	Criteria for design of Hydraulic jump type stilling basins with horizontal and sloping apron.	IS:4997-1968
7	Criteria for hydraulic design of Cross Regulators for canals	IS:7114-1973
8	Guide for location, selection and hydraulic design of Canal Escapes	IS:6936-1992
9	Code of practice for design of cross drainage works : part - I, (General features)	IS:7784(part-I)-1993
10	Code of practice for design of cross drainage works : part - 2, (section -1,Aqueducts)	IS:7784 (part-2/section-1)-1995
11	Code of practice for design of cross drainage works: part -2, (section -2 super passages)	IS:7784 (part-2 / section-2)-2000
12	Code of practice for design of cross drainage works:part-2,	
	(section-3 canal Syphons)	IS:7784 (part2 / Section-3)-1996
13	Code of practice for design of cross drainage works: part-2,	
	(section-4 Level crossings)	IS:7784 (part-2/-section-4)-1999
14	Code of practice for design of cross drainage works:part - 2,	
	(section-5 Syphon Aqueducts)	IS:7784 (part-2/-section-5)-2000
15	Code of practice for laying of concrete pipes	IS:783 - 1985
16	Specification for precast concrete pipes (with & without Reinforcement).	IS:458 - 1988
17	Standard specifications and code of practice for road bridges, section - I, General features of Design	IRC : 5 - 1998
18	Standard specifications and code of practice for road bridges, section - II, Loads and Stresses.	IRC: 6 - 2000

19	Standard specifications & code of practice for road bridges, section -III, Cement Concrete (Plain and Reinforced).	IRC: 21 - 2000
20	Standard specifications & code of practice for road bridges section - IV, Brick, Stone and Block Masonry.	IRC : 40 - 1995
21	Standard specifications & code of practice for road bridges, section - VII, Foundations and Sub structure.	IRC : 78 - 2000
22	Standard specifications and code of practice for road bridges section-IX, Bearings, Part II : Elastomeric Bearings.	IRC : 83(Part II) - 1987
23	Pocket Book for Bridge Engineers 2000	MOST 2000
24	MOST Standard Plans	
	a) Standard plans for solid slab type high way bridges	MOST Standard plans of 1977, Volume - II
	b) Standard plans for high way bridges, Concrete T- Beam Bridges	MOST Standard plans of 1983, Volume - III
	c) Standard plans for 3.0 m to 10m span RCC Solid Slab Super Structure with & without foot paths.	MOST Standard plans of 1991
	d) Standard plans for High way Bridges RCC T-Beam & Slab Super Structure - Span from 10m to 24m with 12m width	MOST Standard plans of 1992
	e) Standard drawings for Road Bridges solid slab Super Structure (15° & 30°) skew span 4m to 10m (with and without foot paths)	MOST Standard plans of 1992
	f) Standard drawings for Road Bridges solid slab Super structure (20°,25° &35°) skew span 4m to 10m (with and without foot paths)	MOST Standard plans of 1992
	g) Standard drawings for Road Bridges solid slab Super structure (22.5°,35°) skew span 4m to 10m (with & without foot paths)	MOST Standard plans of 1996
25	Code of Practice for Concrete Structures for storage of liquids, Part - I (General requirement)	IS: 3370 - (part I)-1965
26	Code of Practice for Concrete Structures storage of liquids, Part - II (Reinforced Concrete Structure)	IS: 3370 - (part II)-1965
27	Code of Practice for Plain and Reinforced concrete	IS: 456 - 2000
28	Code of Practice for Design and Construction of Raft Foundations, Part 1 :Design.	IS:2950 (part I) - 1981

29	Code of Practice for Design and Construction of Pile Foundations, Part I :Concrete.	IS: 2911:Part I : 1979
30	Code of Practice for Design and Construction of Piles,	
	Section 1, Driven cast in situ Concrete Piles	IS: 2911:Part I : Section 1: 1979
31	Code of Practice for Design and Construction of Piles,	
	Section 2 , Bored cast in situ Concrete Piles	IS: 2911:Part I : Section 2: 1979
32	Code of Practice for Design and Construction of Piles,	
	Part 3, Under reamed Piles	IS : 2911 : Part 3 : 1980
32(a)	Code of Practice for Design and Construction of foundations	
	in Soils-Genreal Requirements	IS : 1904-1986 (RA 2000)
33	Design aids for Reinforced Concrete to IS:456 - 1978.	SP - 16 , 1980
34	Hand Book of Concrete Reinforcement and Detailing	SP - 34 (S&T) - 1987
35	Current Practices In Canal design in India	CBIP , Technical Report no - 3
36	Design practices for Unlined incised Canals	CBIP , Technical Report no - 7
37	Manual on Irrigation and Power Channels	CBI - Publication no: 171
38	Design of weirs on permeable foundation	CBI - Publication no: 12
39	Design of Small Dams	USBR
40	Manual on Canal Falls	CW & PC Publication
41	TVA Hand Book	
42	Test Book of Irrigation Manual	W.M.Ellis

LIST OF I.S CODES FOR MECHANICAL COMPONENTS

S.No.	IS:CODE No. YEAR (latest)	DESCRIPTION
1	IS:800-1994	General construction in steel code of practice
2	IS:807-1976	Code of practice for Design,manufacture,erection and testing structural portion of cranes and hoists
3	IS:3177-1999	Code of practice for Electric overhead traveling cranes and Gantry cranes other than steel work cranes
4	IS:4622-1992	Recommendations for fixed wheel gates structural design
5	IS:4623-2000	Recommendations for structural design of Radial gates
6	IS:5620-1985	Recommendations for structural design criteria for low head slide gates
7	IS:13591-1992	Criteria for design of Lifting Beams
8	IS:6938-1989	Code of practice for design of Rope drum and chain Hoists for Hydraulic Gates
9	Is:9349-1986	Recommendations for structural design of medium and high head slide gates
10	IS:10210-1993	Criteria for design of hydraulic hoists for gates
11	IS:11228-1985	Recommendations for design of screw hoist for hydraulic gates
12	IS:11388-1985	Recommendations for design of Trash racks for intakes
13	IS:11855-1986	General requirements for rubber seals for hydraulic gates
14	IS:14177-1994	Guide lines for painting system for hydraulic gates and hoists

1 Materials:- The materials to be adopted are as given the relevant IS codes for the type of gates

2 Welding:- Welding shall conform to IS:816 and other relevant IS codes.

Note:- BIS codes shown are indecative. Latest codes with updated amendments, if any, shall be followed

8.4 BUILDING CONSTRUCTION

8.4.1 BRICK WORK

Damaged Bricks can be used in internal walls that will be plastered.
PLUMB LINE need only be used at the ends of walls and then use a string to get a straight wall between
The plumb line should always fall to the bottom course, not to the brick immediately below.

FOR ORDINARY SMALL HOUSES
4½" brick walls can be used for short stretches of wall.
9" brick walls are adequate for almost all walls.
13½" brick walls are very rarely necessary.

You can build a 4½" brick wall. If the wall is too long and too high it will fall over.

But if the lengths of wall are shorter, they will support and strengthen adjoining walls and not fall over.

So a house with walls that go in and out like this will be strong, will not fall over, and can carry the weight of a roof.

8.4.2 MORTARS -

CEMENT and SAND
1 part of Cement : 8 parts of sand. This sets quickly — Use cement only if nothing else is available

LIME and SAND
1 part of lime : 3 parts of sand. This sets slowly but is strong — This can be used for all types of brick work.

LIME CEMENT and SAND
1 part Cement : 4 parts lime : 14 parts sand
This sets nearly as quickly as cement. — Use this if you need the mortar to set more quickly than lime mortar.

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Chapter - VII

A - Surveying

B - TOTAL STATION

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Cement Paints :
cement paints are supplied in powder form and are available in 5 Kg. container or 25 Kg. and 50 Kg. drums. Cement paint powder and water is mixed as per directions of manufacturer. Only sufficient quantity is mixed at a time to meet an hour's working requirement.

Enamel Paints :
Enamel Paints are of various qualities. Superior quality enamels cost quite high and cheaper inferior quality enamels are also available.
Enamels have hard mirror-like gloss. Top quality gloss enamel is used for exterior use and to withstand rugged conditions such as finishing trucks, transport coaches, machinery, furniture fixtures, shining sign-board, toys e.t.c.

The paints have an average covering capacity of 14-16 - sq.meters per litre and they are available in 20 litre, 4 litre and 1 litre as also 100 ml., 200ml., packing.

Colour Management

Introduction
The term colour management connotes actions involved in Designing, Choosing, Arranging, Placing, Coordinating and Controlling the 'inputs' of colours to achieve desired effective 'outputs' - colour schemes useful to Architects, Artists, Impressionists, Exterior/ Interior Decorators, Graphic Designers, Fashion Designers, Process Colour Printers and painters.
Colour management, therefore, presupposes knowledge of colours and their characteristics, psychological impact and emotional instinct associated with some colours and colour combinations to produce colour schemes acceptable to end users or professionals. The final impact would, however, depend on the physiological perceptions of the target audience.
There are a number of colours, but all of them are blending of three primary colours - Red, Yellow, and Blue, plus varying degrees of Black and white. With green, Purple and Brown, these colours 'the living palette' make up hues that we see in daily life.

PRACTICAL APPLICATION
Choosing colours , be it a fun to some one or far more serious purposes , is by no means a simple task. There is more to choosing an effective colour combination than simply picking the colours that appeal to ones. The Picking is colour management action, for the combination should be logical, based on the scientific relationship of different colours.

their placement or arrangement and co-ordination, turning to harmony for appropriate desired effect.

Select background colour first. For interior painting, pale or pastel colours for walls and ceiling, then pick a darker or vivid colour for the molding and then trim.
For exterior painting of buildings, observe design (front elevation and also other sides). The first selection of colour should be for the largest area i.e. body colour then go for the next largest and so on-contrast colour if felt necessary to give effect to design aspects - columns-bends-boarders-projections-sunken portion, etc.

Limit the number of colours. Whatever may be the combination of colour - two, three, or four, take one dominant colour that sets the tone for the whole colour scheme, the others should be clearly subordinate in hue, value and intensity.

Use natural colours, Natural colours by definition are harmonious.

Use familiar colours. Avoid uncommon colour like Magenta, Cyans, Purple.

All said and done, and clear though an array of colours may appear on a design to some, a great number of factors affect the perception of them, including physiological capacity and immeasurable powers of emotional association and above all, stylish preference. Colours may often have tremendous emotional and personal impact on account of psychological factors, rather than from any scientific application and determining application and determining agreement in reactions to colours is sometimes difficult. However, most people do agree that some colour combinations imply heat and some cold and that some bring forth pleasure and others pain. Red can be among the other things, either a welcome or a warning signal conveying either warmth or danger. The 'Voice' of colour depends largely on the colours that are placed next, to it - the essence of this exercise-COLOUR MANAGEMENT.

8.4.4 SPECIFICATIONS FOR RESIDENTIAL BUILDINGS

8.4.4.1 Foundation & Plinth

Sl.No.	Item	Specifications
1	2	3
1.	Foundations And Plinth	(i) 15 cm to 20 cm thick cement concrete (1:5:10) or lime concrete (with stone or

		brick aggregates) with 40 per cent (1:2to 3) mortar
		(ii) Brick work or Random Rubble (R.R) stone masonry in cement mortar (1:8) or lime mortar (1:2: to 3)

Note:
a) Higher contents of cement or lime to be used only when required under local conditions.
b) Anti- termite treatment of foundation as per IS: 6313be provided in termite -infested areas.
c) Under - reamed piles are recommended in black cotton soil, being effective and economical.
d) Depth of foundations will depend upon local conditions . National Building Code provides for a minimum foundation depth of 50 cm which would be adequate in normal conditions.
e) Plinth height will depend on local conditions like flooring , sub-soil ,water level etc. In well - drained area, the plinth height need not be more than 30cm.

8.4.4.2 WALLS: (i) Brick work (ii) R.R. stone masonry
(iii) precast stone block masonry (usually 20cm thick)
(iv) Timber posts and Ekra walling (with reeds , split bamboo or expanded metal)
(v) Timber posts and planks (vi) Precast RCC posts and 11.5 cm brick nogging.

Note: a) Whenever stone masonry is required due to non - availability of bricks or whenever brick wall thicker than 23cm becomes necessary due to poor quality of bricks , use of precast stone blocks item (iii) is recommended as they would be usually found more economical.

b) In brick work and R.R. masonry , mortar mixes usually used are cement mortar (1:8) for one and two storeyed construction and (1:6) in construction higher than two storeys, or cement lime mortar (1:2:9) or lime mortar (1:2:to 3) . However, the mortar to be actually used will depend on locally available materials and the actual strength of masonry required. The strength of mortar should match the strength of bricks. Thickness of walls and mortar mixes (or strength design of walls) should be worked out as per provisions of National Building Code.

c) For earthquake resistant construction of load- bearing walls structures zones III, IV and V, IS:4326 should be referred to.

8.4.4.3 FLOOR AND ROOF SLABS:
(i) RCC slab M15(1:2:4)
(ii) Precast roofing systems, core units, channel units, doubly curved shells etc.,
(iii) Reinforced brick panels
(iv) Brick tiles over timber joists
(v) Brick jack arches with precast RCC joists
(vi) Stone putty slabs (10to 15cmthick)
(vii) 40 to 50 mm stone slabs (like Agra stone) over precast RCC or wooden joists
(viii)Madras terrace flooring over timber joists
(ix) Sloping roof with A.C / C.G.I sheets Manglore tiles , slates, shingle, etc.

Note:
a) For economy , RCC slabs should be designed as per limit state theory or yield line method and high strength deformed bars should be preferably used as reinforcement . Where roofs are used in heavy rainfall regions they may be preferably laid in a sloping position.
b) Use of item (ii) results in saving of cement and steel and leads to faster work..
c) Items (iii) to (v) can be used where bricks of a good quality are available.
d) Items (vi) to (vii) can be used in Delhi / Rajasthan areas where suitable stone slabs are available
e) Item (viii) is used in certain areas of Karnataka , Kerala and Tamil Nadu.
f) Item (ix) is appropriate in heavy , rainfall regions like Assam , Kashmir, Himachal Pradesh, Eastern and Western coastal regions.
g) Saving in cement and steel can be effected through the adoption of items(ii) to (ix).

