BANGLADESH WATER DEVELOPMENT BOARD

STANDARD DESIGN MANUAL

VOLUME-I: STANDARD DESIGN CRITERIA

PREPARED BY
STANDARD DESIGN MANUAL COMMITTEE

OFFICE OF THE
CHIEF ENGINEER, DESIGN, BWDB
72, GREEN ROAD, DHAKA

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PREFACE

The Standard Design Manual has been prepared to make available to the designers the information needed for preparation of the detailed engineering designs of various Flood control, Drainage, Irrigation and Road Structures generally undertaken by Bangladesh Water Development Board.

Material used in preparing the manual was drawn from various sources e.g. text books, journals, manuals and other usual practices followed in Design offices. Reference of these sources is made at the end of relevant chapters.

The sub-committee entrusted to the preparation of Standard Design Criteria, Standard Design of Structural Members, Standard Layout Plan and Drafting Details and Standard Design of Hydraulic Gates took the arduous task of doing the job in addition to their normal work load.

Mr. Mukhlesuzzaman, Superintending Engineer, Design circle-II, Mrs Khaleda Shahriar Kabir, Superintending Engineer, Design Circle-VI, Mr. A.N.M. Wahedul Huq, Superintending Engineer, Design Circle-I, Mr Md. Anwar Hossain Bhuiyan, Superintending Engineer, Design Circle-V and Mr. Aminul Haque Shah, Superintending Engineer, Design Circle, North Eastern Zone and Mr. Matiur Rahman, Superintending Engineer, Design Circle-III supported by the Executive Engineers, Sub-divisional Engineers, Assistant Engineers and Draftsmen selected for each sub-committee prepared the draft Design Manual for BWDB. Mr. Mukhlesuzzaman, Superintending Engineer, Design Circle-II, co-ordinated the works of all Sub-Committee and Prepare, type and print the report in Design Circle-II, for which contribution of Mr. S.M. Azizul Hoque, Mr. Babul Chandra Shil, Mr. Jiban Krisna Das, Sub-divisional Engineers, Design Circle-II, Mrs, Kamrunnahar, Sub-divisional Engineer, Design Circle-I, Mr Wahid Hussain and Mr. Sohel Masud, Assistant Engineers, Design Circle-II need to be mentioned specifically.

The draft report has been edited by Mr. Mukhlesuzzaman, Superintending Engineer, Design circle-II, Mr. Md. Anwar Hossain Bhuiyan, Superintending Engineer, Design Circle-V, Mr. A.N.M. Wahedul Huq, Superintending Engineer, Design Circle-I and Mr. Aminul Haque Shah Superintending Engineer, Design Circle, North Eastern Zone under the overall guidance of Mr. Syed Shahadat Hossain, Additional Chief Engineer, Design.

Comments and suggestions on draft Design Manual received from different corners has been incorporated in the final version of the Design Manual, so long the comments are found relevant to the contents of the Report.

The final version of the Design Manual has been rearranged and re-edited by Mr. Mukhlesuzzaman, Superintending Engineer, Design Circle-II, with the Help of Mr. Anwar Hossain Bhuiyan, Superintending Engineer, Design Circle-V and Mr. A.N.M. Wahedul Huq, Superintending Engineer, Design Circle-I

The Standard Design Manual is expected to meet the requirement of adopting uniform design criteria, procedure and drafting practices in all the Design Offices of Bangladesh Water Development Board.

Syed Shahadat Hossain
Additional Chief Engineer,
Design, BWDB.
72, Green Road, Dhaka-1215.

INTRODUCTION

The Aide Memoir finalized on August, 28, 1994, between GOB and World Bank Mission recorded some recommendations and findings to improve the performance of Water Sector. The recommendation to improve the performance in Sectoral Planning, Project Preparation, implementation and operation maintenance of Water Sector included Standardization of System Elements/Design and developments of Standard criteria/methods to ensure multipurpose design of project System and elements. As per Aide Memoir, a detailed work plan to achieve the objectives was submitted to the Board by December 1994.

The objectives on Standardization of design as submitted in work plan are:

- To maintain uniform design and drafting practice in all the design office.
- To prepare typical design and drawings of structures or elements of similar category to avoid time consuming detail calculation in every aspect.

With the objectives mentioned, the following aspects were proposed to be included in the Standardization of design:

- A uniform design criteria and Procedure.
- Standard Design of Structural Members containing design tables and charts.
- Standard layout Plan of Individual and Typical Structures.
- Reinforcement drawings introducing Bar Bending Schedule.
- Standard detailing and drafting guideline.
- Drawings of Hydraulic Gates and Hoists for Standard Size.

Accordingly BWDB, vide its no. 190-WDB/Sectt/Imp-2 dt.8-4-1995 constituted a committee to formulate the Standard Design Manual, with the instruction to complete the task by June '95. Board's order issued in this respect contains the suggestions given in the work plan to prepare and submit the reports on Standardization in five volume as follows:

Vol. I.	Standard Design Criteria
Vol. II.	Standard Design of Structural Elements
Vol.III.	Standard Layout Plan of Hydraulic Structure
Vol. IV.	Drafting & Detailing Standard
Vol. V.	Standard Drawings of Hydraulic Gates

The Standard Design Manual, prepared by the Committee, under the overall guidance of the Chief Engineer, Design-I, contains the suggestions given in Aide Memoir and follows the layout of reports and contents given in work plan as approved by the Board vide its no. 190-WDB/Sectt/Imp-2 dt. 8-4-1995. The contents of Design Manual in brief are:

Volume-I of Standard Design Manual contains Standard Design Criteria, design constants, useful formula and basic guidelines for selection of a particular type of structure. This volume comprises of 10 Chapters.

The subjects covered are Hydrology, Hydraulics of flow, Structural design criteria and design constants by Working Stress Method and Ultimate Strength Method of design, Prestress design methods, Foundation Design, Embankment Design, Design of Irrigation Structures, Design of Road Structures, Design of Bank protection and River Training works and Design of Gates and hoists.

Volume-II contains Standard design of elements, needed frequently for different Hydraulic Structures.

Volume-III contains Standard layout plan of Hydraulic Structure.

Volume-IV contains Standard detail of Drafting and detailing.

Volume-V contains Standard Drawing of Hydraulic gates and hoists for Standard Size.

The design criteria is prepared with due attention to appropriate technology taking into account prevailing practices followed in all the Design offices of Bangladesh Water Development Board.

The design manual prepared for the design offices of BWDB, with an aim to eliminate a one-off approach for each individual system element, will guide the designers to select a particular type of layout for certain category of structure from the Typical layout (Volume-III) and detail of sections/elements from the relevant Chapters of the Design Manual.

List of Personnel involved in preparation of "Standard Design Manual" under reform task no-26 of BWDB.

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1.0 GENERAL HYDROLOGIC DESIGN FOR DRAINAGE STRUCTURES

1.1 General

The hydrological design standards for the drainage system can broadly be divided into the standards for the drainage canals and standards for the regulators. The basic standards are the same, the requirement of post-monsoon drainage in non-tidal area and structural safety considerations give rise to additional design standards for the regulators.

1.1.1 Drainage Canal

To determine drainage discharge of a canal section, the drainage rate or drainage module expressed in mm/day has to be transformed into drainage discharge rate expressed in l/sec or cum/sec. In a particular reach of channel the drainage discharge shall be drainage module (mm/day) x area (ha) x 1/8640 m^3/sec.

1.1.2 **Drainage Module**

A design 10-day rainfall of a 1 in 10 year return period occurring during the pre-monsoon period of May and June will be used to derive the drainage module.

The derivation of drainage module will be based on the following criteria:

(i) An initial pre-storage level of 50mm.
(ii) A deep percolation loss of 50 mm/day.
(iii) An evapotranspiration loss of half the average potential rate.
(iv) Little or no submergence above 150mm from ground level during the storm period.

The drainage module D_m can be derived by a relatively simple graphical analysis on the basis of the above criteria or by the method described in publication 16 of The Institute of Land Reclamation and Improvement (ILRI). The module can also be computed from the following equation:

$$D_m = \frac{I_p + R_t - T(E_t + S) - F_{1t}}{T}$$

where,

I_p = pre-storage (assumed as 50 mm)

R_t = cumulative rainfall for T days (T assumed as 10 days)

E_t = evapotranspiration in mm/day (assumed as half the average potential rate for May-June).

S = deep percolation (mm/day)

F_{1t} = long-term flood depth (assumed as 150 mm)

The above formula is valid if the line of maximum slope on the net cumulative rainfall curve drawn from the long-term flood depth (F_{1t}) at zero day passes through the first two days cumulative rainfall points.

1.2 Drainage Regulator (Non Tidal Area)

1.2.1 Design Standards

To ensure satisfactory drainage performance as well as to ensure security of the structure, the design of the drainage regulator will be carried out to satisfy the following criteria corresponding to 1 in 10 and 1 in 25 year return periods respectively.

a. A pre-monsoon routing using 10-day rainfall in the basin with 1 in 10 year return period and 1 in 10 year river water level during critical drainage period, will be used in assessing the following requirement:

   The incremental area inundated in addition to the existing low lands that cannot be drained by gravity to depth greater than 300mm for a period of 3 days should be less than 5 percent of the basin area.

b. A post-monsoon routing using 1 in 10 year rainfall within the basin and 1 in 10 year 10 day average water level in the outfall channel will be carried out to check that the head difference across the regulator during post-monsoon drainage (September-November) should be within 250mm to 300mm for a prolonged period and upto maximum 500mm for 3 consecutive days.

c. To ensure security of the structure against seepage, uplift, scour and excess energy of flow, the design analysis will be carried out for a 1 in 25 year return period pre-monsoon storm. The tail water level corresponding to the design storm shall be based on condition anticipated at the time of occurrence of such event.

d. Where flushing is not a requirement, the country side apron, cutoff wall, scour and seepage protection will be designed with a polder water level corresponding to 1 in 2 year storm and an outfall channel stage corresponding to a 1 in 20 year flood level.

1.2.2 Pre-monsoon routing

The pre-monsoon flood routing will be carried out using 10 day rainfall generally for the period April to June for the different return periods described in 1.1.1. An inflow hydrograph will be derived using the Snyder's unit Hydrograph Method for which the following characteristics are considered:

- Basin area
- Length of main drainage channel
- Distance to basin centroid
- Average channel slope
- Channel roughness factor

Rainfall excess will be computed on the following assumptions:

a. Paddy land

- Initial depth of storage in basins 100 mm
- Initial soil moisture deficit nil
- Subsequent percolation and evaporation loss [Art.1.5.2(d)]

b. Non-paddy land

- Initial soil moisture deficit 12.5 mm
- Subsequent evaporation and percolation loss 25.0 mm/day
- Depression storage over the area at maximum constant rate of 0.80 mm/hour provided rainfall is available, until a total of 25.0 mm is stored.

1.2.3 Post-monsoon Routing

The Post-monsoon drainage starts with the recession of outfall river stages. The post-monsoon routing will be carried out using the 1 in 10 year 10 day rainfall for the period after drainage is allowed by outfall river stage.

Inflow is computed separately for the non-flooded area and flooded area as follows:

a. Non-flooded area

Effective rainfall is calculated as the rainfall less soil moisture deficit. The deficit is computed assuming actual evaporation E is (SM/CAP)xET, where SM is the relevant soil moisture content and CAP the field capacity (taken as 100mm) and ET is the potential evapotranspiration. Deep percolation (at the specific rate until the end of July, and zero thereafter due to saturation of the underlying aquifer) is subtracted from the soil moisture reservoir.

b. Flooded area

The soil is assumed to remain at or above field capacity and effective rainfall is computed as Rainfall-ET-Deep percolation. Deep percolation varies as stated above.

The computations are carried out on a daily basis assuming uniform rainfall intensity for the rainfalls stored during the period.