8.4.4.4 FLOORING:
(i) 30to 35mm thick CC 1:2:4 flooring to be used generally. Better specifications like mosaic flooring should be confined only to bath and W.C.s of all type of quarters and living and dining rooms of quarters.

(ii) In ground floor , use 75-80 mm thick lean concrete base with lime concrete or CC1:5:10.

8.4.4.5 DOORS AND WINDOWS

A-Frames
(i) Wooden frames - Generally a size of100X60mm is adequate for door frames (This may be increased to 120mm X60mm where fly- proof shutters are also provided) . The size of the frames could be reduced up to 75mmX60mm in the case of windows.
(ii)M.S. sections (sizes generally used are 40X40X6mm T.section or 45X30X5 mm angle iron section.).
(iii) Precast RCC frame (100X75) mm size.

*Note:*a) For wooden frames locally available seasoned timber as deodar , sal, Bijasal, Pillamarudu, Karimarudu, Hollock etc. and other secondary species as per IS :399 - 1964 may be used.

B- Shutters
(i) Panelled or battened and braced shutters for doors and partly or fully glazed shutters for windows made of Deodar , Kail or other locally available secondary species of timber .
(ii) Commercial type black board flush shutters.
(iii) Wood particle board panelled or flush shutters.
(iv) Kitchens may be provided with fly- proof shutters.
(v) External windows to be provided with simple grills made of M.S. rounds and flats.
(vi) Oxidised iron fittings to be generally used. Only in special quarter anodised aluminium fittings may be used.

Note :
a) Shutter thickness: 35mm is usually adequate for shutters of over 800 mm width .Thickness could be reduced to 30 mm for shutters of less width.
b) For secondary species of timber please refer to IS : 399 - 1964.
c) Windows opening area - 10- 12% in hot and dry regions. 16 - 18% in hot and humid regions.

8.4.4.6 WALL FINISHES

a) Internal Surfaces	12 mm thick plaster with cement mortar (1:6) or cement lime mortar (1:2:9) finished with white wash.
b) External Surfaces	(i) Same as above (finished with colour wash) (ii) Cement /lime pointing.

Note :

a) The thickness of plaster on rough surface of brick work may be increased to 16mm.

b) The thickness may be reduced to 10mm for plastering precast masonry blocks.

c) Plastering on external surfaces need not be done in case of R.R. stone masonry and precast masonry blocks.

8.4.4.7, TERRACING OVER FLAT ROOFS

(i) The top of roof slab shall be painted with bitumen.

(ii) On the bitumen painted roof shall be laid 10cm thick mud phuska and 2.5 cm thick mud gobri plaster to be overlaid by flat brick tiles.

If earth suitable for mudphuska is not available use lime concrete terracing 7.5 cm average thickness. Where quality of lime concrete is not satisfactory , it needs to be overlaid by a layer of pressed clay tiles or two layers of country flat tiles or 30 to 35 cm thick 1:2:4CC panels...

8.4.4.8 Rough Estimate for Cost of Buildings

	Name of Item	Percentage of Total cost
1.	Labour	30 to 35
2.	Cost of Material	70 to 65
3.	Foundation upto and including plinth	15 to 20
4.	Superstructure	80 to 85
5.	Second Storey	75 to 85 of 1st storey
6.	Direct and overhead cost :	
	a) Direct on actual work	85
	b) Overhead costs due to establishment supervision, incidentals etc	15
7.	Sanitation and Electrification	
	a) Sanitary and water supply	7 to 9
	b) Electric installations without Fans	7
	c) Electric Fans	5
8.	Cost break-up of different components :	
	a) Earthwork in Excavation and filling	1-2
	b) Foundation Concreting	4 to 6
	c) Damp proof course	2
	d) Brick work	34
	e) Roofing	20
	f) Doors, Windows and Ventilators	16-20
	g) Plastering and Pointing	10
	h) White and colour washing, painting	2-4
	i) Miscellaneous	4-8
9.	Cost of material and labour (Probables)	
	a) Bricks	15
	b) Cement	15
	c) Steel Bars	15
	d) Timber	15
	e) Lime	5
	f) Labour	30
	g) Miscellaneous	5
10.	Material required on plinth area basis for single storey buildings : per square metre of plinth area (Probables)	
	a) Brick	500 Numbers
	b) Cement	1.5 bags
	c) Steel	10 kg

8.4.4.9 BRIEF SPECIFICATION FOR IMPORTANT WORKS IN BUILDING CONSTRUCTION

PART - I. CIVIL WORKS

1. Earth work excavations in all soils for foundation of narrow width.
2. Supplying and filling in foundation and basement with sand
3 Filling basement with excavated soils in layers and consolidation.
4 Cement concrete 1:5:10, using 40mm broken stone jelly for foundation.
5 R.R. masonry in C.M. 1:5 for foundation and basement with bond stones and simultaneous flush pointing in C.M. 1:5
6. Brick work in C.M 1:3/1:5/1:6 for foundation and basement.
7. Damp proof course in C.M.1:3/1:4 mixed with water proofing compound at 2% by weight of cement.
8. Brick work in C.M.1:5/1:6, using ....Bricks of size.......for superstructure in Ground Floor/First Floor /Second Floor etc.
9. Supplying and erecting shuttering for RCC plane surfaces such as column footings, plinth beam etc.in foundation &basement.
10. Supplying and erecting centering for sides and soffits including structuring upto 3.29m
    a) for RCC floor slabs,rectangular beams lintels, bed blocks, staircase waist slabs, canopy etc.
    b) For vertical slabs, concrete walls, RCC columns ,sunshades etc.
    c) For curved column surfaces including circular columns.
11. Cement Concrete 1:2:4,using 20mm size broken stone jelly for all RCCworks.
12. Plain Concrete 1:2:4/1:3:6 using 20mm size broken stone jelly.
13. Supplying, fabricating and placing in position MS/High strength steel grills for all RCCworks.
14. Supplying and fixing TW/C.W. frames for doors, windows and all ventilators.
    - Single shutter /two shutters with steel grills /without steel grills etc.
15. Supplying and fixing shutters for doors, windows and ventilators including labour, wrought and put up.
    - with multipanel glazed shutter panel/ - with single panel glazed shutter
    - with 3 panel glazed shutter /- with TW/C.W/Andaman padauk fly proof shutters
    - with vertical glazed louvers for ventilators.
16. Supplying and fixing TW/C.W weldmesh doors /windows /ventilators.
17. Floor with cuddapah slabs of 40mm thick, over 20mm cm 1:3.
18. Cement concrete 1:5:10, using 40mm hard broken stone for flooring.

19. Weathering course in brick jelly 20mm size lime concrete in-pure slaked lime (lime to brick jelly in the proportion of 12.5:32).
20. Paving the floor with mosaic by hydraulic pressed square tiles over a base layer of C.M.1:3< 20mm thick.
21. Finishing the flooring with mosaic in- situ over a base layer of C.M. 1:3,20mm thick and top layer of 12mm thick mosaic in situ marble chips.
22. Dadooing walls with mosaic in- situ work with a base layer of C.M.1:3:,4mm thick and a top layer of 8mm thick mosaic in situ, using marble chips.
23. Providing granolithic floor finish with plain concrete 1:2:4, using hard broken stone of 10mm and 12mm size ;-
    -20mm thick for residence /-25mm thick for offices /-40mm thick for workshop &godowns.
24. Finishing the top of roof with one course of machine pressed tiles of size 200mmx200mmx200mm in C.M. 1:3 with water proofing compound at 2%by weight of cement.
25. Dadooing the walls with glazed square tiles of 150mmx150mmx5mm in C.M. 1:2 and pointing with C.M.1:2.
26. Brick partition wall of thickness 11.5cm, using Country Bricks/Stock bricks/ I/II Class bricks in C.M. 1:3.
27. Plastering the surfaces of walls with C.M. 1:5, 12 mm thick in brick masonry/ 20mm thick in RR masonry.
28. Special ceiling plastering and finishing the exposed surfaces of RCC works in C.M, 1:3; 10mm thick.
29. White washing with freshly burnt shell lime-with one coat/- two coats.
30. Colour washing two coats with freshly slaked shell lime.
31. Supplying and painting the walls with two coats of Cement paint.
32. supplying and painting the Ceiling or walls with two coats of synthetic enamel paint/ plastic emulsion paint of approved quality.
33. Painting new work with two coats of synthetic enamel paint of approved quality.
34. Painting new Iron work with two coats of synthetic enamel paint of approved quality.
35. Providing and fixing AC/PVC Downwall pipes with fixtures.
36. TW/CW wrought and put up for AC sheet roofing.
37. Precast RC slab 20/25mm thick using 10mm & below hard broken jelly.
38. Precast RC slab 40/50/75 mm thick, using 20mm hard broken jelly.
39. Supplying and fixing MS grills for doors, windows & ventilators.
40. Supplying and fixing TW hand rail with size of 150 X 50mm.

8.4.4.10 Stripping time for Reinforced Concrete

(As per IS : 456-1978 (Clause 10.3))

Note : 1 In formal circumstances and where ordinary Portland cement is used, forms may be generally be removed after the expiry of the following periods :

a)	Wall, columns and vertical faces of all structural members	24 to 48 hours as may be decided by the engineer - in charge.
b)	Slabs (props left under)	3 days
c)	Beam soffits (props left under)	7 days
d)	Removal of props under slabs	
1)	Spanning upto 4.50 m	7 days
2)	Spanning over 4.50 m	14 days
e)	Removal of props under beams and arches :	
1)	Spanning upto 6 m	14 days
2)	Spanning over 6 m	21 days

For other cements, the stripping time recommended for ordinary Portland Cement may be suitably modified.

Note : 2 The number of props left under, their sizes and disposition shall be such as to be able to safely carry the full dead load of the slab, beam or arch as the case may be together with any other live load likely to occur during curing or further construction.

8.4.4.11 Water Supply requirements for buildings : (As per part IV of NBC 1983)

Water Supply for residences : The requirements regarding water supply, drainage and sanitation for residences shall assume that a minimum water supply of 200 litres per head per day is assured together with full flushing system.

Note : The minimum value of water supply given as 200 litres per head per day may be reduced to 135 litres per head per day for houses for Lower Income Group (LIG) and Economically Weaker Sections of Society (EWS) depending upon prevailing conditions.

8.4.4.12 Occupant Load - (Para 7.3 - Part IV of N.B.C - 1983.)

For determining the exact quantity required, the number of persons within any floor area or the occupant load shall be based on the actual number of occupants, but in no case

less than that specified below

Group of Occupancy	Occupant Load Floor Area in m^2/person.
Residential	12.5
Educational	4
Institutional	15

8.4.4.13 USEFUL NOTES FOR LAYING OF RCC COLUMNS AND FOOTINGS

I. EXCAVATION OF TRENCH

1. A trench for the footing of size given in the drawing providing 15cm extra on four sides to get required size of footing at the bottom of trench shall be excavated.
2. The depth of the pit should be minimum 1m and should be decided by the consulting engineer if the hard strata is not met with in 1m depth.
3. Any deviations regarding the depth of pit should be referred to the consulting engineer.

II. BASE CONCRETE

4. All the loose materials at the bottom of the pit should be removed before laying the base concrete.
5. The base concrete should be of 23 cm thick with c:c (1:4:8) using 40mm size down grade metal.
6. The base concrete should be tamped well to get dense concrete.

III. FOOTING CONCRETE

7. The footing concrete mix shall be V.R.C.C (1:1½:3) (M 15) using 20mm hard broken graded metal.
8. Cement slurry should be put over the base concrete before laying footing concrete.
9. Minimum cover of 5cm should be provided for the reinforcement of footing.

10. Pin vibrator must be used for vibrating the concrete of more than 10 cm.
11. Machine mixing should be done.
12. Minimum 4 cubes of 15 cm X 15 cm X 15 cm should be extracted during concrete and should be got tested for 14 days and 21 days strength for every days work and record shall be maintained.

IV. STEEL

13. Tor steel grade 40 shall be used unless specified in the drawing.
14. The steel should be free from rust.
15. Correct dia of steel should be used with correct spacing and cover as specified in the drawing.
16. The reinforcement bars should be tied properly.

V. SAND, CEMENT AND METAL

17. The cement should be fresh without lumps, cement more than 3 months old should be got tested for its strength. Water cement ratio is maintained for workability of the mix and the wet mix should be placed with in 30 minutes to obtain maximum strength.
18. Approved brand cement shall be used.
19. The 20mm size metal should be crusher metal and should be free from dust.
20. The sand should be free from dust, inorganic materials and pebbles.
21. The sand shall be screened for removing over size materials.
22. Coarse sand only shall be used for concrete work.

VI. COLUMN

24. The concrete mix for the column shall be as specified in the drawing.
25. Steel form work should be used
26. The over lapping length for column bars should be 35D where D is the dia of the

bars. The column bars should be vertical.

27. The leg of the column bar should be minimum 45 cm and should be tied to the footing mat.
28. Minimum 4cm cover should be provided for reinforcement.
29. The column should be vertical through out the floors and no eccentricity is allowed.
30. Pin vibrator must be used to vibrate the concrete.
31. Minimum 4 cubes of 15 cm X 15 cm X 15 cm size should be extracted during concrete and should be got tested for 14 days and 21 days strength for each days work and record shall be maintained.

8.4.4.14 USEFUL NOTES FOR LAYING OF R.C.C. SLAB

I. FORM WORK

1. Steel form work is advised for neat and level ceiling.
2. The form work shall be strong and should withstand the impact loading of concrete. All the holes in the form work shall be closed to avoid possible bleeding of concrete during vibration.
3. The centering shall be removed only after the concrete attains the desired strength.

II. PRECAUTIONS

4. The column levels shall be at the same level (or lower) with respect to the bottom of the beam of maximum depth.
5. All the column junctions are cleaned and no oil should be seen on column surface.

III. STEEL

6. 12.5mm clear cover for slab reinforcement and 25mm cover for beam reinforcement shall be maintained.
7. The steel should be fresh and free from rust and correct diameter of steel shall be used.

8. Overlapping length shall be 45 times the dia of the reinforcement.

9. For the rods of dia above 6mm the TOR steel of grade 40 and the bars of 6mm dia shall be in MS grade.

10. All the reinforcement steel shall be tied effectively with binding wire at the joints.

11. The reinforcement shall be got checked by the design engineer before slab is cast.

IV. MATERIALS

12. The sand shall be screened and shall be free from dust and silt.

13. Coarse sand shall be used for RCC works, machine crushed 20mm size down grade metal shall be used for RCC works.

14. Always fresh cement shall be used and should not contain lumps and 3 months old cement shall be tested for its strength properties.

V. CONCRETE

15. The RCC mix (1:2:4) M15 with 20mm down grade metal shall be used for slab and beams.

16. Machine mixing and measurement boxes shall be used for loading the sand and metal in to the hopper.

17. Pin vibrator for beams and pan vibrator for slab shall be used.

18. The thickness of slab shall be provided as per drawings.

19. The water cement ratio and workability of concrete are to be maintained under the supervision of an Engineer.

20. 5 or 6 concrete cubes of 15cmX15cmX15cm size shall be extracted and tested for its compressive strength at 14 days and 21 days age.

21. The junctions are to be treated with cement slurry between old and new concrete whenever there is time delay. Lapping shall be maintained at the junctions of old and new layers.

22. In case of rain during concreting the concrete shall be covered with gunny bags

available at site to avoid erosion of concrete.

23. Curing shall be done for minimum of 21 days with 1" depth of standing water. Start curing on the next day of work, before sun rays fall on the concrete.

VI. LEAK PROOF TREATMENT

24. The top of the roof shall be plastered with CM (1:4) mixed with 1 kg of ACCO proof compound per 1 bag of cement and fine rendering shall be done over the plastering.

NOTE: Any deviations of the above shall be brought to the notice of the engineer before execution of the work.

8.4.5.1 THEORETICAL CEMENT CALCULATION FOR VARIOUS ITEMS OF WORK.

Description of work		Cement for 1 Cum of work or 10 Sq.m of Finished Surface kgs
Cement mortar 1:2		720
Cement mortar 1:3		480
Cement mortar 1:4		360
Cement mortar 1:5		288
Cement mortar 1:6		240
Cement mortar 1:8		180
Cement concrete 1:1 1/2:3	0.45 X 960 =	432
Cement concrete 1:2:4	0.45 X 720 =	324
Cement concrete 1:3:6	0.45 X 480 =	216
Cement concrete 1:4:8	0.45 X 360 =	162
Cement concrete 1:4:10	0.38 X 360 =	137

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Cement concrete 1:5 :10	0.45 X 288 =130 kgs/cum
Cement concrete 1:5:121/2	0.38 X 288 =110 kgs/cum
Special ceiling plastering for RC slabs and beams with cement mortar 1:3,10 mm thick - 10 sqm	0.10 X 480 =48 kgs/ 10 sqm
Providing through vertical joint with 15 mm space filled with bituminous conpound ... with c.m. 1:3, 15 mm thick - 10 Sq.m	0.15 X 480 =72 /10 sqm
Brick work in cement mortar (1:6) using	
Bricks of size 9" X 4 3/8" X 2 3/4"	= 0.25 X 240 = 60 / 1 cum
Bricks of size 8 3/4" X 4 1/4" X 2 3/4"	= 0.28 X 240 = 67.2 / 1 cum
Bricks of size 8 3/4" X 4 1/4" X 2 1/4"	= 0.30X 240 = 72 /1 cum
Bricks of size 19 cm X 9 cm X 9 cm	= 0.22 X 240 = 52.80/ 1 cum
Bricks of size 19 cm X 9 cm X 5.7 cm	= 0.27 X 240 = 64.80/ 1 cum
Granolithic flooring c.c. 1:2:4 using 20mm thick using 12.5 mm size jelly	0.09 X 324 = 2916/ 10m^2
Pebble ash plastering 1:5 /12 mm thick	40.32 Kg /10 Sq.m
20mm thick	63.36 Kg/ 10 Sq.m
Honey comb brick work using bricks of 19 cm X 9cm X9cm in c.m. 1:3	0.7 X 288 = 201.6/10 Sq.m
Brickwork 10cm, thick-partition walls using 1class bricks in c.m. 1:3	0.14 X 480 = 67/ 1 cum
Plastering with c.m 1:5, 12 mm thick (20sq.m). 0.12 X (20/10) X288 =	69 / 20 Sq.m
Cutstone in cement mortar 1:2	0.16 X 720 = 115 / 1 cum

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Coursed Rubble masonry in c.m.1:2- 1 st Sort	0.28 X 720 =	202 kg/1 Cum
Coursed Rubble masonry in c.m. 1:5- 1 st Sort	0.28 X 288 =	81 „
Coursed Rubble masonry in c.m. 1:2- II nd Sort	0.32 X 720 =	230 „
Coursed Rubble masonry in c.m. 1:5- II nd Sort	0.32 X 288 =	92 „
Random Rubble masonry in c.m. 1:3	0.34 X 480 =	163 „
Random Rubble masonry in c.m. 1:6	0.34 X 240 =	82 „
Damp proof course in c.m. 1:4, 20mm thick -10 Sq.m	0.21 X 360 =	75.6 kg /10 Sq
Manufacturing sand cement blocks of size 60 X 30 X 20 c.m. in C.M 1:8 and packing them in C.M 1:6 in aprons and revetments -1 Cum	manufacturing 171.0 packing 14.4	
Manufacturing sand cement blocks of size 60 X 45 X 15 c.m. in C.M 1:6 and packing them in C.M 1:6 in aprons and revetments -1 Cum	manufacturing 228 packing 14.4	
Finishing the floor with 20mm cement concrete (Ellis pattern - 1 st Sort -No sand to be used) - 10 Sq.m		117
Paving Hydraulic pressed cement mosaic tiles of size 20cm X 20cm X 20mm thickness with c.m. 1:3, 20mm thick and pointed with same cement -10 Sq.m	0.21 X 480 =	101
Terraced roofing with brick on edge, 75mm concrete, 2 courses of flat tiles to top and 1 coat of cement plaster 1:3, 15mm thick	c.m. 0:24 X 480 = Plastering - 0.15 X 480 =72 Pointing - 0.05 X 480 =	115 24

Reinforced concrete 1:2:4 roofing 125mm thick, two courses of flat tiles with C.M. 1:3 -10 Sq.m	1.25 X 0.45 X 720 = 405 kgs
	Flat tiles 0.27 X 480 = 130
Terrace flooring with one course of pressed tiles 20cm X 20cm X 20mm using CM 1:3 10 Sq.m	0.04x 4.80 = 19
Pointing Flat tiles	0.05 X 480 = 24
Pressed tiles	0.04 X 480 = 19
Plastering with C.M. 1:3, 12mm thick - 10 Sq.m	0.12x480 = 58
Plastering with C.M. 1:5, 12mm thick - 10 Sq.m	0.12x288 = 35
(For country bricks) 1:5,12mm thick - 10 Sq.m	0.14x288= 40
Plastering with c.m 1:3,20mm thick - 10 Sq.m	0.22x480= 106
Stucco Plastering 12mm thick using blue granite chips of size 10mm and below over a base plastering in C.M 1:5, 12mm thick - 10 Sq.m	Base 1:5 -0,12 X 288 = 35
Pointing with CM 1:3 flush pointing Brick work - 10 Sq.m	0.06x480 = 29
Pointing with C.M 1:3 flush pointing Random Rubble Masonry - 10 m^2	0.09 x 480 = 43
Pointing with C.M 1:3 to full depth of tiles(marble slabs, pressed tiles, ornamental tiles) - 10 Sq.m	0.04 x 480 = 19

Pointing with C.M 1:3 cuddapah slabs to full depth - 10 Sq.m	0.05 x 480 = 24
Pointing with C.M. 1:3 square brick flooring - 10 Sq.m	0.05 x 480 = 24

8.4.5.2 Table : Fair Estimate of Cement and Steel in Building Works

	Residential Bldgs	Office Buildings (Multistoreyed)		Commercial Bldgs such as Bus stands, Restaurents etc. Framed structure
		Load bearing	Framed structure	
1. Cement (i) For 100 sqm of plinth area	16 to 20 tons (320 to 400) bags	12 to 15 tons (240 to 300) bags	25 tons (500) bags	22.5 tonns (450) bags
2. STEEL (i) For 100sqm of plinth area.	1.8 to 2.0 tonnes	1.5 tonnes	2.5 tonnes	2.25 tonnes
(ii) For 1 cum of RCC	100 Kgs	80Kgs	150 Kgs	120 Kgs
(iii) For Rs1.0 lakh cost estimate (1989-90) rates	1.5 tonnes	1.20 tonnes	2.00 tonnes	1.75 tonnes
Various sizes				
Dia 6mm	15%	15%	10%	10%
8 mm	50%	40%	30%	25%

10 mm	15%	10%	10%	15%
12 mm	10%	15%	15%	10%
16 mm	10%	15%	10%	20%
20 mm	-	5%	15%	10%
25 mm	-		10%	10%

For Foundation & Basement of Framed Structures (Upto 4 storeyed) alone

CEMENT

For 100sqm of Plinth area	2.0 tonnes (40 bags)	1.75 tonnes (35 bags)	2.5 tonnes (50 bags)	2.25 tonnes (45 bags)

STEEL

For 100 sqm of Plinth area	500kg	400kg	650 kg	750kg

Materials required for single Storey Building

*Cement	: 3.50 to 4.00 Bags / sqm of Plinth area
*Steel	: 12 Kg to 15 Kg / Sqm of Plinth area
* Bricks	: 250 to 300 nos / sqm of Plinth area

8.4.5.3 Task or out turn per day per skilled labour

White washing or colour washing three coats	70.00 sqm per white washer
White washing or colour washing door one coat	200.00sqm per white washer
Painting or varnishing door or windows one coat	25.00 sqm per painter
Painting large surface one coat	35.00 sqm per painter
Distempering one coat	35.00 sqm per painter

2.5 cm C.C. floor	7.50 sqm per mason
Timber framing sal or teak wood	0.07 cum per carpenter
- Do - Country wood	0.15 cum per carpenter
Door or Window shutter panel or glazed	0.70 sqm per carpenter
- Do - battened	0.80sqm per carpenter
Single Allahabad tiling or Mangalore tiling	6.00sqm per tile layer

8.5.1 Horse power required to lift different quantities of water to elevations 3m to 30m

H.P. = (Quantity of rpm x Head in metres x 2 / 60x76) Assuming 50% efficiency of pump.

Qty in lpm	HP required for the elevation of						
	3m	5m	10m	15m	20m	25m	30m
500	0.66	1.10	2.19	3.29	4.39	5.48	6.58
1000	1.32	2.19	4.39	6.58	8.77	10.96	13.16
1500	1.97	3.29	6.58	9.87	13.16	16.45	19.74
2000	2.63	4.39	8.77	13.16	17.54	21.93	26.32
2500	3.29	5.48	10.96	16.45	21.93	27.41	32.89
3000	3.95	6.58	13.16	19.74	26.32	32.89	39.47
3500	4.61	7.68	15.35	23.03	30.70	38.38	46.05
4000	5.26	8.77	17.54	26.32	35.09	43.86	52.63
4500	5.92	9.87	19.74	29.61	39.47	49.34	59.21
5000	5.68	10.96	21.93	32.89	43.86	54.82	65.79
5500	7.24	12.06	24.12	36.18	48.25	60.31	72.37
6000	7.89	13.16	26.32	39.47	52.63	65.79	78.95
6500	8.55	14.25	28.51	42.76	57.02	71.27	85.53
7000	9.21	15.35	30.70	46.05	61.40	76.75	92.10
7500	9.87	16.45	32.89	49.34	65.79	82.24	98.68

NORMS FOR ELECTRICAL INSTALLATION IN RESIDENTIAL BUILDING

1/2 H.P monoblock, centrifugal motor pumpset is adequate in case of open well with suction height upto 26 feet - for a family of eight.

1/2 H.P. jet motor - pumpset is adequate in case of open well or bore well with suction height ranging from 26 feet to 50 feet - for a family of eight.

8.5.2. COST OF ELECTRIC PUMPING

To pump one gallon of water in one minute against a head of one foot with 100% over all efficiency requires 0.000189 kilowatts. Pumping 1,000 gallons per minute per foot head at 100% efficiency requires 0.189 kilowatts.

The following formula can be used for determining power costs of pumping.

Cost per 1000 gallons for each foot of head

$$= \frac{0.189 \times R}{P.E. \times M.E. \times 60} = \frac{0.00315 \times R}{O.P.E.}$$

Where :

R = Power cost per kilowatt hour.
P.E. = Pump efficiency
M.E. = Motor efficiency
O.P.E. = Overall plant efficiency

Cost per hour

$$= \frac{0.000189 \times \text{gallons per min} \times \text{total head} \times R}{O.P.E.}$$

To determine H.P. at electric meter

$$H.P.\ (\text{at meter}) = \frac{R.M.D. \times K \times 3600}{t \times 746}$$

Where :
RMD = revolution of meter disc in time t.
K = disc constant in watt hours per revolution of disc (generally found stamped on the meter)
t = time in seconds

$$\text{Specific speed of a pump (Metric) } nq = \frac{3.65\, n\, Q^{1/2}}{H^{3/4}}$$

Specific speed of a pump (British/U.S.) $nq = nq / H^{3/4}$

Conversion factor : If nq = 1 Metric, it is 12.9 British and 14.2 U.S. specific speeds.