The design outfall river water levels are taken from a typical hydrograph derived from statistical analysis of outfall river levels.

1.3 Drainage Regulator in Haor Area

1.3.1 General

The submersible embankments are designed to provide flood protection and prevent entry of flood water into the sub-project area upto 31st May.

The Regulating structures will be designed to:

- drain out excess water during the pre-monsoon (if the water level in the outfall Channel permit).

- provide flushing to fill the protected area to such a level that when overtopping of the embankments occur it does so at safe hydraulic head. Head difference immediately before overtopping should not exceed 0.30 m. Average year hydrograph of peripheral channel should be basis for flushing after harvesting.

- drain the accumulated flood water from the sub-project during the post-monsoon season with permissible velocities and discharge rates to ensure no damage of the structure. The head difference for this type of drainage shall be as per post-monsoon drainage criteria of regulator in non-tidal zone. The cultivable land within the protected area shall be free from water as soon as outfall river water level permits.

- Introduce and retain water in the internal canals for lift irrigation during winter.

- provide fish passage or make suitable flow condition for fish migration.

- provide boat pass (if required) during flood time.

1.3.2 Hydraulic Head

Design capacity of the regulators in the haor area will be determined on the flushing and drainage requirement i.e. the need to fill up and drain the sub-project area as per requirement. Flushing should start on 1st June with gradual opening of gates to prevent channel erosion. The following criteria shall be adopted for determining the capacity of such regulators:

- The hydraulic head is limited to 0.30 m at the time of overtopping of the embankment.

- The hydraulic head is limited to 0.230 m to 0.30 m for prolonged period and 0.50 m for not more than 5 days during post monsoon drainage.

- Weir flow through at least one of the opening to facilitate fish migration.

1.3.3 Inflow Routing

A river water flow routing will be carried out beginning on 1st June to determine the opening size of the structure which will meet the criteria of safe hydraulic head. The following initial condition shall be determined :

- river water level on 1st June shall be determined with respect to 1 in 10 year return period and the year corresponding to this level shall be selected as design year.

- the initial storage volume on 1st June shall be assessed as the accumulated rainfall-runoff over the project for an average year condition plus the permanent storage corresponding to minimum haor water level.

- initial water level of the haor corresponding to the initial storage shall be computed from storage-elevation relationship of the haor area.

Structure size shall first be determined by considering the design year. later the size shall be tested for several historical years of data and a suitable size shall be recommended.

1.4 Drainage Regulator (Tidal Area).

1.4.1 General

Drainage sluices in tidal area are designed to drain the polder area upto design level. A tidal drainage sluice with lowest possible invert level and a flap gate at the river/outfall side is the most appropriate structure. To control the drainage flow and to maintain certain water level inside the polder, the drainage sluice is to be equipped with a sliding gate at the country side.

During certain period of the year, the drainage sluice may be used as inlet structure in most of the areas. provision of such type of use of sluices are to be made where required.

To estimate the design discharge or drainage module of the drainage basin the method described in Publication 16 of the Institute of Land Reclamation and Improvement (ILRI) may be used. A detail example is available in the Design Manual, Delta Development Project. To determine the drainage requirement rainfall frequencies have to be worked out.

Sluice dimension is calculated, based on the average design drainage discharge. During stages of design drainage discharge the polder water level may be assumed to be halfway in between the design drainage level (d.d.l) and maximum storage level (m.a.s.l). In monsoon period the design drainage level corresponds to the required water level in the rice fields.

The design drainage discharge through the sluice is to be calculated at average tide condition of the river water. Average tide is defined as the average tidal amplitude ranging from M.H.W (mean high water) to M.L.W (mean low water) at average river level (F.M.L = fortnightly mean level) for the considered time of the year.

1.4.2 Drainage Routing

Drainage routing shall be carried out using 1 in 10 year 10 day rainfall amounts for the period June-July in similar manner to the pre-monsoon routing described above. A 10 day storm of 1 in 10 year return period shall not cause submergence of more than 5% of the incremental area in addition to the area that can not be drained by gravity to a greater depth than 0.3m for a period of 3 days. Alternately, drainage module for the desired period may be calculated as per ILRI method. A design tide curve shall be considered as tail water level.

For maximum velocity during drainage conditions, the river water is taken at L.L.W.S (lowest low water spring tide), exceeding 50% of the years of record for the considered period of the year. The polder water level during this stage is considered to be at maximum allowable storage level i.e. (+) 0.30m above design drainage level (d.d.l).

1.4.3 Inflow Routing

During certain period of the year the sluice may be used to let in fresh river water into the polder. Polder water levels during these stages is assumed to be lower than the design drainage level.

To account for most unfavourable conditions to calculate the maximum inlet flow, the polder water level is assumed at 0.30m below average ground level of the lowest drainage unit. For the river water, the H.H.W.S (highest high water spring) exceeding during 50% of years of record, for the considered time of the year, is to be taken.

1.4.4 Head and Design Discharge

The design head and design discharge for the calculation of seepage and uplift pressure, design of cut-off wall, scour protection and stilling basin will be derived from 1 in 25 year monsoon routing.

1.5 Flushing Regulator

1.5.1 General

Flushing regulators shall be designed based mainly on irrigation requirement and limited drainage requirement.

1.5.2 Irrigation requirement

The irrigation requirement is calculated as follows:

a. *Crop water requirement (ET_c)*

These will be based on the proposed cropping pattern. Potential crop evapotranspiration (ET_o) for the project area and crop coefficient for different crop types can be obtained from Table 1.1 and 1.2 respectively.

b. *Land preparation requirement*

Land preparation usually takes place over a period of one month before the crop is transplanted. An allowance of 200 mm is suggested as the land preparation requirement for HYV Boro and T Aus crops. The requirement of T Aman shall be less than other rice crops due to higher residual moisture and rainfall during the monsoon period. Hence a lower land preparation amount of 100 mm is therefore suggested for T Aman.

c. *Effective rainfall*

This is based on 80% dependable rainfall. A rainfall factor of 0.4 during land preparation and 0.7 during crop growth is suggested to determine effective rainfall.

d. *Percolation losses (paddy crops)*

Percolation losses from the paddy fields depend on type of soil. Suggested typical values for different soil types are:

soil type	losses (mm/day)
sandy	10.0
Loamy	7.0
Clay loam	5.0
Clayey	2.0

If no soil data is available the percolation losses will be determined by crop type:

crop type	Losses (mm/day)
Aus	3.0
Aman	2.5
Boro	2.0

e. *Overall Efficiency*

The overall application efficiency losses (FE) will be estimated as 50% (non-paddy) and 60% (paddy).

The diversion requirement will be calculated as follows:

(i) Crop water requirement (ET_c):

= potential Crop Evapotranspiration (ET_o) x Crop co-efficient (k_c).

(ii) Gross Irrigation requirement or Diversion Requirement

= [ET_c + Land preparation (LP) + Percolation losses - Effective rainfall (ER)]/Overall Efficiency (FE).

1.5.3 Design of Opening Size

The design discharge for which the opening size (vent size and number of vents) of a flushing regulator shall be determined, is the gross irrigation requirement or diversion requirement for a particular area. The diversion requirement is the irrigable area multiplied by the overall duty (m^3/sec/ha) at the point of diversion.

1.5.4 Hydraulic Design Conditions

The hydraulic design with respect to energy dissipation, seepage, uplift and scour shall be determined under the following conditions:

1.5.4.1 Design Discharge

For the design of country side stilling basin and scour protection works, design discharge shall be two times the design discharge as mentioned above. The corresponding water levels shall be 80% dependable river water level and country side water level occurring from 80% dependable rainfall.

1.5.4.2 Design Head

The head difference for design of country side apron and floor shall be the maximum difference of 1 in 20 year river flood level and 1 in 5 year minimum basin water level occurring during the months of July to September.

The head difference for design of river side apron and floor shall be the criteria described in Article 1.2.1.(b).

1.6 Opening Size/Vent size

1.6.1 Standard Design of opening size

The opening size (vent size x number of vents) of drainage/ flushing regulator is the outcome of flood routing as per criteria given in different sections above. Size of opening suggested for general use are:

a. *R.C.C.Box conduit*

size:	600 mm x 900 mm	: 1 vent
	900 mm x 1200 mm	: 1 or 2 vent.
	1200 mm x 1500 mm	: 1 or 2 vent.
	1500 mm x 1800 mm	: 1 to multi vent
	1800 mm x 2400 mm	: - do- - do-
	2400 mm x 3000 mm	: - do- - do-
	3600 mm x 3000 mm	: - do- - do-

b. *Brick work and R.C.C. slab*

600 mm x 900 mm	: 1 vent

c. *R.C.C.Pipe (conduit)*

100 mm ϕ, 150 mm ϕ, 225 mm ϕ, 300 mm ϕ, 450 mm ϕ	: 1 vent
600 mm ϕ	: 1 to 2 vent
900 mm ϕ, 1050 mm ϕ, 1200 mm ϕ, 1350 mm ϕ	: 1 to 2 vent

1.6.2 Multipurpose use of Regulator

In order to ensure multipurpose use of regulator, the size of conduit as shown in 1.6.1.(a), may be suggested with the top slab at or above the crest level of the embankment, to allow fish migration or navigation to a limited extent. Weir flow is a pre-requisite for fish migration. For boat pass a submerged weir flow as well as navigation clearance is also required. In a multi-vent structure one or two vents may be selected for boat pass or fish migration.

Table 1.1

Average, monthly Evapotranspiration (ET_o mm/day)

STATION	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
1. BARISAL	2.73	3.60	4.54	6.34	6.44	3.93	3.38	3.68	3.84	3.68	3.26	2.63
2. BOGRA	2.80	3.81	5.27	6.29	5.86	4.46	4.23	4.09	4.10	3.06	3.18	2.63
3. CHITTAGONG	2.36	4.02	4.93	6.93	6.70	4.42	4.78	4.56	4.62	4.02	3.49	2.99
4. COMILLA	2.75	3.79	4.80	5.49	5.70	4.43	4.36	4.29	4.10	3.79	3.20	2.68
5. COX'S BAZAR	3.99	4.48	5.55	6.23	6.26	4.67	4.45	4.26	4.50	4.38	4.27	3.72
6. DHAKA	2.87	3.92	5.44	6.26	6.06	4.42	4.66	4.52	4.28	3.87	3.31	3.02
7. DINAJPUR	2.76	3.43	4.71	6.13	5.50	4.33	4.21	4.04	4.05	3.42	2.93	2.36
8. FARIDPUR	2.73	3.42	4.70	6.81	6.89	4.06	4.42	4.41	4.23	4.03	4.31	2.62
9. ISHURDI	2.67	3.61	6.10	6.61	6.30	4.54	4.20	4.11	4.92	3.62	3.16	2.62
10.JESSORE	2.98	3.90	6.41	7.12	6.97	4.66	4.49	4.42	4.09	4.07	3.33	2.73
11.KHULNA	2.86	3.82	4.83	5.39	5.63	3.83	3.82	3.66	3.74	3.86	3.43	2.84
12.MYMENSINGH	2.68	3.49	4.47	6.16	5.26	3.99	4.20	4.-7	3.99	3.64	3.17	2.61
13.NOAKHALI	2.86	3.79	4.77	5.59	6.54	4.16	4.[illegible]7	4.21	4.18	3.69	3.30	2.68
14.RANGAMATI	3.61	4.43	6.73	6.36	6.05	4.48	4.66	4.37	4.48	4.26	4.35	3.14
15.RANGPUR	2.32	3.32	4.36	5.65	5.43	4.44	4.32	4.16	4.10	3.66	2.98	2.36
16.SYLHET	2.67	3.67	4.71	4.86	4.72	3.56	3.83	3.70	3.40	3.64	3.10	2.67
17.SATKHIRA	2.68	3.65	4.72	6.36	5.77	3.96	3.82	3.61	3.74	2.38	2.38	3.30
18.NARAYANGANJ	2.87	3.83	5.51	5.69	6.86	4.39	6.6[illegible]	4.60	4.26	4.22	4.44	2.86

Source: Karim and Akand 1982: See also MPO Second Interim Report, Vol-VI, June, 1984.