8.6 TRIGONOMETRICAL FUNCTIONS OF ANGLES

Angle Deg.	Radn	Chord	Sine	Tan	Cot	Cos	Chord	Radian	Deg.
0	0	0	0	0	o	1	1.414	1.5708	90
1	.0175	.017	.0175	0.175	57.2900	.9998	1.402	1.5533	89
2	.0349	.035	.0349	.0349	28.6363	.9994	1.389	1.5359	88
3	.0524	.052	.0523	.0524	19.0811	.9986	1.327	1.5184	87
4	.0698	.070	.0698	.0699	14.3006	.9976	1.364	1.5010	85
5	.0873	.087	.0872	.0875	11.4301	.9962	1.351	1.4834	85
6	.1047	.105	.1045	.1051	9.5144	.9945	1.338	1.4661	84
7	.1222	.122	.1219	.1228	8.1443	.9925	1.325	1.4486	83
8	.1496	.139	.1392	.1405	7.1144	.9903	1.312	1.4312	82
9	.1571	.157	.1564	.1584	6.3138	.9877	1.299	1.4137	81
10	.1745	.174	.1736	.1763	5.6713	.9848	1.286	1.3963	80
11	.1920	.192	.1908	.1944	5.1446	.9816	1.272	1.3788	79
12	.2094	.209	.2079	.2126	4.7046	.9781	1.259	1.3514	78
13	.2269	.226	.2250	.2309	4.3315	.9744	1.245	1.3439	77
14	.2443	.244	.2419	.2493	4.0108	.9703	1.231	1.3265	76
15	.2681	.261	.2588	.2679	3.7321	.9659	1.217	1.3090	75
16	.2793	.278	.2756	.2867	3.4874	.9613	1.204	1.2915	74

[illegible] Radn	Chord	Sine	Tan	Cot	Cos	Chord	Radian	Deg.
17 .2967	.296	.2924	.3057	3.2709	.9563	1.110	1.2741	73
18 .3142	.313	.3090	.3249	3.0777	.9511	1.176	1.2566	72
19 .3316	.330	.3256	.3443	2.9042	.9455	1.171	1.2392	71
[illegible] .3491	.347	.3420	.3640	2.7475	.9397	1.147	1.2217	70
[illegible] .3665	.364	.3584	.3839	2.6055	.9336	1.133	1.2043	69
[illegible] .3840	.382	.3746	.4040	2.4751	.9272	1.118	1.1868	68
[illegible] .4014	.399	.3907	.4245	2.3559	.9205	1.104	1.1694	67
[illegible] .4182	.416	.4067	.4542	2.2460	.9135	1.089	1.1519	66
[illegible] .4363	.433	.4226	.4663	2.1445	.9063	1.075	1.1345	65
[illegible] .4538	.450	.4384	.4877	2.0503	.8988	1.060	1.1170	64
[illegible] .4712	.467	.4540	.5095	1.9626	.8910	1.045	1.0996	63
[illegible] .4887	.484	.4695	.5317	1.8807	.8829	1.030	1.0821	62
[illegible] .5061	.501	.4848	.5543	1.8040	.8746	1.015	1.0645	61
[illegible] .5226	.518	.5000	.5774	1.7321	.8660	1.000	1.0472	60
[illegible] .5411	.531	.5150	.6009	1.6643	.8572	.985	1.0123	59
[illegible] .5585	.551	.5299	.6249	1.6003	.8480	.970	1.0297	58
[illegible] .5760	.568	.5446	.6494	1.5399	.8387	.954	.9948	57
[illegible] .5934	.585	.5592	.6745	1.4826	.8290	.939	.9774	56
[illegible] .6109	.601	.5736	.7002	1.4281	.8192	.923	.9599	55
[illegible] .6283	.618	.5878	.7265	1.3761	.8090	.908	.9425	54
[illegible] .6458	.635	.6018	.7536	1.3270	.7986	.892	.9250	53
[illegible] .6632	.651	.6157	.7813	1.2799	.7880	.877	.9076	52
[illegible] .6807	.668	.6293	.8098	1.2349	.7771	.861	.8901	51
[illegible] .6981	.684	.6428	.8391	1.1918	.7650	.845	.8727	50
[illegible] .7156	.700	.6561	.8693	1.1504	.7549	.829	.8552	49
[illegible] .7380	.717	.6691	.9004	1.1106	.7431	.813	.8378	48
[illegible] .7505	.733	.6820	.9325	1.0724	.7314	.797	.8203	47
[illegible] .7670	.749	.6947	.9657	1.0355	.7193	.781	.8029	46
45 .7854	.765	.7071	1.0000	1.0000	.7071	.765	.7854	45

8.7.1. ELECTRICITY - GENERAL USEFUL FACTS

1. H.P. = 745 watts	= 0.745 kw - 23,000 ft.lbs,per min.
	= 1.104 metric HP " 76.11 kg.m/sec
1. Metric H.P.	= 75 m.kg/sec
1. B.O.T. unit	= 1,000 watt Hours (or) 1 kilowatt hour
Torque (ft.lbs)	= H.P. X 33,000 (R.P.M. X 2)
Current, Amperes	= watt / volts
Motor Output in H.P.	= (Kilowatt input X Efficiency) /0.746
K.V.A.	= (Volts X Amps) / 1,000
Power factor	= ratio of True Power/Apparent power
	= Kilowatt / K.V.A.
True power in 3 phase Circuit in kilowatt	= (1.73 X volts X Amps X p.p) / 1000
Single phase motor Amperes	= H.P. X 746 / (Efficiency X volts X P.F)
Two phase motor Amperes	= H.P X 746 / (Efficiency X volts X P.F X 2)
Three phase amperes	= H.P X 746 / (Efficiency X volts X P.F X 1.73)
1 Electrical unit (B.O.T.)	= 1 kilowatt hour
1 kilowatt (K.W.)	= 738 ft per sec.
	= 102 m kg. per sec.
	= 1.341 horse power
	=1.360 Metric horse power
1 kilowatt hour (KWH)	= 3.413 B.Th.U.
	= 880 Calories
	= 0.1383 mkg.
1 B.Th.U.	= 778.3 ft lb.
	= 107.6 mkg.
	= 0.2520 calories
1 Calorie (cal.)	= 3.088 foot pounds
	= 3698 B. Th.U.

8.7.2 AN IDEA WHAT CAN BE DONE APPROXIMATELY

By one unit of Electricity in the House Hold

Raise	2500 gallons of water over 50 ft
Light	One 40 watt lamp 25 hrs
Make	60 cups of water to bath temperature
Operate	A floor polisher of vacuum cleaner six hours

Run	A sewing machine for 20 hrs
Run	A washing machine to do two weeks washing for six persons
Run	A 10" fan for 150 hrs
Run	A ceiling fan for 10 hrs
Milk	52 cow
Separate	3000 gallons of milk
Churn & Work	180 lbs. of water
Heat	2 gallons of water
Make	10 lbs. of ice

The probable consumption of electricity by appliances in general use in hrs/ Unit

Lamps	(Vacuum or filament type)
Marked 20W	50 hrs
Marked 40W	25 hrs
Marked 60W	16 hrs
Marked 100W	10 hrs
Night Lamp	200 hrs

Fires

Marked 1KW(1000W)	1 hr
Marked 2KW(1000W)	½ hr

Irons

Marked 330W	3 hrs
Marked 500W	2 hrs

Cookers

(4 to 5 KW) Varying with nature of use, generally one unit per day for each person in a small family where all cooking is done by electricity.

Water heaters (Storage type) for bath : 3 to 4 units per bath

Boiling vessels (½ pint capacity)
Milk heater, shaving water mugs etc 25 to 30 operations/unit

Fan (small) :	10 15 hrs.
Vaccum cleaner :	4 hrs
Cloths washing machine :	4 hrs

8.8. CONVERSION TABLE

1. LENGTH

1 inch	=	2.54 cms	1 cm	=	0.393701 inch.
1 ft	=	30.479 cms	1 cm	=	0.0328 feet
1 yd	=	0.91438 cms	1 m	=	1.09361 yds
1 mile	=	1.609344 km	1 km	=	0.62187 mile
1 furlong	=	201.188 metres	1 metre	=	0.004971 furlong
100' chain	=	0.03048 km	1 m	=	3,28084 feet

2. AREA

1 in^2	=	645.16 m.m^2	1mm^2	=	0.00155 m.m^2
1 ft^2	=	0.0929 m^2	1m^2	=	10.7639 ft^2
1 yd^2	=	0.836127 m^2	1m^2	=	1.196068 yd^2
1 Acre	=	(4840 sq. yards)	1 hectare	=	2.4698676 acre
	=	0.40468 hectare		=	10,000 sq.m.
1 acre	=	4046.856 m^2	1 m^2	=	0.000247 acres
1 sq. mile	=	2.589988 km^2=640 Ac	1 m^2	=	0.386106 mile^2
1 mile^2	=	258.999 hectare (ha)	1 ha	=	0.00386 mile^2

3. VOLUME AND CAPACITY

1 inch^3	=	16.3871 cm^3	1 cm^3	=	0.0610237 inch^3
1 ft^3	=	0.0289168 m^3	1 m^3	=	35.3147 ft^3
1 ft^3	=	28.316 litres	1 litre	=	0.035315 ft^3
	=	6.24 gallons			
1 acre ft	=	1233.50 m^3	1 m^3	=	0.00081071 acre ft
1 yd^3	=	0.764555 m^3	1 m^3	=	1.30795 yd^3
1 acre ft	=	0.1234 hectare metres	1 hectare mt	=	8.1037 acre ft
1 acre ft	=	1613.33 yd^3	1 yd^3	=	0.00061984 acre ft
1 yd^3	=	0.764555 kilo litres	1 kilo litre	=	1.30795 yd^3
1 Imp gallon	=	4.54596 litres	1 litre	=	0.219975 Imp gallon
1 US. big gallon	=	3.78531 litre	1 litre	=	0.264179US.big gallon

FLOWRATE- VOLUME

One cusec day	1.9835 Acre feet
	0.0864 Million cubic feet
	8.64 Hectare Metre
One cusec day	0.0864 Million cubic Metres

4. WEIGHT

1 lb average	= 0.4535929 kg	1 kg	= 2.20462 lbs average
1 kip	= 1000lbs (ave)	1 lb (ave)	= 0.001 kips
1 poundals	= 0.13826 newtons	1 newton	= 7.233 poundals
1 cwt	= 50.802 4.kg	1 c.kg	= 0.01968 cwt
1 cwt	= 0.508024 Quintals	1 Quintal	= 1.9684 cwt(100 kg)
1 ton	= 19.684 cwt	1 cwt	= 0.05081 tonnes
1 ton	= 1016.0536 kgs	1 kg	= 0.00098421
1 ton (2240 lbs)	= 1.01606 MT	1 MT	= 0.98419 tonnes
1 Tola	= 0.0116635 kgs	1 kg	= 85.735 Tolas
1 maund	= 37.324157 kgs	1 kg	= 0.0267923 Mound
1 seer	= 0.933106 kg	1 kg	= 1.07169 seers
1 ounce (ave)	= 28.35 gm	1 gm	= 0.03527 ounce (ave)
1 Tola	= 0.411429 ounce	1 ounce	= 2.43056 Tolas
1 kg	= 9.807 Newton	1 Newton	= 0.1020 kgs
1 Tonne	= 9.807 Kilo Newton	1 Kilo Newton	= 0.102 Tonnes

5. WEIGHT PER UNIT LENGTH

1 lb/inch	= 17.8582 kg/m	1 kg/m	= 0.055996 lb/inch
1 lb/yd	= 0.496055 kg/m	1 kg/m	= 2.015906 lb/yd
1 lb/yd	= 0.333333 lb/ft	1 lb/ft	= 3.000 lb/ft
1 lb/ft	= 1.48882 kg/m	1 kg/m	= 0.6720 lbs/ft
1 lb/inch	= 12.00 lb/ft	1 lb/ft	= 0.083333 lbs/inch
1 kg/m	= 1 t/ k.m	1 t/k.m	= 1 kg/m
1 Ton/ft	= 3.3335 t/m	1 f/m	= 0.30000 t/ft

6. DENSITY

1 lb/inch^3	= 27.68032 gm/cm^3	1 gm/cm^3	= 0.0361267 lb/ inch^3
1 lb/ft^3	= 16.0187 kg/m^3	1 kg/m^3	= 0.06243 lb/ft^3
1 lb/yd^3	= 0.593278 kg/m^3	1 kg/m^3	= 1.68555 lb/yd^3
1 lb/inch^3	= 27679.900 kg/m^3	1 kg/m^3	= 0.0000361272 lb inch^3
1 lb/inch^3	= 1728 lbs/fts^3	1 lb/ft^3	= 0.0005787 lb/inch^3
1 gm/cm^3	= 1000 kg/m^3	1 kg/m^3	= 0.001 gms/cm^3
1 T/yd^3	= 1328.959 kg/m^3	1 kg/m^3	= 0.75246 X 10^{-3} t/yd^3
1 T/yd^3	= 1.328959 t/m^3	1 t/m^3	= 0.752488 t/yd^3
1 lb/cu.in	= 27.68032 gm/c.c.		
1 lb/cu.ft	= 16.0187 kg/cu.m		

7. STRESS

1 lb/inch^2	= 0.7030 kg/cm^2	1 kg/cm^2	= 14.223 lb/inch^2
1 lb/ft^2	= 4.86243 kg/m^2	1 kg/m^2	= 0.205 lb/ft^2
1 t/inch^2	= 1.575 kg/mm^2	1 kg/mm^2	= 0.635 t/inch^2
1 T/ft^2	= 1.0943 kg/cm^2	1 kg/cm^2	= 0.91383 t/ft^2
1 T/ft^2	= 10.937 t/m^2	1 t/m^2	= 0.914 t/ft^2
1 kg/cm^2	= 10 t/m^2	1 t/m^2	= 0.1 kg/cm^2
1 kg/mm^2	= 9.807 Newton/mm^2	1 Newton/mm^2	= 0.102 kg/mm^2
1 kg/cm^2	= 0.09807 Newton/mm^2	1 Newton/mm^2	= 10.20 kg/cm^2
1 lb/sq.in	= 0.7030 kg/sq.cm	t	= Metric Ton
T	= British Ton		

8. POWER

1 H.P.	= 0.74570 K.W. (British H.P.)	1 K.W.	= 1.34102 H.P.
1 H.P.	= 1.01387 M.H.P.	1 M.H.P.	= 0.98632 H.P. (British)
1 H.P.	= 745.70 Watts	1 Watt	= 0.001341 H.P.
1 H.P. hour	= 2.68452 X 10^6 Joules	1 Joule	= 0.3725 X 10^{-6} H.P. hour
1 H.P. hour	= 641.19 Kilo calories	1 Kilo calory	= 0.001559 H.P. hour
1 H.P. hour	= 0.74570 kilowatt hrs	1 kilo watt hour	= 1.341 H.P. hour
		1 cheval vapour	= 75 kg.m/sec = 736 watts (Metric H.P.)
		1 Metric/min	= 0.65616 in/sec

9. VELOCITY

1 ft/sec	= 0.3048 m/sec	1 metre/sec	= 3.2808 ft/sec
		1 km/hr	= 27.7778 cm/sec
1 cm/sec	= 0.022369 miles/hr	1 mile/hr	= 44.704 cm/sec
1 ft/sec	= 0.68182 miles/hr	1 mile/hr	= 1.46667 ft/sec
1 m/sec	= 2.236936 miles/hr	1 mile/hr	= 0.44704 metre/ sec
1 ft/sec	= 1.09728 km/hr	1 km/hr	= 0.91134 ft/sec

10. DISCHARGE

(Cusecs)		(Cumecs)	
1 ft^3/sec	= 0.028317 m^3/sec	1 m^3/sec	= 35.3147 ft^3/sec
1 gl/sec	= 4.5460 lit/sec	1 lit/sec	= 0.21997 gl/sec

11. SPEED

1 mile/hr	= 1.609344 km/hr	1 km/hr	= 0.52137 mile/hr

12. MOMENT

1 lb.feet	= 0.138 kg cm	1 lb.in	=	1.1521 kg cm
1 kg cm	= 10^{-5} tm	1 kg metre	=	7.233 lb feet
		1 tm	=	10^{5} kg cms
1 kg.metre	= 9.807 Newton metre	1 Newton metre	=	0.102 kg.mt
1 ton metre	= 9.807 kilo Newton metre	1 kilo Newton mt	=	0.102 ton metre

13.PRESSURE

1 atmosphere	=	14.7 lbs/inch^2	1 lbs/inch^2	=	0.06805 atmosphere
	=	1002 m of water column			
1 inch. of Hg	=	0.491145 lbs/inch^2	1 lbs/inch^2	=	2.03606 inch of Hg
1 inch. of Hg	=	0.0345316 kg/cm^2	1 kg/cm^2	=	28.95898 inch of Hg
				=	736.6mm of Hg at NTP
					= 1 Atmosphere
1 lb/ft^2	=	4.8868 kg/m^2	1 mm of Hg	=	2.78507 lbs/ft^2
1 lb/inch^2	=	2.30672 ft of water	1 ft of water	=	0.433515 lg/inch^2
	=	0.0703 kg/cm^2			
1 ft of water	=	0.03048 kg/cm^2	1 kg/cm^2	=	32.8084 ft of water
				=	14.223 lbs/inch^2
1 ton/sq.cm	=	10.9367 tonnes/m^2		=	10.20 m of water Col

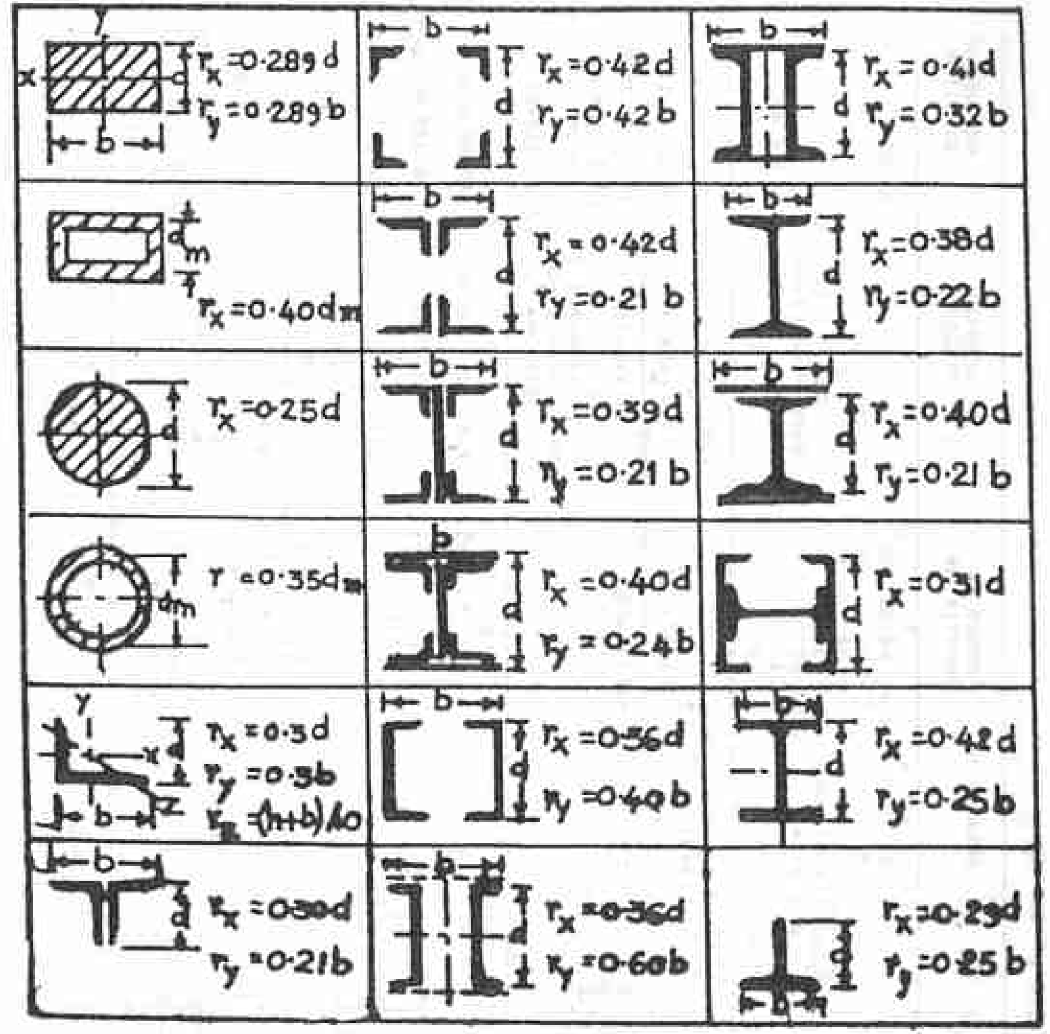
14. MISCELLANEOUS

1 lb/sec	=	0.4535924 kg/sec	1 kg/sec	=	2.204523 lbs/sec
1 lb/sec	=	1.53293 ten/hr	1 t/hr	=	0.612396 lb/sec
1 degree	=	60 Minute	1 Minute	=	0.0167 degree
1 degree	=	0.017453 Radians	1 Radian	=	57.296 degrees
1 fatham	=	6 feet	1 ft	=	0.1655 fatham
1 I.N.M.	=	1.852 kilo Metres	1 K.M.	=	0.53996 INM
1 I.N.M.	=	1.15078 Miles	1 Mile	=	0.858976 INM
1 lb/acre/day	=	0.122 gm/day/sq.m	1 lb/acre/day	=	1.22 kg/Ha/day
1 lb/day/cu.ft	=	16.0 kg/day/cu.m	1 lb/day/sq.ft	=	0.0049 gm/day/sq.m

15. TEMPERATURE

C/5 = (F-32)/9;

C = Temperature in Centigrade, F = Temp. in Fareignheit.



8.9 APPROXIMATE RADII OF GYRATION OF STRUCTURAL ELEMENTS

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Table 8-10 PROPERTIES OF SECTIONS

Sl. No.	SECTION	AREA	MOMENT OF INERTIA ON	SECTION MODULUS	RADIUS OF GYRATION
1	TRIANGLE	1/2 bc sin A $\frac{BD}{2}$ $\sqrt{s(s-a)(s-b)(s-c)}$ $s = \frac{a+b+c}{2}$	x.x. $\frac{BD^3}{36}$ zz. $\frac{BD^2}{4}$ y.y. $\frac{BD^3}{12}$	x.x. BASE $\frac{BD^3}{12}$ APEX $\frac{BD^3}{24}$	ABOUT X.X 0.236 D
2	TRAPEZIUM	$(A+B)\frac{D}{2}$	X.X. $\frac{(B^2+4BA+A^2)}{36(A+B)}D^2$	X.X BASE $\frac{(B^2+4BA+A^2)}{36(A+B)(D-Y)}D^2$ TOP $\frac{(B^2+4BA+A^2)}{12(B+2A)}D^2$	ABOUT X.X $\sqrt{\frac{2(B^2+4BA+A^2)}{36(A+B)^2}}$
3	RECTANGLE	B.D	X.X. $\frac{BD^3}{12}$ Y.Y $\frac{BD^3}{6}$ Z.Z. $\frac{B^3D^3}{6(B^3+D^3)}$	X.X. $\frac{BD^2}{6}$ TOP OR BASE Z.Z. $\frac{B^3D^3}{6\sqrt{B^2+D^2}}$	ABOUT X.X 0.289 D
4	HOLLOW RECTANGLE	B(D-d)	X.X. $\frac{B(D^3-d^3)}{12}$ Y.Y (X.X) $\frac{BD^2}{4}(D-d)$	X.X. TOP OR BASE $\frac{B(D^2-d^2)}{6D}$	ABOUT X.X $\sqrt{\frac{D^2+dD+d^2}{12}}$
5.	SPHERE	πD^2	$\frac{\pi D^4}{12}$	$\frac{\pi D^3}{6}$	0.5 D or R
6	FLANGED BEAM	bd+b(D -d)	X.X. $+1/3[BN^3+b(D-N)^3-(N-D)^3(B-b)]$	X.X. $\frac{I_{X.X}}{D-N}$	ABOUT X.X $\sqrt{\frac{I_{X.X}}{A}}$
7.	SQUARE	D^2	X.X. $\frac{D^4}{12}$ Y.Y $\frac{D^4}{12}$	X.X. $\frac{D^3}{6}$ Y.Y. $0.118D^3$	ABOUT X.X OR Y.Y. 0.289 D.
8.	REGULAR HEXAGON ON FACE	$0.866\ D^2$	X.X. $0.06\ D^4$ Y.Y $0.06\ D^4$	X.X. $0.12\ D^3$ Y.Y. $0.104\ D^3$	ABOUT X.X OR Y.Y. 0.263 D.
9.	REGULAR HEXAGON ON EDGE	$0.65\ D^2$	X.X. $0.0336D^4$	X.X. $.0676\ D^3$	ABOUT X.X 0.228 D.
10.	REGULAR OCTAGON	$0.828\ D^2$	X.X. $0.055\ D^4$ Y.Y. $0.055\ D^4$	X.X. $0.109\ D^3$ Y.Y. $0.1016\ D^3$	ABOUT X.X. OR Y.Y. 0.257 D
11.	CIRCLE	$0.7854\ D^2$	X.X. $\frac{\pi}{64}D^4$	X.X. $\frac{\pi}{32}D^3$	ABOUT X.X. $\frac{D}{4}$
12.	HOLLOW CIRCLE	$0.7854 \times (D^2-d^2)$	X.X. $\frac{\pi}{64}(D^4-d^4)$	X.X. $\frac{\pi}{32}\frac{(D^4-d^4)}{D}$	$\frac{1}{4}\sqrt{D^2+d^2}$
13.	QUARTER CIRCLE (inner)	$0.7854\ R^2$	X.X. $.0548\ R^4$	X.X. $0.129\ R^2$	0.264 R
14.	QUARTER CIRCLE (outer)	$0.2146\ R^2$ $0.1075\ \text{chord}^2$	X.X. $.0075\ R^4$	X.X. $.0095\ R^2$	.187 R

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Table 8-11 Geometrical Properties of sections

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Sl. No.	Geometrical form	Perimeter	Area	Centre of Gravity
1	Ellipse (axes 2a, 2b)	$2\pi\sqrt{\frac{a^2+b^2}{2}}$ (approx)	πab	AT CENTRE.
2.	Parabola (base = b height = h)	$\sqrt{b^2 + 5.33h^2}$	2/3 bh	In the axis 3/5 of its length from vertex.
3.	Parallelogram (sides = a,b angle between them = θ ht to base a = h)	2 (a + b)	ab sin θ = ah	At the inter section of diagonals.
4.	Quadrilateral (d_1 d_2 = diagonals θ = angle of their intersection)	---	$1/2\ d_1 d_2 \sin\theta$	
5.	a) Trapezium (two parallel sides = a & b, base = a ht = h)	--	1/2 (a + b) h	$\frac{h\ (2a + b)}{3\ (a + b)}$
	b) Trapezium (No side is parallel (b+a+c) = base, h, H heights)	--	$\frac{a(H+h)+bh+cH}{2}$	
6.	Quadrant of circle	---	---	On the middle radius at 0.90028 X radius from centre.
7.	Semi parabola	---	---	3/5 of the axis from vertex & 3/8 of semi base from axis.
8.	Rectangular solid or cubical edges a,b,c.	---	2 (ab + bc + ca)	AT CENTRE.
9.	Cylindrer (radius r, height h)		$2\pi r\ (h+r)$	AT CENTRE.

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Sl. No.	Geometrical form	Area	Volume	Centre of Gravity
10.	Cone (radius r, height h, slant height l)		$1/3\pi r^2 h$ $\pi r\ (l+r)$	h/4 above base.
11.	Sphere (radius r)	$4\pi r^2$	$4/3\pi r^3$	AT CENTRE.
12.	Hemisphere	$3\pi r^2$	$2/3\pi r^2$	3/8 radius from base to the vertex or 5/8r from Appex
13	Sector of sphere (Spherical sector)	(2h+0.25c)	$2/3\pi r^2 h$	3/4 (r - h/2)
14.	Pyramid (A-Area of base s = length of each side, n = number of sides, r = half the base, h = height) r = radius of inscribed circle R = radius of circumscribed circle	or	1/6 Ah $\frac{rhsn}{6}$ or $\frac{nsh}{6}$	$\sqrt{R^2 - s^2/4}$ h/4 above base
15.	Paraboloid	---	---	In the axis, 2/3 of its length from vertex.
16.	Anchor ring (mean radius R, radius of circular section r)	$4\pi r^2 R$	$2\pi r^3$	
17.	Spherical segment	$2\pi rh$	$\pi\frac{h^2}{3}(3r-h)$	$\frac{h\ (4r-h)}{4\ (3r-h)}$
18.	Circular Sector a = π r θ/180	1/2 a r	---	2/3 br/a (of Semi Circle. $4r/3\pi$ from Dia).