Table 1.2

Crop Coefficients

1/2 monthly interval	1	2	3	4	5	6	7	8	9	10	11
Transplanted aman											
120 day crop	1.1	1.1	1.1	1.1	1.06	1.06	0.95	0.95	-	-	-
135 day crop	1.1	1.1	1.1	1.1	1.06	1.06	1.05	0.95	0.95	-	-
150 day crop	1.1	1.1	1.1	1.1	1.06	1.06	1.05	1.05	0.95	0.95	-
165 day crop	1.1	1.1	1.1	1.1	1.06	1.06	1.05	1.05	1.05	0.95	0.95
Boro											
135 day crop	1.1	1.1	1.1	1.1	1.25	1.25	1.25	1.0	1.0	-	-
150 day crop	1.1	1.1	1.1	1.1	1.25	1.25	1.25	1.25	1.0	1.0	-
165 day crop	1.1	1.1	1.1	1.1	1.25	1.25	1.25	1.25	1.25	1.0	1.0
Aus											
105 day crop	1.1	1.1	1.1	1.1	1.25	0.95	0.95	-	-	-	-
120 day crop	1.1	1.1	1.1	1.1	1.25	1.25	0.95	0.95	-	-	-
135 day crop	1.1	1.1	1.1	1.1	1.25	1.25	1.05	0.95	0.95	-	-
150 day crop	1.1	1.1	1.1	1.1	1.25	1.25	1.05	1.05	0.95	0.95	-
165 day crop	1.1	1.1	1.1	1.1	1.25	1.25	1.05	1.05	1.05	0.95	0.95
Wheat											
105 day crop	0.35	0.75	1.15	1.15	1.15	1.15	0.60	-	-	-	-
120 day crop	0.35	0.60	1.03	1.15	1.15	1.15	1.02	0.55	-	-	-
Potatoes	0.55	0.55	0.76	1.05	1.15	1.15	1.15	0.99	-	-	-
Vegetables (rabi)											
Cabbage/cauliflowers	0.55	0.55	0.68	0.86	1.05	1.05	1.02	-	-	-	-
Brinjals/tomatoes	0.55	0.55	0.70	0.90	1.10	1.10	1.10	1.00	-	-	-
leaf vegetables	0.55	0.55	0.86	1.00	1.00	-	-	-	-	-	-
Maize	0.55	0.62	0.92	1.15	1.15	1.15	1.02	0.74	-	-	-
Oilseeds	0.55	0.75	1.10	1.10	1.10	1.06	0.65	-	-	-	-

Source : Irrigation Management Program, Manual for Upazila Officers, January 1984.

References:

1) BWDB, Hydrologic and Hydraulic Design Procedures for Drainage Structures

2) MPO, Second Interim Report, Vol. VI-Agriculture, BWDB, 1986

3) SRP, Design Criteria, BWDB System Rehabilitation Project, October, 1993.

4) Henderson F. M., Open Channel Flow, Macmillan, London, 1966

5) EPC/SWHP/RDC, Third Flood Control and Drainage Project - Design Manual, Volume-1, 1988

2.0 GENERAL HYDRAULIC DESIGN

2.1 Flow through a Sluice or Regulator

The hydraulic design of a drainage or drainage cum flushing regulator has to be such, that no damage will occur due to extreme velocities during drainage flow as well as maximum inlet flow.

2.1.1 Flow Type and Condition

The flow through a Sluice or Regulator is dependent on the upstream and downstream water levels and is derived according to the hydraulic principles described in the **BWDB Manual of "Hydrologic and Hydraulic Design Procedures for Drainage Structures"**.

The following four types are defined (Figure 2.1)

	Flow Type	Condition
1.	Submerged Orifice	$H_1 > H_2 >$ vent height (ZD)
3.	Free Orifice	$H_1 >= 1.5xZD$ and $H_2 <=ZD$
4.	Submerged Weir	$H_1 < 1.5xZD$ and $H_2 > H_c$
5.	Free weir	$H_1 < 1.5xZD$ and $H_2 <= H_c$

Where,
H_1 = Upstream water depth above invert level
H_2 = Downstream water depth above invert level
H_c = Critical depth
ZD = Vent height

2.1.2 Flow Formula

The formula for each of the above mentioned flow types are as follows. The discharge coefficient corresponding to these flow types, except for type-3 condition, depends on the entrance coefficient K_e. The value taken for K_e=0.5 corresponding to square ended entrance. For flow type-3, the coefficient is assumed equal to 0.6 irrespective of the entrance type.

Flow formula for square ended entrance :

Flow Type	Flow Formula	Coefficient of Discharge C
1	$CA(2g\Delta H)^{1/2}$	0.802
3	$CA(2gh)^{1/2}$	0.600
4	$CBd(2g\Delta h)^{1/2}$	0.816
5	$CBH_1(2g\Delta H_1)^{1/2}$	0.305

Where ΔH is $H_1 - H_2$.

- h is measured to the orifice center line
- Δh is difference between u/s and d/s water levels within culvert.
- d is the depth of flow within culvert.
- A is the cross sectional area of barrel.
- B is the width of barrel.

For type-4 flow condition; upstream water depth H_1 is known and tail water rating curve for the outfall channel is available. Both Q & d are unknown and problem is solved by trial.

2.2 Energy Dissipation

2.2.1 General

Energy dissipation may be of any of the methods listed below:

a. Impact or impingement of water and water, water and hard surface.

b. External friction between water and air, water and channels or internal friction between water and water in turbulence.

The most frequently used energy dissipator is a hydraulic jump. Possible classes of jump forming are :

i) the jump height curve and tail water curve coincides at all flows. Jump will form at every discharge. Only a simple concrete apron with training is required to confine the jump.

ii) the jump height curve is always below the tail water rating curve. This indicates formation of submerged jumps. The solution of this problem is to build a sloping apron, so that a jump is formed somewhere on the apron.

iii) The jump height curve is always above tail water rating curve. The solution of this problem is to provide sufficient depth of water on the apron. This could be arranged by:

- depressing the downstream apron to form a stilling basin.
- installing a secondary dam near the downstream end of the apron accompanied by a sloping floor,

iv) The jump height curve is above the tail water rating curve at low flows and below at high flows. under this condition, it is suggested that a low secondary dam be built to provide proper depth to form a jump at low flow. A sloping apron may be combined with the secondary dam, to provide the desired water depth at high flow. If the flow velocity is not too high, baffle piers or some form of dentated sill near the end of the downstream apron may be used to break the high velocity.

v) The jump height curve is below the tail water rating curves at low flows and above at high flows. This case requires sufficient depth of downstream water to form a jump at high flows. This may be done by a secondary dam or an excavated pool. In order to obtain a satisfactory jump at low flows, a sloping apron may be used between the main structure and the secondary dam.

2.2.1.1 Energy Dissipation through Hydraulic jump

Energy of flow through a sluice or regulator will be dissipated through the formation of a hydraulic jump within a designed stilling basin. This will be based on generalized stilling basin design developed by USBR. Type of stilling basin will be determined as follows which is based on the range of Froude Number of incoming flow to the basin.

- For Froude number between 2.5 and 4.5:
  USBR ERC type stilling basin and Indian Standard Stilling Basin-1

- For Froude number greater than 4.5:
  USBR type - III stilling basin.

2.2.1.2 Design Discharge

The design discharge Q will be calculated from the 1 in 10 years return period pre-monsoon routing and will be checked against 1 in 25 years event.

The performance of the stilling basin will be checked for discharges of Q/10 and Q/2. If sweepout of the hydraulic jump occurs at flows less than Q, than the stilling basin design will be revised so that acceptable performance is achieved at all flows between Q/10 and Q.

2.2.1.3 Stilling Basin Design

2.2.1.4 General Considerations in stilling Basin Design

[a] *Freeboard*

The effect of the impact of the flow on the floor blocks, waves and air entrainment should be taken into consideration. A curve shown in Figure 2.2 may be used as a guide for the determination of freeboard for different values of QV_1D_1/A (notations given in the figure). It should be noted that the freeboard shown is the height above the downstream energy gradient.

[b] *Width of Stilling Basin*

In general the basin width is the same as the spillway, except for high entering velocity, the basin may be widened with a very small diverging angle.

Alternative designs for various combinations of width and length of basin should be attempted to find the most economical structure.

The Corps of Engineers practice suggests that the following items should be taken into consideration in determination of the basin width.

i. uplift on floor slab, taken as equal to the difference of depth before and after the jump, should not be excessive.

ii. the floor should be set high enough, in some cases, to avoid undercutting of foundation material of limited depth.

iii. the exit velocity of flow leaving the paved basin should be reduced sufficiently to avoid excessive channel erosion.

[c] *Length of Stilling Basin*

The stilling basin length is usually determined from the length of hydraulic jump to be confined in the basin.

2.2.1.5 Provisions at the downstream end of stilling Basins

At the downstream end of the basin, care should be taken to avoid following situations:

[a] excessive exit velocity, which may erode the downstream river bed.

[b] rolling of secondary currents which may undermine the basin floor.

[c] excessive retrogression, as a precaution to prevent undermining, which may result from retrogression of the stream bed, a cutoff wall should be installed under the downstream end of the basin.

2.2.1.6 Reduction of Tailwater depth by end sill:

The stilling basin floor is usually required to be set below the original river bed in order to provide a suitable depth for jump forming where insufficiency of tailwater depth occurs. An end sill is commonly used as the terminal wall of the stilling basin.

2.2.1.7 Stilling Basin Parameters:

The various design parameters of stilling basin will be computed as follows:

USBR ERC Type Stilling Basin: $2.5 \le F \le 4.5$ (Figure 2.3)

a. Floor Level of stilling basin : Tail water Level - $1.05xd_2$ (post jump depth).

b. Length of stilling basin: From curve in Figure 2.4 (almost $3d_2$ except for at high Froude numbers for pre-jump flow)

c. The dimension of stilling basin appurtenances shall be selected as follows (Figure 2.3)

Chute Block :	height	= d_1 (pre-jump depth)
	width(w)	= $0.7d_1$
	spacing (s)	= $0.7d_1$
Baffle Block:	As above	
End Sill :	Height (H_s)	= $0.2 x d_2$ (d_2 = Post jump depth)
	width (w)	= $0.15d_2$
	spacing (s)	= $0.15d_2$

d. The distances between chute block, baffle block and end sill (Figure 2.3) can be computed using the curve in Figure.