		T/sft	T/M²
Block masonry in 1:3 cement mortaraverage crushing strength of block not less than			
500 lbs/sq.in		2½	27
1000 lbs/sq.in		6	66
2000 lbs/sq.in		9½	104
Solid cement conrete block masonry in cement	1:3	15	164
(As per the IRC standard the following are the safe working stresses in tension for masonry)			
Ist class dressed stone or 1:3 cement concrete Block masonry in cement mortar		1.30	14
Ist class dressed stone or 1:2 cement concrete Block masonry in cement mortar		0.65	7
Ist class brick masonry in 1:3 cement mortar		1.30	14
In lime mortar	1:2	0.65	7
Lime concrete masonry with stone metal and hydraulic lime mortar		0.65	7

Note:

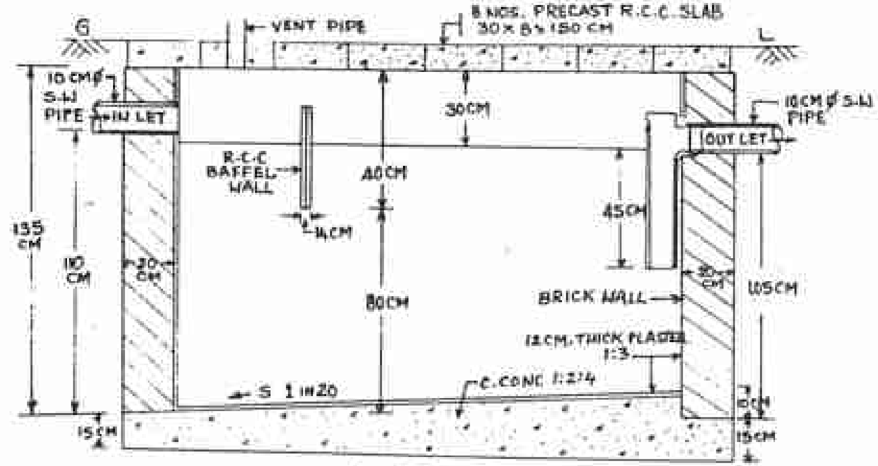
1) It is uneconomical to use cement mortar richer than (1:6) for bricks of ordinary grade (Hyderabad brick whose strength is around 35 kg/cm²) as it does not improve its bearing strength proportionately.

2) Combination of wind/earthquake loads :- Increase the permissible stresses by 33% (wind and earthquake does not simultaneously act)

3) As lintel beam bearings/coloumn bases etc to increase upto 20% is permissible (local stress) and the contact area may be checked for this uniformly distribution pressure.

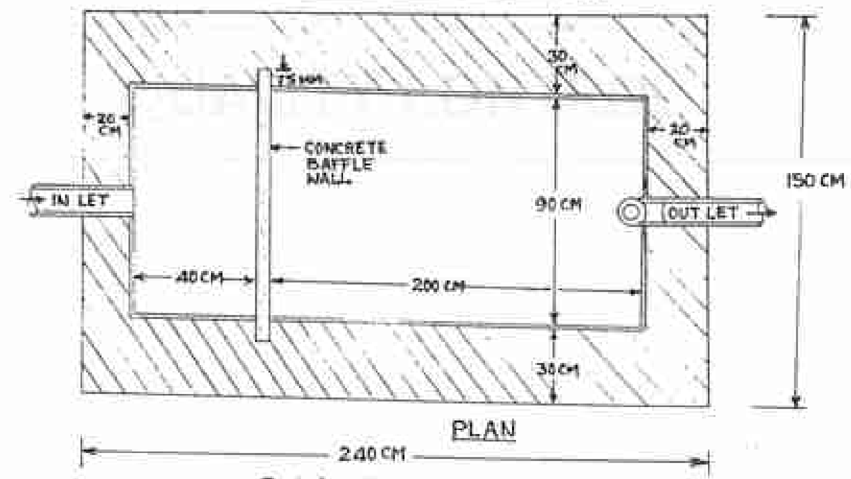
4) For eccentric loadings - 25% excess permissible provided there is not tension in masonry.

5) Permissible stress in shear and or tension should be not more than 10% of the allowable comparative stresses.



L. SECTION

SECTION OF SEPTIC TANK



PLAN

PLAN OF SEPTIC TANK

SIZES OF SEPTIC TANK				LIQUID CAP PER
No. of users	Length	Breadth	Liquid depth	Cum
5	1.2	0.4	1 to 1.05	0.1
10	1.44	0.48	1 to 1.40	0.1
15	1.50	0.50	1 to 2.00	0.1
20	1.80	0.60	1 to 1.80	0.1
50	2.70	0.90	1 to 2.00	0.1

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QUALITY CONTROL

DO'S AND DO NOT'S OF WORKS OF IMPORTANT SPECIFICATIONS OF EARTH WORK
(IS CODES, 2720, 4701, 8237, 9451, 4081, 1200)
EXCAVATION OF CANALS :

DO'S	DO NOTS
1. Fix up the centre line and set the curves correctly.	1. Do not avoid approval of the deviation statement.
2. Take working levels, real variation in ground levels and classification of soils as per Govt. Memo No. 1970-12/72-11, dated 4-7-1994.	2. Avoid over break and loosening of canal.
3. Get top soil vegetation (organic/inorganic) etc. removed.	3. Do not mix up useful soils with other soil of cutting
4. Provide treatment with C.N.S. Soils in B.C. Reaches (expansive soils)	
5. Form spoil bank as per drawing and away from side drain with suitable gaps for drainage into the valley.	
6. Form Dowel Bank, as per drawing.	
7. Form Inspection path to a uniform longitudinal gradient and with gentle transverse slope towards drains.	
8. Compact over excavation/breakage portion with suitable soils, gravel, spalls.	

FORMATION OF EXBANKMENTS

DO'S	DO NOTS
1. Get the top spoil, vegetation and sand patches removed to complete depth.	1. Do not raise the bank in piecemeal.
2. Scarify the ground and wet properly.	2. Do not allow new layer without scarification and wetting of old layer.
3. Obtain P.D. OMC for the useful soils and borrow soils.	3. Do not allow new layer unless required degree of compaction is achieved.
4. Provide C.O.T.s according to height of bank.	4. Don't leave any loose layer un-rolled at the end of the day in rainy season.
5. Raise embankment to full width with uniform horizontal layer of 25cm thickness.	5. Don't allow compacted layer to be more than 150mm.
6. Break clods, remove roots, big boulders other materials etc., larger than 80mm from the soils used in embankment.	6. No new layer to be laid unless the over moistured layer is either completely removed or allowed to dry.
7. Supplement deficit moisture whenever required.	7. Don't dump soils in heaps.
8. Provide 0.45 Mtr. Extra offset on both sides of bank.	8. Don't dump the soils in water and slush.
9. Compaction with 8 to 10 to Tonnes Power roller, vibro max roller	9. No. of roller passes should not be less than 10.
10. Conduct field compaction tests and determine compaction efficiency.	10. Do not forget to provide settlement allowance of 2cm/mtr. Height of bank.
11. Check embankment profiles periodically.	11. Don't use expansive soils in banking.
12. Ensure 8 minimum No. of Passes.	
13. Provide 10% allowance in setting profile of the embankment.	

CANAL LINING	
DO'S	**DO NOTS**
1. Check the canal prism and verify the bed levels.	1. Do not allow the concrete over loose subgrade.
2. Check the gradation analysis of fine and coarse aggregats to the requirement of mix at batching plant.	2. Do not allow lining without wetting subgrade.
3. Allow the ingredients of fine and coarse aggregate as per required mix by weigh batching.	3. Do not allow C.C. Lining manually without vibration.
4. Check the calibration of weighing machine at batching plant.	4. Do not allow segregation of concrete while laying through discharge conveyor.
5. Check the water meter and its discharge.	5. Do not allow concrete directly on subgrade from transit mixer.
6. Check the batch of cement, its make and test results.	6. Do not form contraction joints over longitudinal drains.
7. Check the water cement ratio and record the slump.	7. Do not fill up contraction joints with sealing compound without cleaning with air water jet or sand blast.
8. Check whether any retarders and air entraining agents are added.	8. Do not allow any projections or contraction joint over the surface of the lining.
9. Maintain load register.	9. Do not allow the C.C. Lining without applying suitable primer to sides.
10. Record the No. CC Cubes cast and its compressive strength.	10. Do not remove the channels immediately before setting of C.C.
11. Cure CC Lining with water for 28 days.	11. Do not use untested cement.
12. Ensure smooth surface with paver roller passes.	12. Do not allow to sink the porous plugs in the drains.
13. Ensure the contraction and construction joints as per approved drawing.	13. Do not allow lining without making proper arrangements for curing with water.
14. Check the thickness of C.C. Lining for each panels.	14. The Co-efficient of variation in the compressive strength of cement should not be more than 8%.
15. Checking placing of mastic pad at structures of contrution joints.	
16. Allow concrete lining at temperature between 15°C and 32°C.	
17. Check periodically the coefficient of variation in the compressive strength of cement.	
18. The batching plant to be used shall confirm to the requirements of IS 4925-1968.	

FOR FOUNDATION	
DO'S	**DO NOTS**
1. Verify dimensions and foundation levels as per drawing.	1. Do not forget to compare bearing capacity of actual soils met with design strength.
2. Wet the foundation surface to a depth of 150 mm or to impermeable material.	2. Don't lay the foundation concrete without wetting the surface.
3. Ensure the rock surface free from oil, objectionable coating unsound fragments.	3. Do not lay the concrete under water and over slush.
4. Check-up correct batching of ingredients.	4. The minimum mixing time should not be less than 2 min.
5. Check the batch of cements and its make.	5. Do not forget to keep stand by vibrator and needles.
6. Check-up water cement ratio and slump test.	6. Do not place concrete in raw in sufficiently heavy to wash mortar from concrete.
7. Ensure uniform mixing.	7. Do not forget to cast the cubes.
8. Ensure proper compaction with vibrators and keep stand-by vibrator and needles.	8. Do not allow segregation of concrete.
9. Operate immersion type vibrators nearly in vertical position to vertical drain.	9. Do not use unsatisfactory mix.
10. Cure with water for 28 Days.	10. Don't allow admixtures which will harm the strength of concrete.
11. Compact with suitable bedding materials in case of over excavations and with M-5 grade concrete in case of rock.	
12. Allow admixtures as per I.S. 9103-1979.	

FOR STRUCTURE :	
DO'S	**DO NOTS**
1. Check the form work. 2. Apply cement slurry after cleaning the surface at vertical joints. 3. Clean and cover with a layer of 10 to 15 mm thick mortar of the same proportion of concrete mix for horizontal joints. 4. Place the concrete at temperature between 15ºC to 30ºC. 5. The concrete shall be discharged with in half an hour after introduction of the mix water and cement.	1. Avoid abrupt surface irregularities. 2. Do not deviate from specified dimensions of cross section from - 6mm to +12mm. 3. Do not allow concreting until all form work installation of items to be embedded and preparation of surface involved are approved.

MASONRY (IS codes 1597, 1812, 1200, 383, 269, 2116)	
DO'S	**DO NOTS**
1. The stone shall be of uniform colour, texture, strong, hard durable. 2. Dressing of C.R.S. Stone to a depth of 75mm on all four sides. 3. Wet the stones before placing in position clean and cover with fresh mortar. 4. Place stones in layers to the line and plumb. 5. Provide weep holes at 2 mtrs interval staggered as per drawing. 6. Chisel dress the corner stones. 7. Face stones shall be laid alternately in headers and stretchers. 8. Provide bond stones at 2 mtrs. Interval in each layer and mark. 9. Place the hearting stones on its broadest face. 10. Ensure perfect hearting to make the masonry water tight. 11. Mortar shall be used within 30 min. after discharged from mixer.	1. Do not use stones other than granite of crushing strength less than 1000 kgs/ sq.cm. 2. Do not allow brushing more than 40 mm on the face. 3. Do not allow stones of length more than 3 times the height. 4. Do not allow stone of breadth less than height of ¾ of thickness of wall. 5. Do not allow breaking of vertical joints less than 75 mm. 6. Header shall not project not less than 10 cm beyond stretcher. 7. Do not place stones in position without cleaning and wetting. 8. Do not allow skin stones, weathered stones. 9. Do not place stone in position without wetting.

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DO'S	DO NOTS
12. Sieve analysis for sand shall be done periodically which confirm to : I.S. Sieve Designation — % of passing 4.75 mm — 100 2.36 mm — 90 to 100 1.18 mm — 70 to 100 600 micron — 40 to 100 300 micron — 5 to 70 150 micron — 0 to 15	10. Smallr stones shall not be placed in lower course. 11. Joints thickness should not be more than 12 mm. 12. Do not allow mixing less than 3 minutes for thorough mix. 13. Do not add more water than required to have a consistency of 90 mm to 130 mm.
13. For flush pointing the mortar shall be finished off flush and level with edges of the stones.	14. Avoid spreading of mortar over the surface of the masonry.
14. Joints shall be racked out to minimum depth of 12 mm when the mortar is green.	15. No Pointing to be commenced without washing and wetting the joints thoroughly.
15. Cure the masonry with water for 2 weeks.	
16. Cure the plastered surface with water for 14 days.	
17. Cure the pointing surface with water for 7 days.	

REINFORCED CEMENT CONCRETE SLABS
(Is codes 2502, 1786)

DO'S	DO NOTS
1. Check the reinforcement as per drawing.	1. Do not pass without proper cover.
2. Provide asphaltic pad and water stopper as per drawing.	2. Do not allow less lengths in over laps.

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LINING SUBGRADE

DO'S	DO NOTS
1. Check the model Section to the canal profile i.e., bottom or lining.	1. Do not allow concrete lining on loose subgrade.
2. Check the canal profile with reference to model sections.	2. Do not allow any root or stumps to be on sub-grade.
3. Remove roots and stumps completely from sub-grade.	3. Do not allow lining in expansive soils without treatment with C.N.S. Soils.
4. Compact over-excavation in soils with gravel duly wetted.	4. Do not place the porous plug below the surface of the lining.
5. Compact over -excavation in rocky area with gravel spalls and aggregate spalls and aggregate duly wetted.	5. Do not allow lining without wetting the sub-grade suitably.
6. Provide treatment with C.N.S.Soils in expansive soils i.e., 0.6 mtrs., thick for discharge up to 50 cusecs.	
7. Provide Porous plugs of size 375 mm long x 100 mm dia in each panel with local filters of graded metal and sand size 600 x 600 x 750 mm.	
8. Provide longitudinal and transverse drain of size 600 x750 mm filled up with graded metal and sand as per drawing.	
9. Check whether porous plugs are freely draining or not.	
10. Note down December water table for providing longitudinal drains and other relief measures (as per drawing).	

CHECKING SLIP TO ACCOMPANY THE PROPOSALS FOR DESIGN MIX CONCRETE:		
Package No: Circle: Division		
1 Grade of concrete		M20/M15/Other
2 Whether PCC/ RCC		PCC/ RCC
3 Maximum size of coarse aggregate		mm
4 Intended use and exposure:		
5 Exposure condition		If not 'moderate' reasons for the same
6 Degree of Quality control		If not 'good' reasons for same
7 Required slump		mm
8 Sand grade		
9 Max W/C ratio permitted as per		0.55 for RCC & 0.60
agreement:		for PCC
10 Whether air -entrained OR not -air entrained		if air- entrained reasons for same

11. PROPOSED MIX DESIGN:

W/C Ratio	
Water	Ltrs
Cement	Kgs
Sand	Kgs
Coarse aggregate	Kgs
Total	Kgs
Air-Content	%

12	If air-entrained concrete whether W/C ratio increased accordingly?	
	Reasons for adopting air -entrained concrete:	
	Recommended air -entrained agent:	
13	Whether W/C ratio comply with agreement ?	
14	Recommendation	Accepted / not -accepted

PREMISSIBLE VALUE / FREQUENCIES OF TESTS ON ENGINEERING MATERIALS

SNO	DESCRIPTION OF TESTS	PERMISSIBLE VALUES					FREQUENCIES	REMARKS
1	**Cement**							
	a) Specific Gravity	Greater than 3						
	b) Initial Setting Time	Not Less than 30 minutes						
	c) Final setting time	Greater than 600 minutes						
	d) Specific surface	OPC -Greater than 2250 cm² /gr						
	by Blain's air Permeability						one test from	For more
	e) Cube strength						Consignment having	details
	3 days, 7 days, 28 days						same batch & No. of	refer IS 269
							the same factory	IS 1489
		Cement	Min. Compressive Strength (Kg/sqcm)					
		Grade	3 days	7 days	28 days			
		a) 43 Grade	220	290	430			
		b) 53 Grade	270	370	530			
2	**Coarse Aggregate**							
	a) Gradation							
		IS sieve	**% passing for graded aggregate of nominal size**					
		Desg	40mm	20mm	16mm	12.5mm		
		80mm	100				One test daily	For more
		63mm						details
		40mm	95-100	100				refer IS 383 &
		20 mm	30-70	95-100	100	100		(Part 1 to 8)
		16 mm			90-100			IS 515
		12.5mm			90-100			
		10 mm	10-35	25-55	30-70	40-85		
		4.75mm	0-5	0-10	0-10	0-10		

SNO	DESCRIPTION OF TESTS	PERMISSIBLE VALUES	FREQUENCIES	REMARKS
	b) Specific Gr.	Minimum -2.6		
	c) Water absorption	less than 5% when immersed in water for 24 hours.		
	d) Aggregate crushing value	45 % for Mass Concrete, 30 % for Wearing surfaces	one test Whenever Quarry	
	e) Aggregate impact value	45 % for Mass Concrete, 30% for Wearing surfaces	Changes or strata change	
	f) Abrasion value	50 % for Mass Concrete, 30 % for Wearing surfaces	Occurs	
	g) Soundness test	Less than 12 % when tested with Na_2SO_4		
		Less than 18 % when tested with $MgSo_4$		
	h) Flakiness index	Less than 35 %		
	i) Elongation index	Less than 35 %		

SNO	DESCRIPTION OF TESTS	PERMISSIBLE VALUES					FREQUENCIES	REMARKS
3	**Fine Agrregate**							
	a) Gradation	IS sieve	Percentage passing for grading					
			Zone-I	Zone - II	Zone - III	Zone -IV		
			100	100	100	100		
		a) 4.75 mm	90-100	90-100	90-100	95-100		
		b) 2.36 mm	60-95	75-100	85-100	95-100	One test in morning	For more
		c) 1.18 mm	30-70	55-90	75-100	90-100	One test in Afternoon	Details 383
		d) 600 micron	15-34	35-39	60-79	80-100		& IS 2383
		e) 300 micron	5-20	8-30	12-40	15-50		(Part 1 to 8)
		Note : Where the grading falls outside the limits of any particular grading of zone of sieves other than 600 micron IS sievs by a total amount not exceeding 5% it shall be regarded as falling within the grade.						
		Aggregate conforming to Grading Zone -should not be used in reinforced concrete.						
	b) Fineness Modulii	2.00 -3.20						
	c) % silt & clay content	Less than 3%						
	d) % deliterius materials	Less than 5%					As per requirement	
	e) % Soundness	Less than 10 % When tested with Na_2SO_4						
		Less than 15 % When tested with $MgSO_4$						

SNO	DESCRIPTION OF TESTS	PREMISSIBLE VALUES		FREQUENCIES	REMARKS
4	**Water**				
	a) PH	6 to 8			
	b) Suspended Matter	Less than 0.2 %			
	c) Organic Matter	Less than 0.2 %			
	d) Inorganic Matter	Less than 0.3 %			
	e) Chlorides	Less than 0.2 % for PCC			for more
		Less than 0.1 % for RCC			details refer
	f) Sulphates	Less than 0.05 %			IS 456, IS 516
5	**Concrate**				
	a) Mix design	Cubes should satisfy the target strengths adopted in mix design			
	b) Slump	a) Road work	20 to 30 mm		For more
		b) Beams and Slabs	50 to 100 mm		details refer
		c) Walls	75 to 150 mm	Twice for Shift	IS 456, IS 516
		d) Vibrated Concrete	15 to 25 mm		IS9103, IS 6441
		e) Mass Concrete	25 to 25 mm		IS 1199, IS 3035
	c) Temperature	Between 15 to 32 deg C			S.P 23
	d) Unit weight	2400 Kgs/cum for plain concrete, 2500 Kgs/cum RCC		For mass concrete one specimen for 150 Cum	
	e) Cube Strength	Minimum Characterstic Strength of Proposed mix		For structural concrete four specimens upto 50 Cu.m and 12 cubes /Shift one each element of structure	
	f) Non destructive test by Rebound Hammer	Minimum Characterstic strength of Proposed mix			
	g) Core test	Not less than 75 % of cube strength for individual sample and 85 % on average of atleast 3 sample		One test for 2000 sqm of lining and one test for each element of structure	
		Note : As per gradation analysis at site bin correction for concrete mix shall be given every day			
6	**Mortor**				
	a) Slumps test	For workability		Specimen to be cast at the rate of one sample of 3 cubes for 100 Cu.m of Measonry and tested for their 28 days strength	IS:1597 (P1 & P2)
	b) Unit weight for Proportions of ingradient				
	c) 50 mm size of CM test specimen	tested for 28 days strength			
	d) Permeability test for every 3M	3.0 Lugeons limit for rear face			
		2.5 Lugeons for front face			
7	**Bricks**				
	a) Water absorption after 24 Hours	Less than 20 %			for more details
	b) Size	190 x190 x90mm			IS 1077

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SNO	DESCRIPTION OF TESTS	PREMISSIBLE VALUES	FREQUENCIES	REMARKS
	c) Density			
	d) Compresive strength	Greater than 40 Kg /sqcm		
8	**Rough Stone**			
	a) Soundness		when required	
	b) Abration			For more details
				refer
	c) Water absorbtion	Less than 5% when immersed in water for 24 hours		IS1121, IS1124 &
	d) Density			IS 1126
	e) Crushing strength -Granite	Greater than or equal to 1000 kg/ cum		
9	**Reinforcement**			
	a) Diameter	Satisfy the required diameter	1) The spacing of the	
	b) Weight in kgs/m length	Satisfy the required weigth	reinforcement rods is be	
			checked	
	c) 0.2 % Proof strength	Greater than Yield stress	2) The Overlap lengths	For more details
			Provided shall be checked,	refer IS 2502,
			It shall be 45D	IS 1786, IS 5525
	d) Ultimate tensile strength-	10 % more than the actual		
	for Fe 415	0.2 % Proof		
		stress but not less than	3) The cover shall be checked	
		485 N/ Sqmm		
	e) % elongation	Mild steel -23%	4) The bearing capacity of the	
		HYSD -Fe 415 -14.5%	From work shall be verified	
	f) Bend and rebend test	Satisfactory		
10	**Drilling & Grouting**	After grouting another test hole	The following test are required	
	a) Loging of the drilling	shall be drilled in that area	before starting of grouting	
	Operations	and tested for the	1) The logging of the drilling	
		Permeability	operations and core recovery	
	b) Permeability test		shall be analysed for the seam	
			and the voids in the substrata	
			2) The Permeablity test shall	For more
			be conducted and there is any	details refer
			water loss,will give an indica-	IS 6066, IS
			tion of the cavitation is filled.	5529, IS 4999

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SNO	DESCRIPTION OF TESTS	PREMISSIBLE VALUES	FREQUENCIES	REMARKS
			3) The input materials (i.e.,)	IS 11216 -1985
			cement, sand and mortar shall	
			be got tested	
			The following tests are	
			required during execution of	
			work	
			1) The cement grout consump-	
			tion at every stage of grouting	
			shall be observed & analysed	
			with reference to the core	
			losses previously observed.	
			2) The cement and sand being	
			used shall be got tested at the	
			regular frequency after	
			grouting, another test hole	
			shall be drilled in that area	
			and tested of the permeability	
11	**Lining**			
	a) No fine concrete and porus	Compressive strength for		
	concrete	cylinder 15cm dia & 30cm hit		For more details
		shall not less than 70 kgs/sqcm		IS 3873, IS 6509
		Permeability shall be 500 Lit/		
		min sqm with 10 m head of		
		water		
	b) Filter in expansion joint			
		Asphalt -80 IW -30% by Vol		
		Sand 51% by Vol		
		Cement 17 % by Vol		
	c) CNS soils for bed	Heap cut into length 2% by		IS 3873
		volume Propotion materials as		
		per IS 3873 The soils to be		
		used for CNS layers shall be		

SNO	DESCRIPTION OF TESTS	PREMISSIBLE VALUES	FREQUENCIES	REMARKS
		tested for the following		
		properties clay (<2 micron) -		
		15 to 20% silt		
		(0.06mm -0.002mm)-30 to 40%		
		sand (2mm -0.06mm) 30 to40%		
		grains (2>mm)0 to 10%		
		plasticity index less than 30%		
		but >15% It should have 'C'		
		and ϕ ranging from 10.342 Kn/m²		
		and 25° to 20.684 to		
		27.579 Kn/m² and 12° to 14°		
	d) Morrum back fill behind revetment	The material shall; be tested for		
		the following properties Fine		
		passing 75 micron < 10%		
		Liquid limit < 20%		
		Plasticity index < 6%		
12	**Bearings**			
	a) Hardness	60+5 IRHD		IS : 3400
	b) Min Tensile strength	17 Mpa		IS : 3400
	c) Min. elongation at break	400%		IS : 3401
	d) Max. compression at test			IS : 3402
	e) Accelerated aging			IS : 3403
	f) Max. change in Hardness	+ 15 IRHD		IS : 3404
	g) Max.change in tensile strength	-15%		IS : 3405
	h) Max.change in elogation	-40%		IS : 3406
13	**Rubber water stop**			
	a) Tensile strength	Min 116 kg/sqm		For more
	b) Ultimate elongation	Min 300%		Information
	c) Tear resistance	Min 49 kgs/sqm		refer IS 9766
	d) Stiffness flexure	Min 24.6 kgs/sqcm		
	e) Alkali test	After 7 days of immersion,		
		weight shall increase by 0.25%		
		and decreaase by 0.1%		
		After 28 days immersion.		

SNO	DESCRIPTION OF TESTS	PREMISSIBLE VALUES	FREQUENCIES	REMARKS
		The weight shall not increase by		
		0.4% or decrease by 0.3%		
		The dimension shall not alter by 1.0%		
14	**Bearing capacity of strata**	Should be ensoured for the	For all the important	
		stress indicated in design/	structures	
		drawings		
15.	Embankments			For more details
	a) Cut-off - Permeability	Less than 10' cm / sec		refer
	Filters - test for metal / sand			IS USBR earth
	Gradation for metal	IS Sieve % passing		manual
		80 mm 100		IS-1498, 1970, 6955
		40 mm 80 - 56		1973, IS-5529-
		20 mm 84 - 24		1985, 11293,
		10 mm 64 - 8		11532, 14650,
		4.25 mm 25 - 0		14954.
	i) Uniformity Coefficient	Cu - D60 / D 10 >6 for sand		
		>4 for metal		
	ii) Coeffiecient of Curvature	Cc = $D30^2$ / (D10xD60) lies		
		between 1 to 3		
	iii)	$\frac{\text{D 15 of filter}}{\text{D 15 base}}$ >4 and <20		
	iv)	$\frac{\text{D 15 of filter}}{\text{D 85 of base}}$ <5		
	v)	$\frac{\text{D 50 offilter}}{\text{D 50 of base}}$ <25		
	vi) Relative Density	$RD = \frac{\text{E max -E}}{\text{E max - E min}}$		
		individual sample should not be		
		less than 70% and average		
		shall not be less than 85%		
		E max=voids ratio in loose state		
		E min=voids ratio in compacted		
		state		

SNO	DESCRIPTION OF TESTS	PREMISSIBLE VALUES	FREQUENCIES	REMARKS
		Thickness of each lalyer shall		
		be less than 150 mm		
	c) Compaction		One test whenever borrow	for more details
			area changes	Is 4701, IS 2720
				(Part 1 to 30)
	1) Gradation using Hydrometer			
	analysis			
	2) Attenberg limits	tested for suitability for casing		
	3) Sp. Gr & voids ratio	and hearting soils as per design		
	4) Proctor compation	>98% of Proctor density	For 1500Cu.m of Earthwork	
	(Field density)	controlled within + or -1% omc	or in each layer laid on	
	d) Swell pressure	If swell pressure>50 KN/Sqm,	embankment	
		CNS layer shall be provided as		
		per IS 3873, 9451		
	e) Difference free swell	If DFS > 50, swell pressure test		
		shall be conducted		
	f) Total soluble solids present			
	g) Suitability of soils	Based on soil classification /	One test in one week	
		designs	of every 3 m of embankment	
	h) Stability of soils	Based on soil classification /		
		designs		
	i) Permeability	Field permeability by Japanese		
		method 10^{-7} cm / sec		
	j) Revetment bed			
	pitching & rock toe	as per item 2,3,8		
	k) Shrinkage factor		One test in a week or	
			5m of embankment	
	l) Triaxial shear test	Tested for C& ϕ values	One test in a week,	
			every 3 m of embankment	
	m) Std proctor test	Tested for 98% PD	One test per day for individual	
			borrow area & moisture	
			and content	

As per Engineer -In Chief (Irrigation) circular DCE -1/O T/ MP/AEE/ 29384 /2005 -1 dt:26.12.05

a) 10% of the tests conducted by the EPC agency should be got checked by the Third Party Quality Control Agency apart from conducting independent tests.

b) Tests conducted be EPC agency and Third Party Quality Control agency should invariably be checked by department Quality Control staff.