USBR Type-III Stilling Basin: (Figure 2.6)

a. Bed level of stilling basin : Tail water level - post jump depth.

b. Length of stilling basin : From length of jump curve of Figure 2.6(D).

c. The dimension of stilling basin appurtenances:

Chute block	:	depth (h)	=	pre jump depth, d_1
		width (w_1)	=	d_1
		spacing (s_1)	=	d_1
Baffle Block	:	depth (h_3)	=	from Figure 2.6(C)
		width (w_3)	=	0.75 h_3
		spacing (s_3)	=	0.75 h_3
End Sill	:	depth (h_4)	=	from Figure 2.6(C)

INDIAN Standard Stilling Basin-I: (Figure 2.7)

a. Bed level of stilling basin : Tail water - post jump depth.

b. Length of stilling basin (L_b) : From Figure 2.7 (B)

c. Stilling basin appurtenances:

Chute Block	:	depth (h)	=	Prejump depth, d_1
		width (w_1)	=	d_1
		spacing(s_1)	=	d_1
Baffle Block	:	depth (h_b)	=	from Figure 2.7(c)
		width (w_2)	=	0.75 h_b
		spacing (s_2)	=	0.75 h_b
End Sill	:	depth (h_s)	=	$0.2d_2$
		w = s	=	$0.15d_2$

2.2.1.8 **Governing Formulae**

The formulae for computation of critical depth, Froude number and sequent depth in relation to hydraulic jump as follows:

$$d_c = (q^2/g)^{1/3}$$

$$F_1 = \frac{V_1}{(gy_1)^{1/2}}$$

$$d_2 = \frac{d_1}{2}[(1+8F_1^2)^{1/2}-1]$$

$$\Delta H = \frac{(d_2-d_1)^3}{4d_1d_2}$$

$$e = \frac{\Delta H}{Ef_1}$$

where,

d_c	=	Critical depth (m)
q	=	Unit discharge (m3/sec/m)
g	=	acceleration due to gravity (m/sec^2)
F_1	=	Froude Number of pre-jump flow
V_1	=	Velocity at pre-jump depth (m/sec)
d_1	=	Pre-jump depth (m)
d_2	=	Post-jump depth (m)
H	=	Loss of energy head (m)
e	=	Efficiency of energy dissipation
E_{f1}	=	Energy at pre-jump position

Table 2.1 may be used to compute the different parameters of any hydraulic jump (also refer Figure 2.8)

2.2.2 Baffled Apron Drops

2.2.2.1 General

Baffled apron drops (Figure 2.9) are used in canals or wasteway channels to provide dissipation of excess energy at drops in grade. Energy dissipation occurs as the water flows over the concrete baffle blocks, which are located along the floor of the chute. They are particularly adaptable to the situation where downstream water surface elevation may vary because of degradation or an uncontrolled water surface. It is effective in dissipating excess energy for drops of any magnitude but it becomes uneconomical for large flows with great drops, due to wide section and numerous blocks required.

2.2.2.2 Capacity

The capacity of baffled apron drop is a function of the allowable discharge, q per feet of width as shown in following table. They have been operated for short periods at about twice the design capacity without excessive erosion.

Q Capacity (cfs)	q * Discharge per foot of chute width (cfs)
0 to 39	5 to 10
40 to 99	10 to 15
100 to 189	15 to 20
190 to 460	20 to 25

* Discharge per foot of width, q should be interpolated within the range indicated.

2.2.2.3 Inlet

The simplest type of inlet is used where there is no requirement of control at the upstream water surface for turnout deliveries. To minimize splashing as the flow strikes the first row of baffle block an invert curve may be provided to allow the flow to strike the blocks in a direction normal to the upstream face. The inlet length should be at least 2d. The inlet should be the same width as the baffled apron, and should provide a velocity of approach slower than the critical velocity V_c. Where splashing must be minimized, the entrance velocity should not exceed $V_c/2$, where $V_c = (qg)^{1/3}$ in the rectangular inlet section.

2.2.2.4 **Dimensions of Baffled Apron**

The following steps are suggested as a guide to be used in setting the dimensions (Figure 2.9):

(a) Set longitudinal slope of the chute floor and sidewalls at 1V : 2H.

(b) Approximate width of structure should be set by the relation,

$$B = Q/q$$

Where,

B = width,
Q = maximum total discharge, and
q = allowable discharge per foot of width.

(c) Set the first row of baffles so that the base of the upstream face is at the downstream end of the invert curve and no more than 12 inches in elevation below the crest.

(d) Baffle block height, H should be about 0.9 times critical depth, d_c, to nearest inch.

(e) Baffle block widths and spaces should be equal, and not less than H but not more than 1.5H. Partial blocks, having a width not less than 1/3 H and not more than 2/3 H should be placed against the sidewalls in rows 1,3,5,7 etc.

Alternate rows of baffle blocks should be staggered so that each block is downstream from a space in the adjacent row. The structure width, B, determined above, should be adjusted so convenient baffle block widths can be used.

(f) The slope distance, s, between rows of baffle blocks, as shown in Figure 2.9, should be at least 2H but no greater than 6 feet. For blocks less than 3 feet in height, the spacing may be increased from 2H but should not exceed 6 feet.

(g) A minimum of four rows of baffle blocks should be used. The baffled apron should be extended so that the top of at least one row of baffle blocks will be below the bottom grade of the outlet channel. The apron should be extended beyond the last row of blocks a distance equal to the clear space between block rows.

(h) Baffle blocks are constructed with their upstream faces normal to the chute floor. The longitudinal thickness, T, of the baffle blocks at the top should be at least 8 inches, but not more than 10 inches. See block detail, Figure 2.9.

(i) Suggested height of the walls to provide adequate freeboard is three times the baffle block height measured normal to the chute floor. It is generally not feasible to set the freeboard for these structures to contain all of the spray and splash.

2.2.3 Baffled Outlets

2.2.3.1 General

The baffled outlet (Figure 2.10) is a box like structure having a vertical hanging baffle and an end sill. Excess energy of the incoming waterjet is dissipated primarily by striking the baffle and to a lesser degree by eddies that are formed after the jet strikes the baffle. A tail water depth is not required for satisfactory hydraulic performance as is the case for a hydraulic jump basin. The baffled outlet, if properly designed, is a more effective energy dissipator than the hydraulic jump.

2.2.3.2 Hydraulic Considerations

The baffled outlet was developed using hydraulic model studies from which detailed dimensions for the baffled outlet were determined for various Froude numbers. To standardize the method of computing Froude numbers, the shape of the jet is assumed to be square, thus, the depth of the incoming flow, d, is considered to be the square root of its cross-sectional area using the equation $A = Q/V$. In this equation V is the theoretical velocity and is equal to $\sqrt{2gh}$.

The curve on Figure 2.10 indicates the minimum width of basin that should be used for a given Froude number. However if the basin is too wide the energy will not be effectively dissipated because the incoming jet will spread and pass under the baffle rather than strike the baffle. Also the depth of the baffle should not be less than the diameter of the incoming pipe to prevent the jet from passing over the baffle. For partial discharge as well as design discharge best overall dissipation is achieved only if the basin width is equal to or slightly greater than the width determined from the curve for design discharge. Other basin dimensions are ratios of the width as shown on Figure 2.10. To prevent cavitation or impact damage to the basin, the theoretical pipe velocity ($\sqrt{2gh}$) should be limited to 50 feet per second. The diameter of the pipe outletting into the baffled outlet structure, should be determined using a velocity of 12 feet per second assuming the pipe is flowing full.

If the entrance pipe slopes downward, the outlet end of the pipe should be turned horizontal for a length of at least 3 pipe diameters so as to direct the jet into the baffle.

If there is a possibility of both the upstream and downstream ends of the pipe being sealed, an air vent near the upstream end may be necessary to prevent pressure fluctuations and associated surging of flow in the system.

2.2.3.3 Basin width Design Procedures

For a given design Q and head, h, the width of the baffled outlet basin is determined as follows:

(a) Compute theoretical velocity in feet per second, $V = \sqrt{2gh}$

(b) Then compute the cross-sectional area o the incoming flow in square feet, $A = Q/V$.

(c) Next compute the depth of flow, d in feet, $d = \sqrt{A}$. (This assumes the shape of the jet is square).

(d) Compute the Froude number, $F = V/\sqrt{gd}$.

(e) For this "F" read W/d ratio from the curve on Figure 2.10:

(f) Then W in feet = d(W/d). This is the minimum width which should be used.

2.2.4 Impact Block Type Basin

2.2.4.1 General

Impact block basin gives reasonably good dissipation of energy for a wide range of tailwater depths. The dissipation of high energy is principally by turbulence induced by the impingement of the incoming flow upon the impact blocks. The required tailwater depths is more or less independent of the drop height.

2.2.4.2 Design Procedure

Fig. 2.16 gives a layout for baffle block solution. The drop number is $D = q^2/gy^3$, against the drop number and the assumed dimension of the basin, the details of basin can be worked out after several trials.

2.2.4.3 Limiting Dimension of Basin

Minimum Basin length, $L_B = L_p + 2.55d_c$.

Minimum length of upstream face of baffle block = $L_p + 0.8d_c$.

Minimum tailwater depth, $d_{TW} = 2.15d_c$.

Optimum baffle block height = $0.8d_c$.

Width and Spacing of Baffle Block = $0.4d_c \pm$

Optimum height of end sill = $0.4d_c$.

2.3 Safety Against Seepage or Piping

2.3.1 Method of Analysis

Seepage force is the force which acts on soil particles generated by the flow of water through the sub-soil of a structure. If the seepage force is too large at the exit of the flow line, it may remove soil particles in that region. This progressive action results in a formation of cavities which, if large enough, may cause foundation failure. The Khosla's theory of Exit Gradient is used fro determination of measures to prevent piping underneath the structures. Lane's theory of weighted Creep theory is used to check side seepage.

2.3.2 Factor of Safety

Safety against piping can be ensured by providing sufficient floor length and reasonably deep vertical cutoff walls at the ends of the floor of the structure. These can be obtained by keeping the exit gradient (as per Khosla's theory) or weighted creep ratio (as per Lane's theory) well below the critical values.

Values of safe exit gradient for some of the sub-soil materials are given below :

Type of Soil	Khosla's safe exit gradient
Fine Sand	0.17 to 0.14 (1/6 to 1/7)
Coarse Sand	0.20 to 0.17 (1/5 to 1/6)
Shingle	0.25 to 0.20 (1/4 to 1/5)

Values of weighted creep ratio for different foundation materials are as follows :

Type of Soil	Lane's Weighted Creep Ratio
Very fine sand and silt	8.5
Fine sand	7.0
Medium sand	6.0
Coarse sand	5.0
Soft clay	3.0
Medium clay	2.0
Hard clay	1.8
Very hard clay	1.6

2.3.3 Governing Formula

The governing formula for computing exit gradient and weighted creep ratio are as follows:

Khosla's Theory

$$\alpha = b/d$$

$$\lambda = \frac{1+(1+\alpha^2)^{1/2}}{2}$$

$$G_E = \left(\frac{H}{d}\right) X \frac{1}{\pi(\lambda)^{1/2}}$$

Where:

b = Total length of floor
d = Depth of downstream cutoff wall
H = Maximum differential head (see Figure 2.10)
G_E = Exit gradient

Lane's Theory

The weighted creep distance or percolation path L is defined as follows:

For flow passing under a construction :

$$L_u = \sum V_v + \frac{\sum h}{3}$$

For flow passing along the sides :

$$L_s = 0.75 X \sum V_h + \sum h_n$$

Where,

V_v = vertical path along vertical surface
h = horizontal path along horizontal surface
V_h = horizontal path along vertical surface
h_n = horizontal path along vertical surface normal to axis of structure

Note: Surfaces at 45° or more to the horizontal are considered as vertical surface and surfaces at less than 45° are considered to be horizontal when computing creep length.

The weighted creep ratio (C_r) is defined as follows :

$$C_r = \frac{L_u or L_s}{H} > C$$

Where,

H is the maximum differential head.
C is the safe value of a Lane's Creep Ratio.

Khosla's theory shall be followed to determine the measures to prevent piping underneath the structures. Lane's weighted creep theory shall be used to check the side seepage (refer Figure 2.13).

2.4 Safety Against Uplift

2.4.1 Factor of Safety

Excessive hydrostatic head difference across a regulator causes seepage of water through the underlying sub-soil. The seepage water causes uplift pressures underneath the structure. The net residual uplift pressure may uplift part or whole of a structure if sufficient counterbalance weight is not provided. A factor of safety of 1.10 against uplift will be provided (Ref: Page 433 of Irrigation Engineering by S.K. Garg).

2.4.2 Design Conditions

The uplift pressure under a structure needs to be checked for two conditions.

- Uplift pressure under steady seepage.
- Uplift pressure in the jump trough under flood flow conditions.

2.4.3 Method of Analysis

The uplift pressure may be determined by any one of the following methods.