16	MECHANICAL TESTS	MANUFACTURING STAGE	REMARKS
	1. HYRAULIC GATES AND HOISTS &		
	ALLIED EQUIPMENT		
	a) Manufactruing & installation stage	a) E.M. parts, gates and hoists.	
	b) Straight & co-planary accuracy	b) Straightness and co-planary accuracy be	
		checked for all materials and its components jigs.	
	c) Fabrication tolerances	c) Fabrication tolerances checked and	
		maintained as per IS 7215-1974 etc.	
	d) Weldments	d) inspection and testing of weldmnets (as per	
		IS 822;823-1964:816 3658-1980:8780-1978:	
		3664-1981:2595-1978 and ASME-Section.	
	e) NDT Testing	8) And latest addition for size of welds. Edge	
	1) liguid penetrate testing	preparation-weld procedures for various joints.	
	(IS 3658-1980)	Joint preparation-type of electrodes-root gap.	
		Selection of electrodes for each component as	
	2) magnetic particle testing	per the standards and recommendation of	
	(IS 3703-1980)	manufactures for its load bearing capacity and	
		strength of welds. Preheating of electrodes weld	
	3) ultrasonic testing (IS 3664-1981)	travels speed for automatic welds and manual	
		& ensure follow of weld designs and procedures.	
	4) radiographic testing	e) Testing for soundness and strength of	
	(IS 2595 & IS 1182)	welds (DPT, X rays and gamma ray)	
	f) Destructive testing	f) (1): tensile, compression, elongation, hardness,	
	1) Physical properties	impact, torsion etc.,	
	2) Chemical properties	f) (2): wet analysis, spectra analysis	
	g) Rubber seals :	g) Shore's hardness and water absorption	IS: 3400-1980
	h) Steel wire ropes	h) wire rope and lifting socket assembly, load	IS: 2365-1977 & 2226
		testing of rope and socket assembly	
	i) Welding electrodes	i) As per IS codes.	IS : 1442-1964,
			IS: 814-1974,
			IS: 815-1974/E-7018
	j) Paint Tests	j) Surface preparation sand blasting and painting	IS: 14177-1994
		as per specification.	

	MECHANICAL TESTS	MANUFACTURING STAGE	REMARKS
	II) Tests for Radial Gates	II. Tests for machined items: test for allowances	
	a) Pre tensioning in anchor rod	and tolerances as per drawings BIS specifica-	
	b) Trunnion centre: coplanar and	tions for all items and components for which	
	parallel to sill	machining operations are involved. (such as:	
		turning, planning, grinding, milling, gear,	
		hobbing, drum scouring, drilling, boring and	
		slotting etc.	
	c) Inclinations of yoke and anchore	☛ Dimensional accuracy and critical	
	girder and its co-plane accuracy	dimensions of E.M parts at least in 300mm	
		intervals on asssembly.	
	d) Radius of skin plate & wall plate	☛ Assemblies and sub-assemblies as per	
		drawings and specified tolerances (Is codes	
		7718.10096-1986, etc)	
	e) Side seal face to face	☛ Dimensional accuracy and critical	
	f) Level of horizontal girder	parameters of components / assembly of gates	
	g) Site weld testing strength	as per IS codes.	
		gate assembly * roller assembly * scale	
		assembly * guide roller assembly * roller cage	
		assembly * trunnions and pins assembly*	
		yoke girder assembly * support / chair * end gear	
		box units. assembly. Central drive units	
		assembly * control panels *wire rope and lifting	
		socket assembly * gate lifting measuring dial	
		indicator gauge.	
		☛ Shop painting - surface preparation	
		painting process	
		☛ Tests after Heat treatment of specified	
		components.	

	MECHANICAL TESTS	MANUFACTURING STAGE	REMARKS
	Additional tests for Radial Gates	* Pre tensioning of anchore rods* trunnion	
		centres: co-planer and parallel to sill * chair	
		level/rest plate: co-planer and parallel to sill*	
		inclination of yoke and anchor girders and its	
		co-planar accuracy *radius of skin plate and	
		wall plate * side seal face to face* level of	
		horizontal girder* radius of skin plate and wall	
		plate * pre tension of anchorages * site weld	
		design and sequence * site weld testing for	
		strength and quality as per IS codes.	
		• Only one longitudinal joint permissible.	
		• Welding shall be tested @ 5% by radiography	
		and 95% by ultrasound testing.	IS 800-1984,
	III) water conductor and penstocks	☛ Checks of dimensional accuracy (Cylinder	IS 226-1975,
	a) Raw material tests-tensile,	dimensions, angle incase of bends, coplanar	IS 814-1991,
	bend physical tests etc.	accuracy of flanges after welding and/or	IS 816-1969,
		jointing shells/pipes etc.	IS 3589-2001,
	b) Welding tests	☛ Welding-submerged arc/manual arc	IS 5822-1994,
		welding of shells and bends.	IS4353, IS 2825,
	c) NDT	☛ DPT, RT, UT etc.	IS 4225, IS 823,
	d) Hydrostatic test	☛ Hydro static test-2 times of working	IS 2595 & IS 1182,
		pressure.	IS 3664-1981,
	e) Paint test	☛ Painting: surface preparation, sand	IS 3658-1980,
		blasting & painting as per specification.	IS 3703-1980
		INSTALLATION / ERECTION	
		☛ Thorough testing & check up of	
		components for quality and dimensional	
		accuracy be done on shifting to erection site.	
		Ensure any defect unnoticed during fabrication	
		should be rectified before installation.	
		☛ Checking of critical dimensions of all	
		E.M. parts at least in 300mm intervals (on	

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	MECHANICAL TESTS	MANUFACTURING STAGE	REMARKS
		assembly and installaion) during erection, before & after concreting, as per IS:4622, and 7718 etc. as applicable with in tolerance & limits. *roller track centre to centre-seal track centre to centre-side guide centre to centre* verticality of roller track seal track. Side guide track * co-plenary of roller track and seal track - horizontally of sill beam - levels of sill beam and hoist bridge * roller assembly and its alignment-side roller gaps.	
		☛ Testing of welds-for strength and soundness-DPT. X rays and gamma ray.	
		☛ Dry testing of rubber sealing and pre-compression	
		☛ Alignment of line shafts.	
		Drain holes to horizontal beams, trunnion brackets hoists bridges.	
		☛ Alignment of gears & shafts.	
		☛ Sand blasting and painting (IS 14177-1994) surface cleaning for abrasive blasting (Swedish standards: S. A2.5 white/rough) and or IS: 5905-1989) paint specification binder solids, pigments, additives solvents, driers etc:- Thickness of paint in microns.	
	IV) Lift Irrigation machinery		
	a) Pump sets		
	1) Standard running test		
	2) NPSH test	☛ Pumps shall be tested in accordance with stipulations of Hydraulic institute standards as applicable.	
	3) DP tests on shaft and impeller	☛ Mechnical Balancing, Visual inspection.	
	4) Noise & Vibration	☛ Noise and vibration shall be measured at shop and to be repeated at site also.	

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	MECHANICAL TESTS	MANUFACTURING STAGE	REMARKS
	5) Non-destructive test 6) Field Testing	☛ After installation, the pumps shall be operated to Prove satisfactory performance.	As per relevant BIS Specifications.
	b) Motor		
	1) HV tests	☛ HV tests on field coils assembled on poles	
	2) Impedence & voltage test	☛ Impedence & voltage test on field coils.	
	3) Static flux test	☛ Static flux test.	
	4) Calibration of over speed devices	☛ Calibration of over speed devices	
	5) Insulation resistance test	☛ Insulation resistance test for accuracy of stater and rotor windings RTD & BTD tests on excitation & Regulation equipments	
		☛ Testing of all electromagnetic valves and pressure switches.	
	6) Type Tests	☛ On the completely assembly motor and associated auxillaries shall be carried out at works.	As per relevant BIS Specifications.
	c) Earthing & lightening & protection	Type tests, Galvanizing tests, Welding tests, Earth continuity test, Earth resistance tests of total system.	IS: 2309-1989, IS 3043-1987 IS: 2633-1986 IS: 4759-1984 IS: 3725-1966
	d) Switchgear, Power & control cables instrumentation and control - data Communication systems-other reqd. equipment	As per specifications & standards	As per relevant BIS Specifications

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CATEGORIZATION OF DEFECTS OF TESTS RESULTS FOR ANALYSIS

In general the defects/short comings for not confirming to the required standards of the tests performed for ensuring QA/QC of the works both in-situ and in the laboratory, may be placed under the following category depending upon the type of tests and the site conditions.

Category A: Failure of tests and no scope for rectification.

Action : 1. To be rejected.
2. If alredy used - **Agreement concluding Authority** to assess the impact and initiate action either by recovery if not detrimental else reconstruction by removing the same at the cost of the agency.

Example : 1. In case of foundations, SBC not confirming but foundations already executed, then no payment, immediate analysis of impact, alteration of design if possible, recovery as penalty else reconstruction of the work component at the cost of the agency.

Only Agreement concluding Authority is authenticated for necessary action.

2. Similary for borrow area conveyance for earth work, CNS soils etc. Upon testing if found not suitable shall not be allowed. If already used, the earth, shall be removed if it is detrimental else suitable recovery as penalty to be imposed duly carrying out the rectification.

Only Agreement concluding Authority is authenticated for necessary action.

Note : Certain tests categorized under category B will also be escalate to under this category, based on the site conditions. The Head of the department is the authority to take necessary action.

Category B : Non Confirming to specified limits but scope for rectification.

Action : 1. Analyse and go for remedy action within the time frame.
2. Remedy action out timed, then to be secalated to category A, for action from the agreement concluding Authority.
3. If escalation from Category C then unit officer in coordination with Superintendent Engineer, Quality Control to study the impact and propose recovery accordingly.
4. If the above beyond rectification, immediate stopage of payment and to escalate it to category A from agreement concluding authority to initiate necessary action.

Example: 1. Tests for cement, OMC for embankment, P.D. test etc.

Category C : Non conformity of initial tests for materials, preparatory tests, linear measurements etc.

Action : 1. To be rejected and removall of the same from the site to be ensured.
2. After rejection if used, then to escalate to category B for unit officer to take to necessary action in coordination with Superintendingt Engineer, Quality Control.

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EXISTING TEST DETAILS IN QUALITY MONITORING SYSTEM

S.No.	Material Name	Test Name	Category of Defect
1	Borrow Soils	OMC	A
	Borrow Soils	Soils Classifications	B
2	Soils	Attenberg limits	C
	Soils	Permeability test	B
	Soils	Shear Test	B
3	Embankment	OMC	B
	Embankment	Proctor Density	B
	Embankment	Cut-off permeability	B
	Embankment	Filters	B
	Embankment	Shrinkage, Swel etc.	B
4	Lining	CNS Soils	B
	Lining	Filter in expansion joint	B
	Lining	Morrum Back fill behind revetment	C
5	Foundation Earth	Safe bearing capacity	A
6	Cement	Specific Gravity	B
	Cement	Cubes Strength	B
7	Coarse Aggregate	Flakiness	C
	Coarse Aggregate	Gradation	C
	Coarse Aggregate	Impact Test	C
	Coarse Aggregate	Abrasion	C
	Coarse Aggregate	Soundness	C
	Coarse Aggregate	Crushing	
8	Fine Aggregate	Slit Content	C
	Fine Aggregate	Gradation	C
	Fine Aggregate	Bulkage	C
	Fine Aggregate	Silt & Clay content	C
	Fine Aggregate	Soundness	C

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9	Water	PH	C
	Concrete	Compressive Strength	A
	Concrete	Slump	B
	Concrete	Core Test	A
10	Mortar	Slump	B
	Mortar	Permeability	B
11	Bricks	Water Absorption	B
	Bricks	Compressive Strength	B
12	Rough Stone	Soundness, Abrasion, Density, Compressive Strength	C
13	Reinforcement	Diameter, Weight, 0.2% Proof Strength	C
	Reinforcement	Ultimate Tensile strength	B
14	Foundation Earth	Safe bearing capacity	A
15	Gates	Critical Dimensions for embedded parts	B
	Gates	Full load Test	A
	Gates	No Load Test	A
	Gates	Critical Dimensions	B
16	Motor	Load Test	A
	Motor	SCADA Test	A
17	Pipes	Load Test	A
18	Pressure Main	Welding Test	A
		Hydraulic Load Test	A
19	Pumps	Hydrostatic Test	A
	Pumps	Full Load Test	A
	Pumps	Testing for Vibrations	B
	Pumps	SCADA Tests	B
20	Stone	Size	C

Note : Initially defects will be categorized based on the test name and later based on the site conditions provision will be given escalate from C to B to A etc.