- By Flow Net Theory
- By Khosla's Theory
- By Lane's Theory

The flow net theory requires graphical sketching and a trial and error approach. Hence, it is complicated and time consuming. Khosla has given a simple, quick and accurate approach, called the Method of Independent Variables to compute uplift pressures under a hydraulic structure. Lane's weighted creep theory which is less accurate than Khosla's theory may be applied to compute uplift pressures under minor structures.

2.4.4 Governing Formulae

The uplift pressure under a hydraulic structure according to Khosla's theory of Method of Independent Variable can be computed as follows:

a. The percentage pressure at key points can be computed using curve as in Fig. 2.13 of using the following formulae.

$$\Phi_E = \frac{1}{180}\cos^{-1}\left(\frac{\lambda-2}{\lambda}\right)$$

$$\Phi_D = \frac{1}{180}\cos^{-1}\left(\frac{\lambda-1}{\lambda}\right)$$

$$\Phi_{C1} = 100 - \Phi_E$$

$$\Phi_{D1} = 100 - \Phi_D \text{ (upstream sheet pile)}$$

$$\Phi_{D1} = 100 - \Phi_{D'} \text{ (depressed floor)}$$

$$\Phi_{D'} = \Phi_D - 2/3(\Phi_E - \Phi_D) + 3/\alpha^2 \text{ (depressed floor)}$$

b. The above percentage pressures are valid for a simple profile of the structure i.e. for a straight horizontal floor of negligible thickness with a cutoff on the upstream end or downstream end. For complex profile of the structure, the following corrections to percentage pressures are required.

(i) Correction for mutual interference of pile

$$C = 19[D/b']^{1/2}[(d+D)/b]$$

where,

C = Correction in percent and is positive for points in the rear or backwater direction and negative for points forward in the direction of flow.
b' = distance between two cutoff walls in meter.
D = depth of pile line in meter, the influence of which is to be determined on the neighboring pile of depth, d. D is to be measured below the level at which interference is desired.

d = depth of pile in meter on which effect is considered.
b = total length of the floor in meter.

ii) Correction for floor thickness:

The pressures are corrected by assuming linear pressure distribution. The correction to be applied for the point E1 shall be negative while at C1 is positive. (reference Figure 2.14)

iii) Correction for slope of floor:

A correction is applied for a sloping floor and is taken as +Ve for the down slopes and -Ve for upslopes following the direction of flow. Values of correction for standard slopes are given below:

Slope (V : H)	Correction factor
1:1	11.2
1:2	6.5
1:3	4.5
1:4	3.3
1:5	2.8

2.5 Protection Works

2.5.1 Type of Protection

Cutoff walls at either end of a rigid apron are provided to protect the structure against failure by scour in the channel. In addition, protective works are required at the upstream as well as at the downstream ends of rigid aprons in order to obviate the possibility of a scour hole travelling close to the apron and to relieve any residual uplift pressure through the filter. The respective protection arrangements consist of (a) launching apron and (b) inverted filter. (Where inverted filter implies coarser grains above finer grains). A sketch showing the different protective works is shown in Figure 2.15.

2.5.2 Scour Depth

The regime scour depth R(m) (refer Figure 2.15) in relation to unit width discharge and silt factor is given by -

$$R = 1.35\ (q^2/f)^{1/3}$$

where,

q = discharge per unit width (m^3/sec/m)
f = Lacey's silt factor.

2.5.3 Silt Factor

Lacey's silt factor for various types of soil material may be computed from:

$$f = 1.76\ (d_m)^{1/2}$$

where d_m = weighted mean particle size of the bed material (mm) or taken from Table below.

Lacey's silt factor for different materials

Type of Material	Mean grain size (mm)	Silt Factor (f)
Silt:		
	0.052	
Very fine	0.081	0.40
Fine	0.158	0.50
Medium	0.323	0.70
Coarse		1.00
Sand:		
Medium	0.505	1.25
Coarse	0.725	1.50

2.5.4 Cutoff Walls

The following minimum depth of cutoff walls (D) shall be provided at upstream and downstream ends of structures in consideration of scour protection.

Upstream cutoff wall depth, $D_{u/s}$ = 1.25 R - water depth

Downstream cutoff wall depth, $D_{d/s}$ = 1.50 R - water depth

where, R is the regime scour depth.

The minimum cutoff depths below the underside of apron or floor slab, should, however, be as follows for different water depths:

Water depth (m)	Cutoff depth (m) below bed
Upto 0.90 m	0.60 m
0.90 m to 1.80 m	0.75 m
Over 1.80 m	0.90 m

2.5.5 Loose Protection

Adjacent to the rigid apron, protection is provided by a concrete or brick block apron or stone riprap upon an inverted filter. The various design requirements for the apron are as follows:

2.5.5.1 Length of Inverted Filter

The length of Apron shall be as follows:

- o Upstream : 1.25 $D_{u/s}$
- o Downstream : 1.50 $D_{d/s}$

Where D is the scour depth below channel bed calculated with maximum depth of scour R as below:

- o R upstream = 1.25 R
- o R downstream = 1.50 R

2.5.5.2 Block/Stone Size

The size of block/stone may be determined by formula

U_a = $KD^{1/2}$, where
U_a = average velocity of flow (m/sec)
K = coefficient depending upon type of block/stone as below
D = unit dimension of cubic block (m) or mean diameter of stone (m).

Stone/Type of block	Specific gravity	Value of K
Stone	2.65	4.92
Brick	1.92	5.18
Concrete with brick aggregate	2.08	5.25
Concrete with gravel aggregate	2.24	5.32

2.5.5.3 Filter Design Criteria

The inverted filter shall be designed using the following criteria, (Ref: DRAINAGE PRINCIPLES AND APPLICATIONS - by International Institute of Land Reclamation and Improvement Page - 174 and 175).

i) The gradation of filter should conform to the following rule.

Filter type	p	q
For homogeneous and round grains	5-10	5-10
For homogeneous sharp grains (Sylhet sand)	10-30	6-20
For graded grains (sized brick chips, stone chips)	12-60	12-40

ii) The sieve curves of all layers should be almost parallel in the area of the smaller fractions.

iii) Minimum layer thickness

Sand - 0.10 m
Gravel - 0.15 m
Stone - 2 times stone diameter

2.5.6 Launching Apron

The launching apron is to be provided beyond the flexible apron. The various design parameters are to be as follows :

(a) The length of laid apron shall be :

upstream : 1.50 $D_{u/s}$
Downstream : 2.00 $D_{d/s}$

(b) Length of launched apron = 2.25D (where launched apron means after scour hole has developed)

2.6 Diversion of Channel

The diversion channel connecting the structure with the parent drainage channel/outfall river shall be designed using the following data. Smooth transition shall be made at both ends.

a.	Design discharge flow.	: 75% of 1 in 25 year pre-monsoon routed
b.	Maximum velocity	: 0.90 m/sec
c.	Minimum velocity	: 0.30 m/sec
d.	Side slope	: 1:1.5 to 1:2 depending on the soil types asgiven below.

Material	Side Slope
Clay to gravelly loam	1:1.5
Loose sand loam	1:2
Very sandy soil	1:2

References:

1) Chow V. T., Open-Channel Hydraulics

2) Richard H. F., Open-Channel Hydraulics

3) BWDB & CIDA, Design of Small Scale Water Control Structures

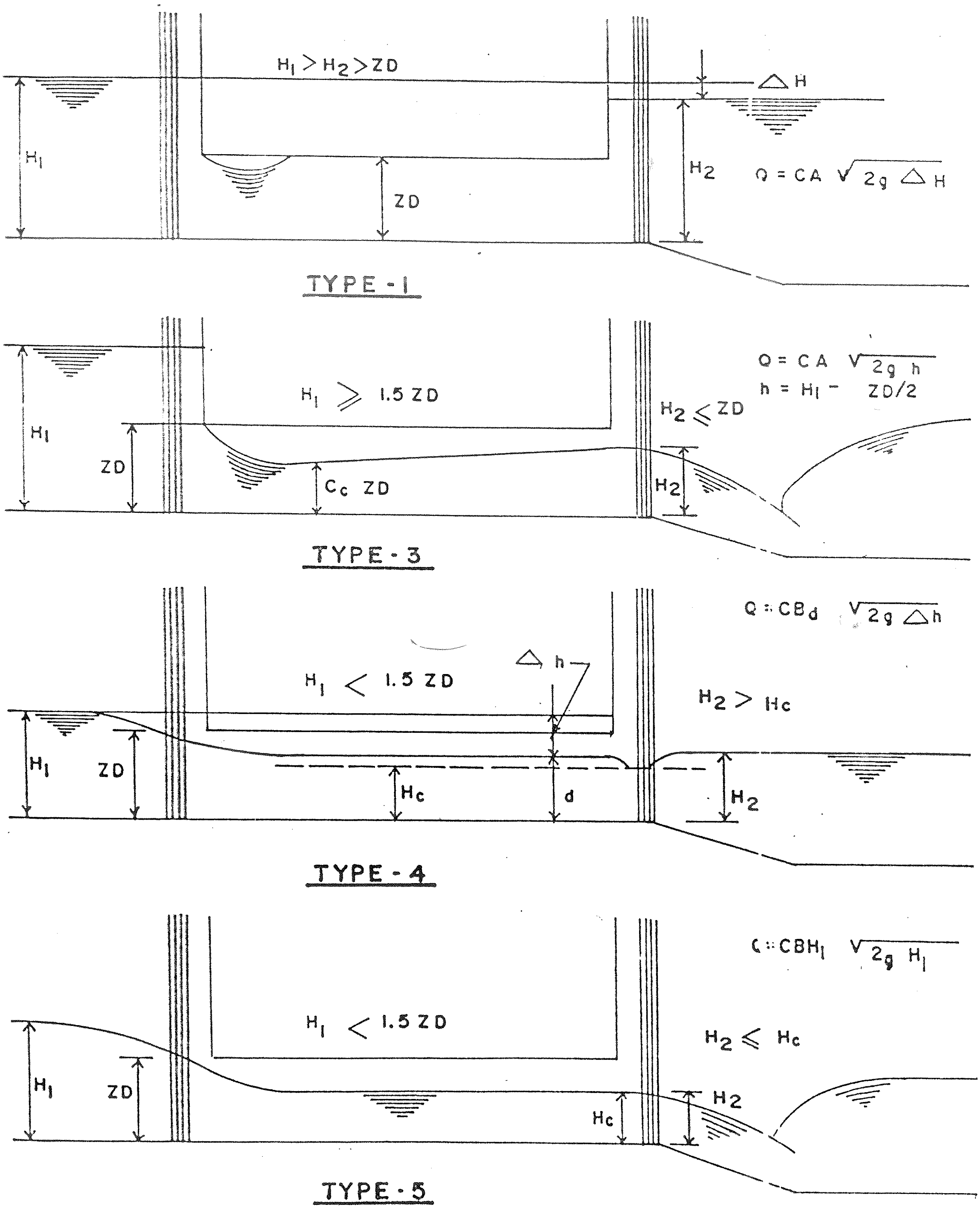
4) Varsaney, Theory and Design of Irrigation Structures

5) United States Department of the Interior, Design of Small Dams

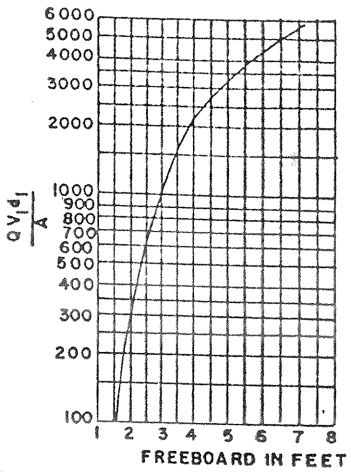
6) SRP, Design Criteria

7) EPC/SWHP/RDC, Third Flood Control and Drainage Project, Design Manual, Volume - 1, 1988

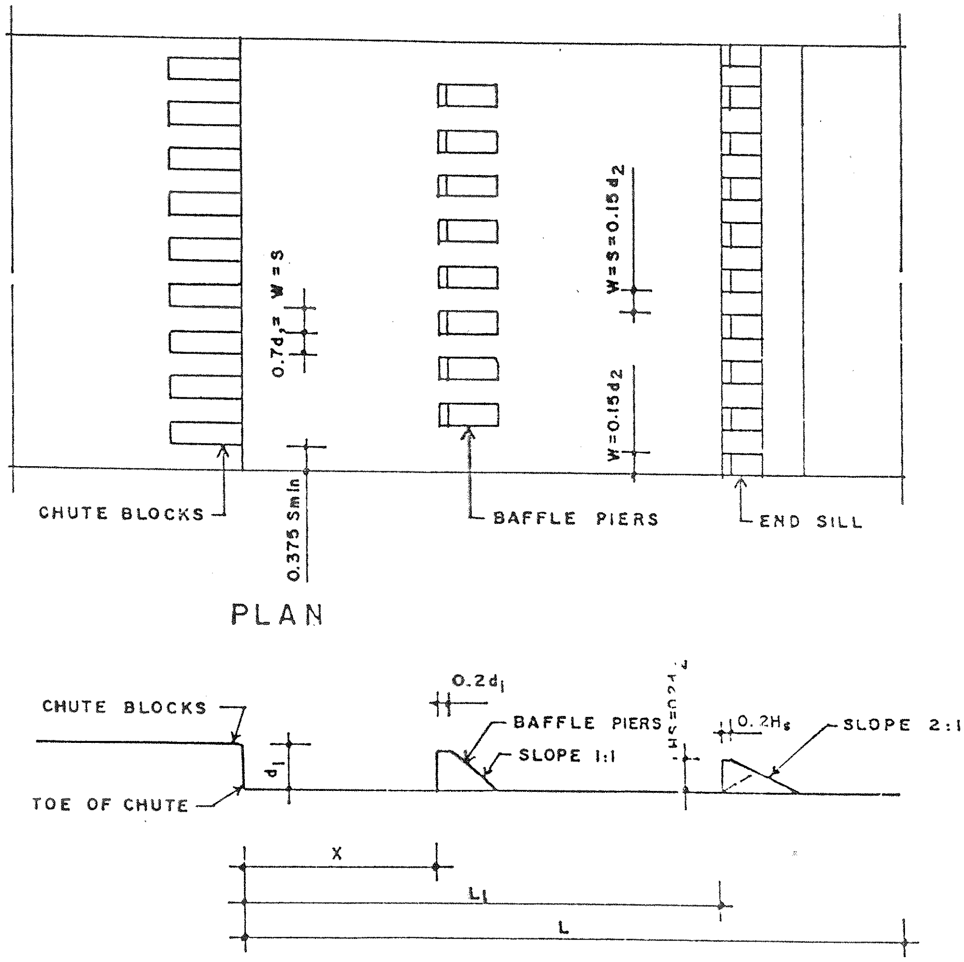
Figure: 2.1



CONDITIONS OF FLOW THROUGH SLUICES

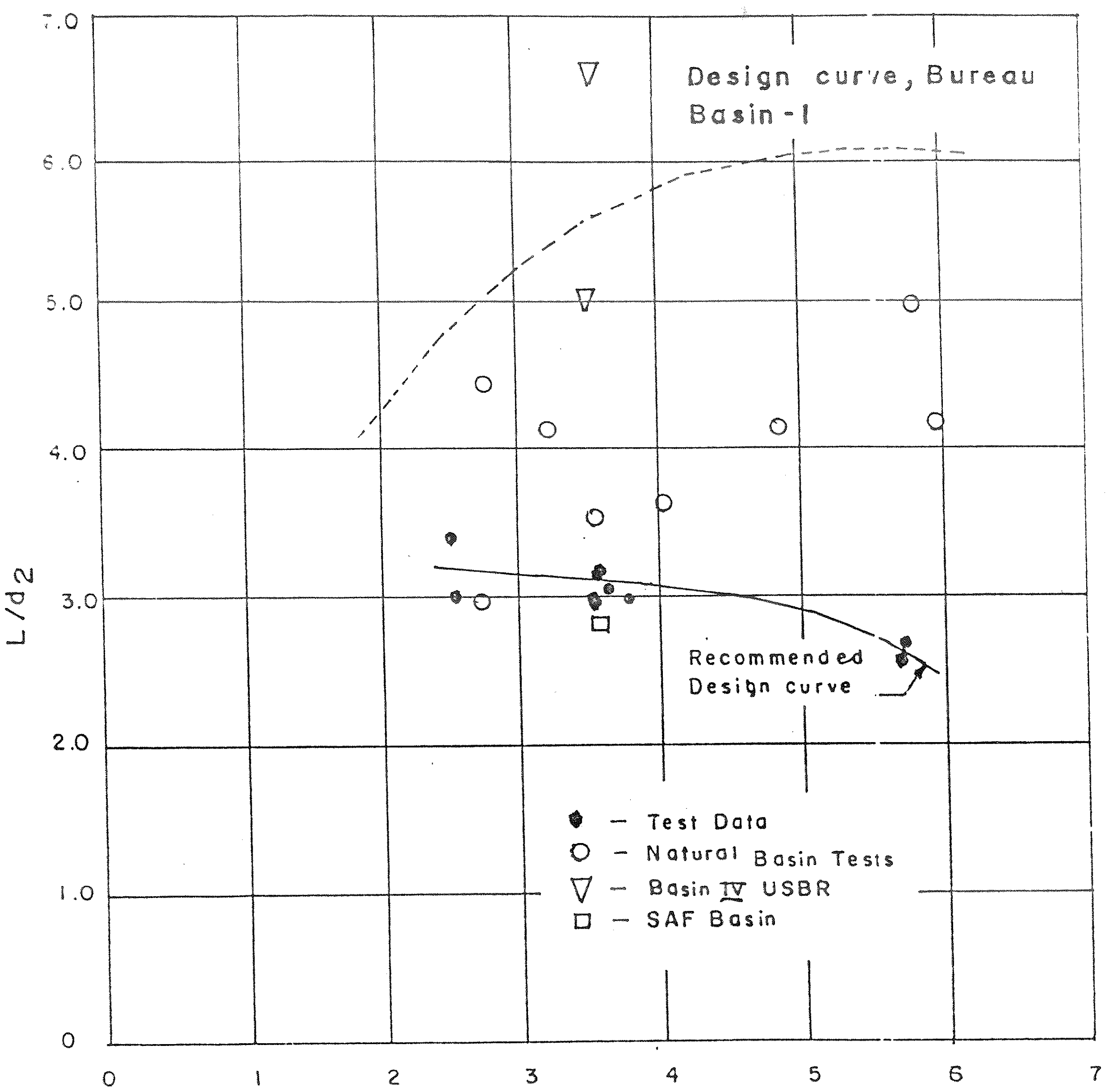


CURVE FOR FREEBOARD IN STILLING POOL
Fig. 2.2



ELEVATION
USBR ERC TYPE STILLING BASIN
(2.5 ∠ F ∠ 4.5)
Fig. 2.3

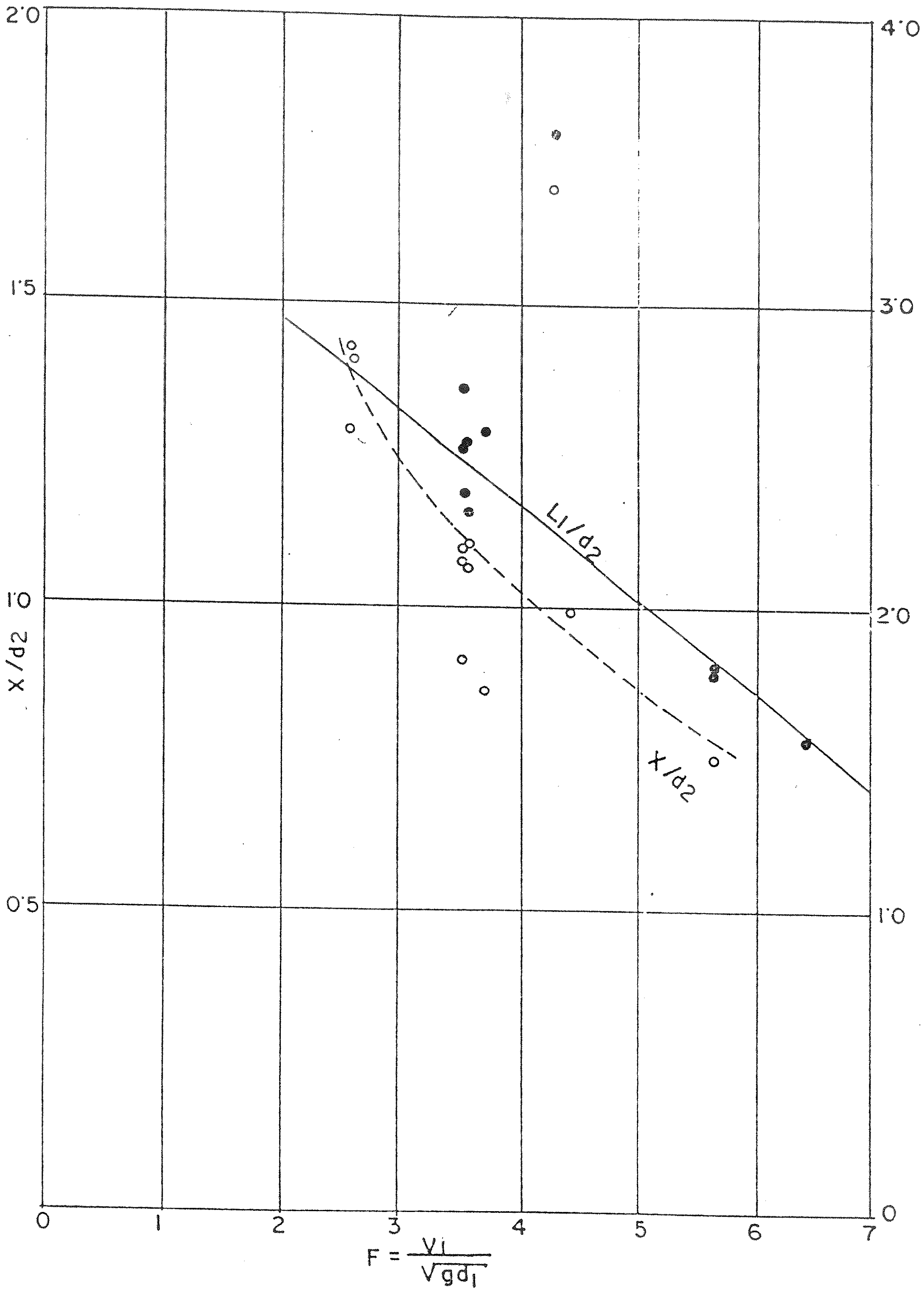
Figure 2.4



$$F = \frac{V_1}{\sqrt{g d_1}}$$

DIMENSIONLESS LENGTH OF STILLING BASIN

Figure 2.5



Distance from toe of chute to baffle piers (X) and end sill (L)

USBR Type III Stilling Basin

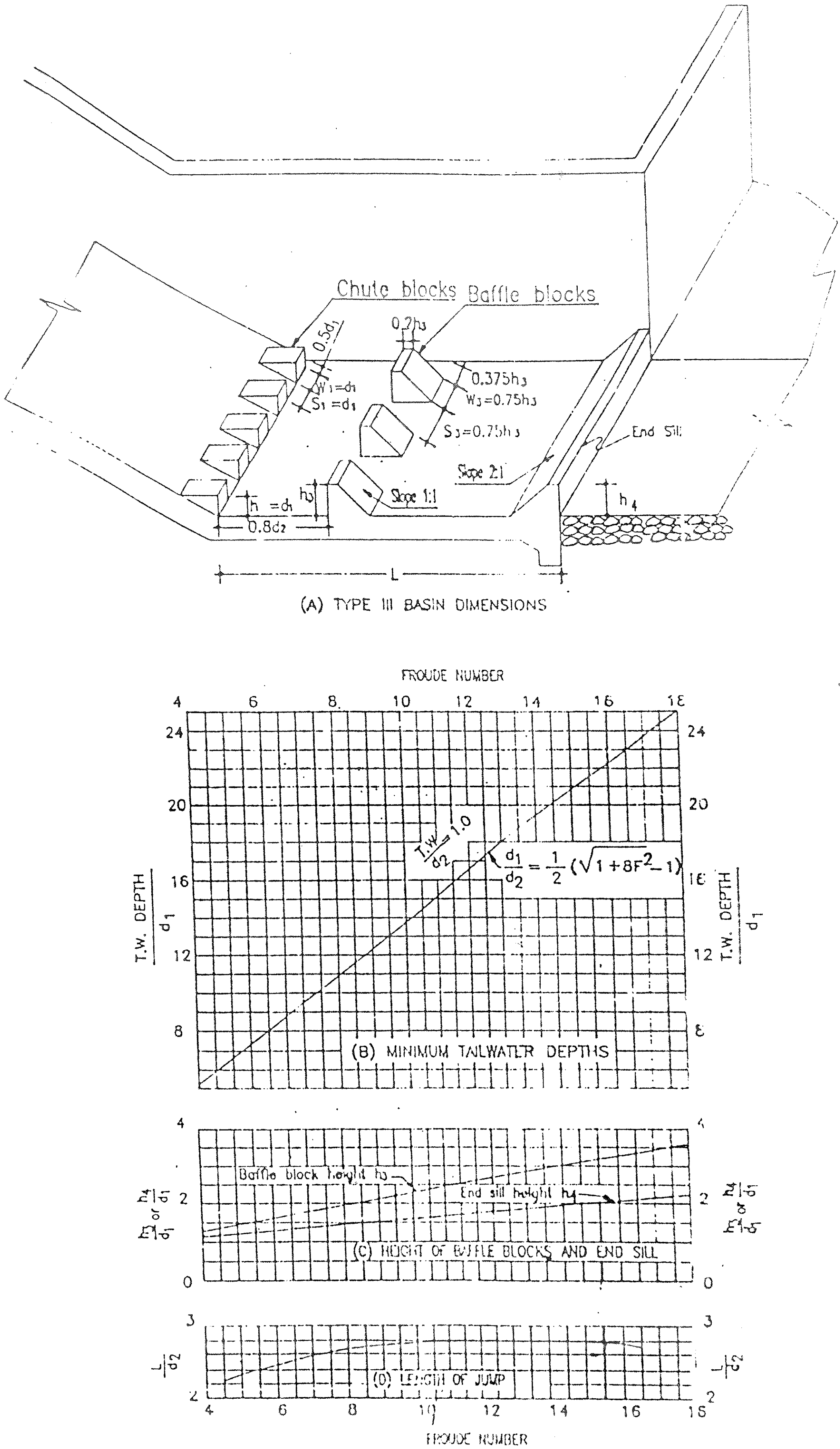
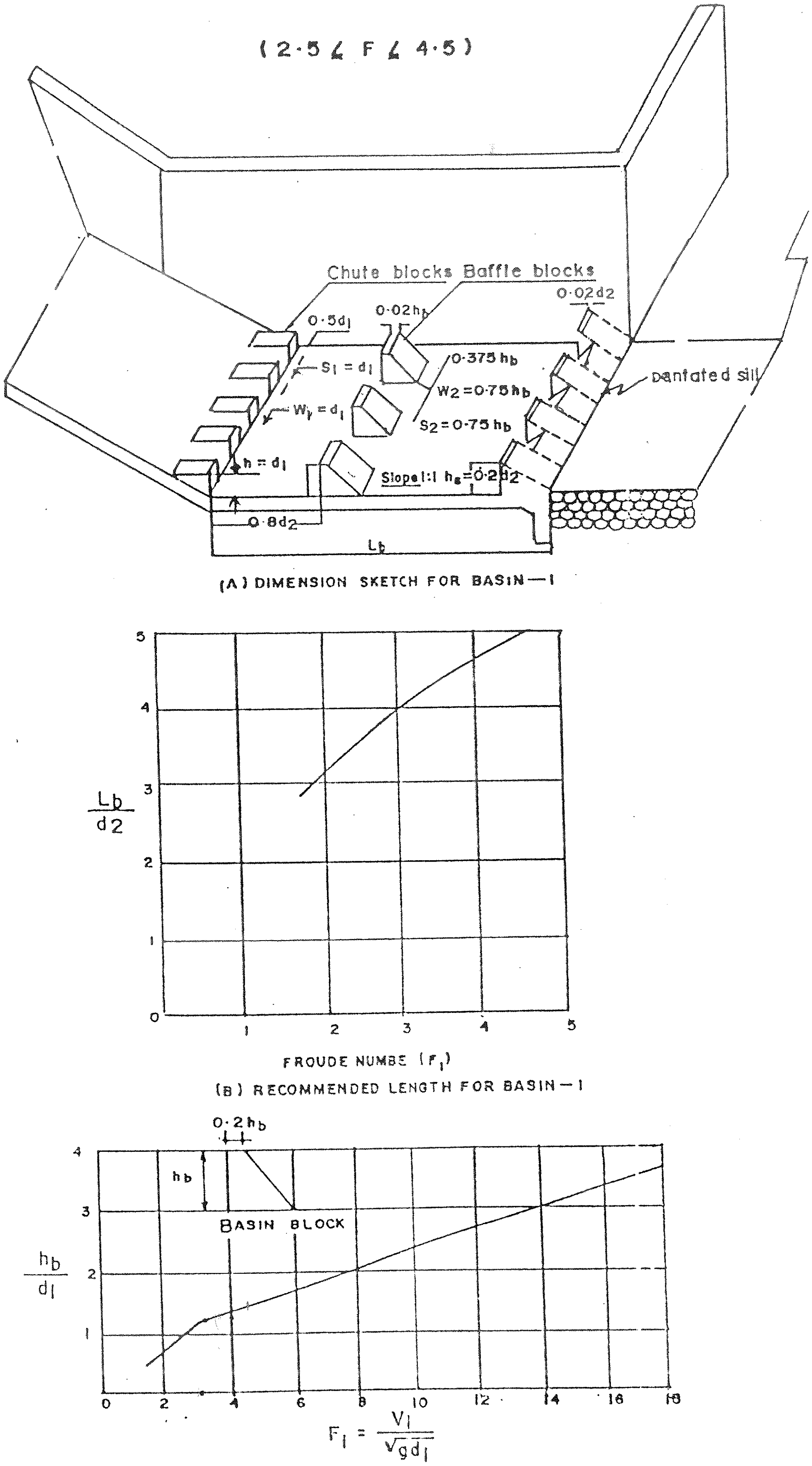


Figure 2.7

INDIAN Standard Stilling Basin — I

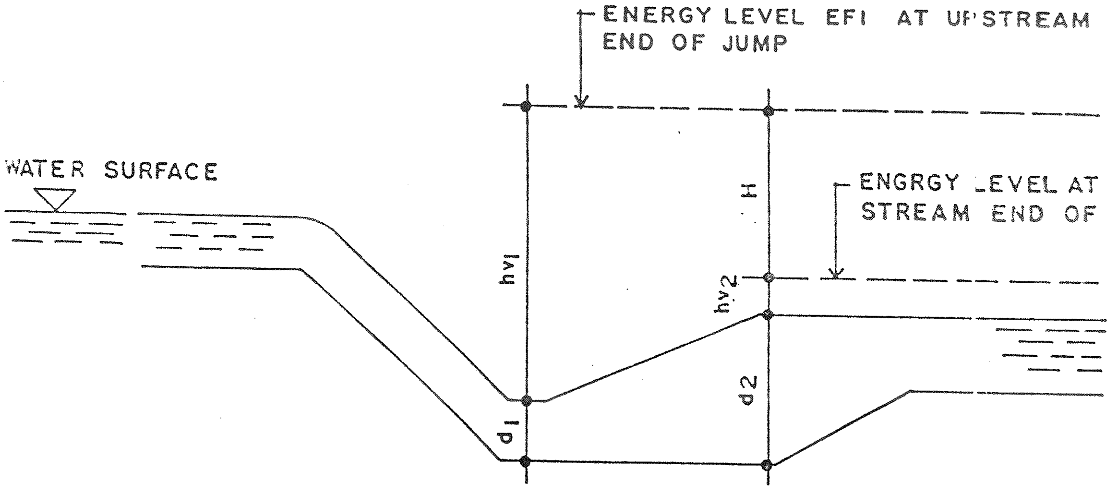
(2·5 ≤ F ≤ 4·5)



$$F_1 = \frac{V_1}{\sqrt{g d_1}}$$

(c) APPURTENANCES FOR BASIN—I

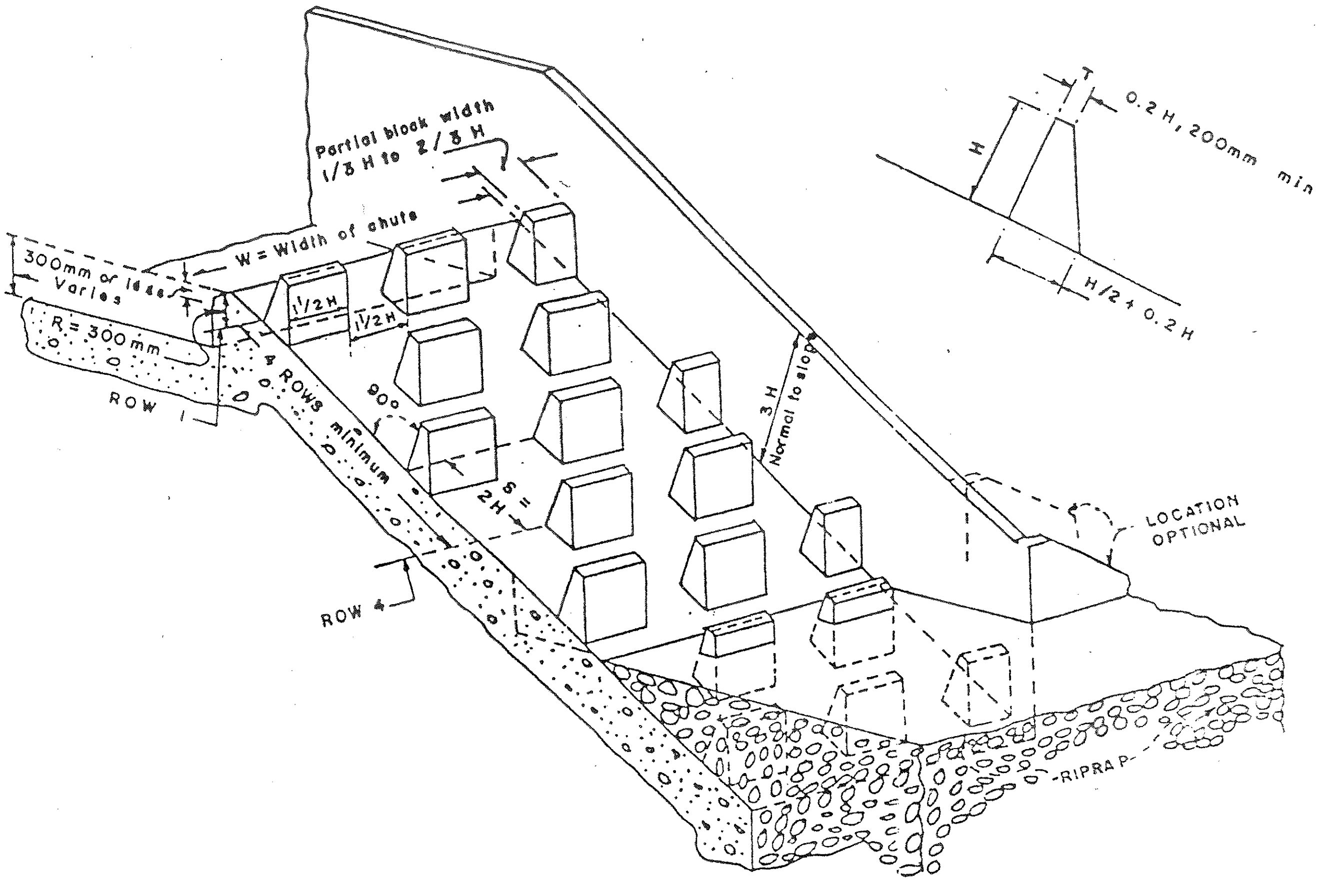
Figure: 2.8



RELATION OF ENERGY LOSS, CRITICAL DEPTH, DEPTH BEFORE AND AFTER JUMP FOR HYDRAULIC JUMPS IN RECTANGULAR CHANNELS WITH LEVEL FLOOR.

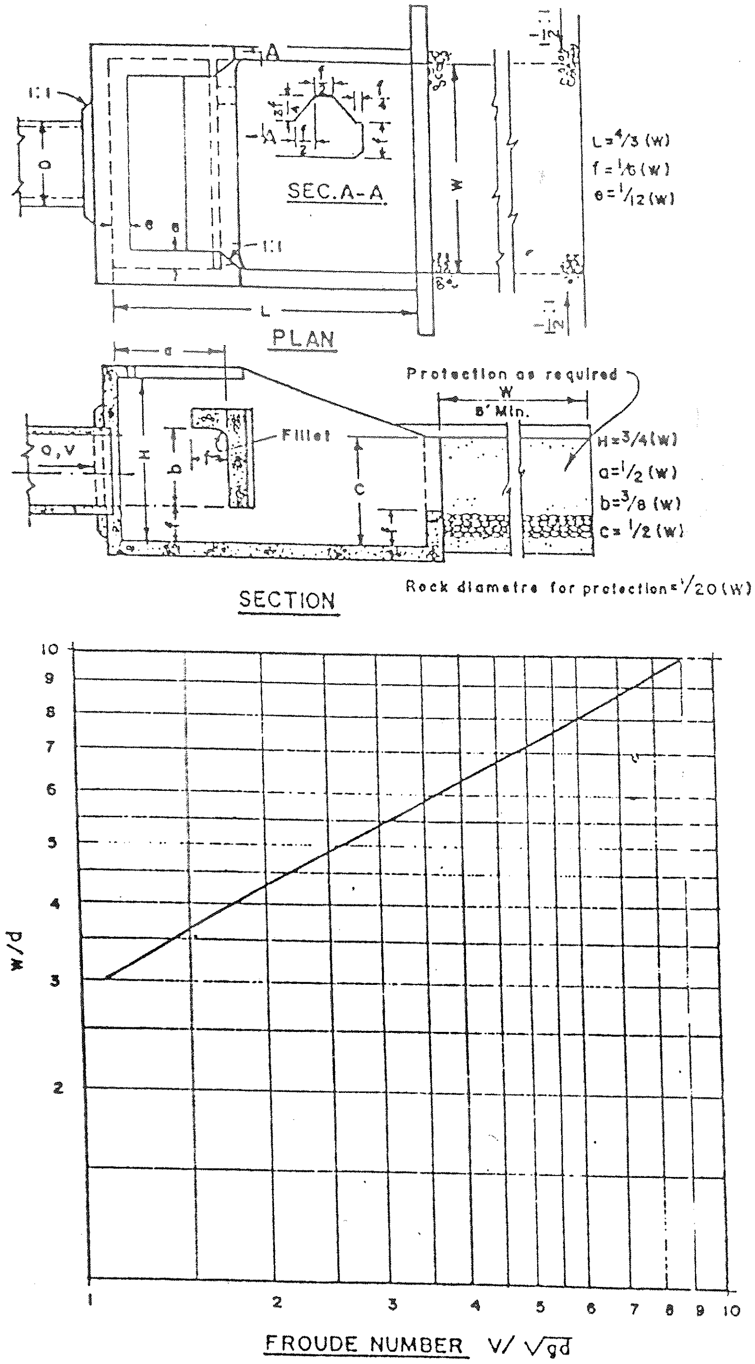
ENERGY LOSS IN HYDRAULIC JUMP

Figure 2.9



BASIC PROPORTIONS OF A BAFFLED APRON DROP

Figure 2.10



W, ft, is the inside width of the basin
V, fps, is the theoretical velocity of the incoming flow and is $\sqrt{2gh}$
h, ft, is the head to be dissipated
A, ft^2 is the area of flow entering the basin and is Q/V
d, ft, Represents the depth of flow entering the basin and is $\sqrt{A}$

Design width of basin

BAFFLED OUTLETS

Figure: 2.11

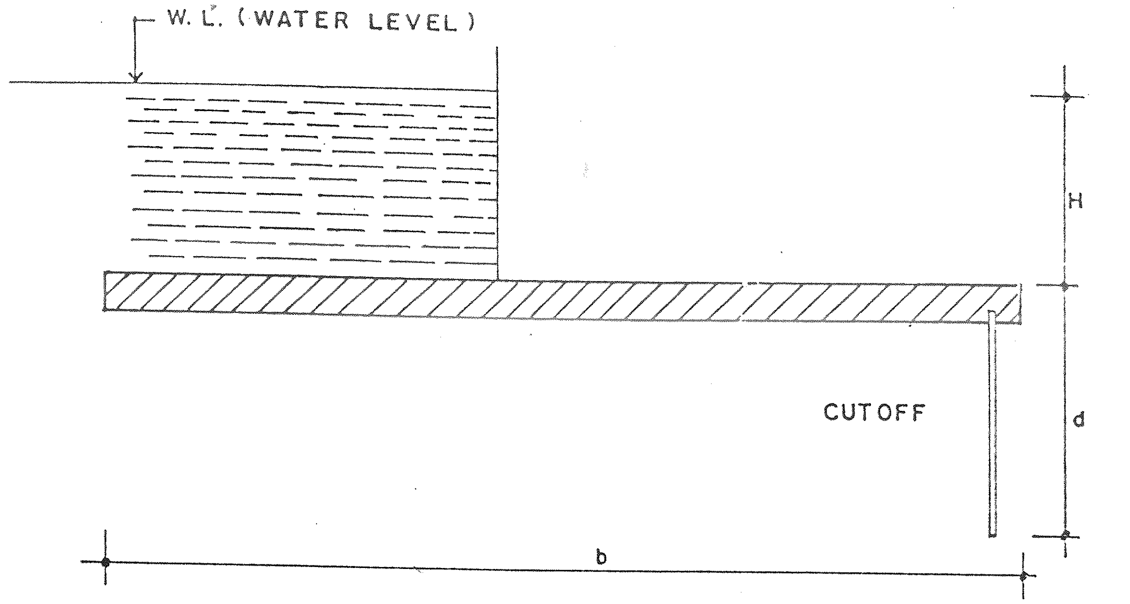
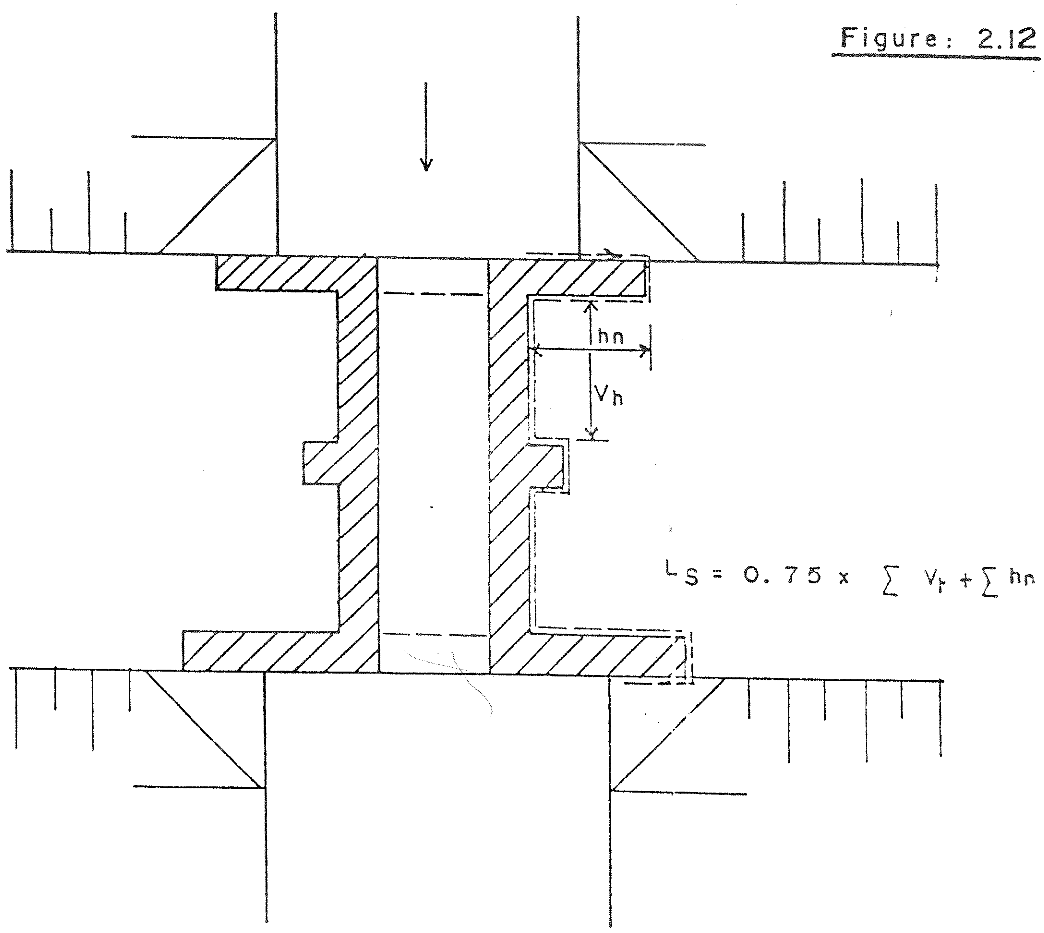
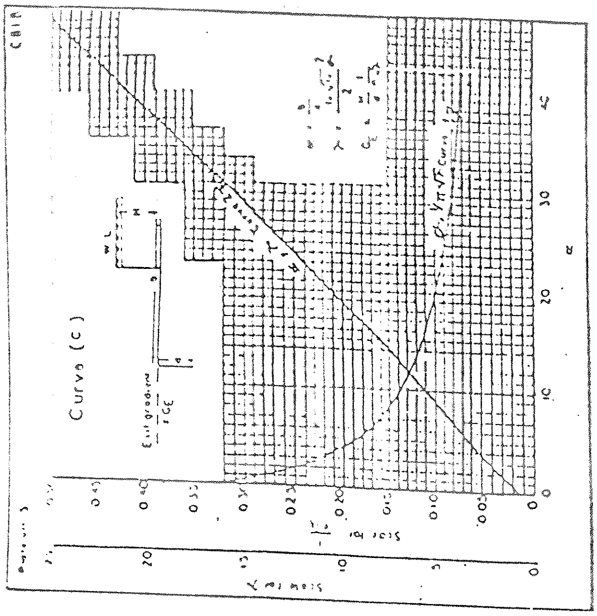


Figure: 2.12



LANE'S CREEP THEORY FOR SIDE SEEPAGE

Figure 2.13



·KHOSLA'S
CURVES

For explanation of these curves refer
"Design of weirs on Permeable Foundation"
by Khosla Bose

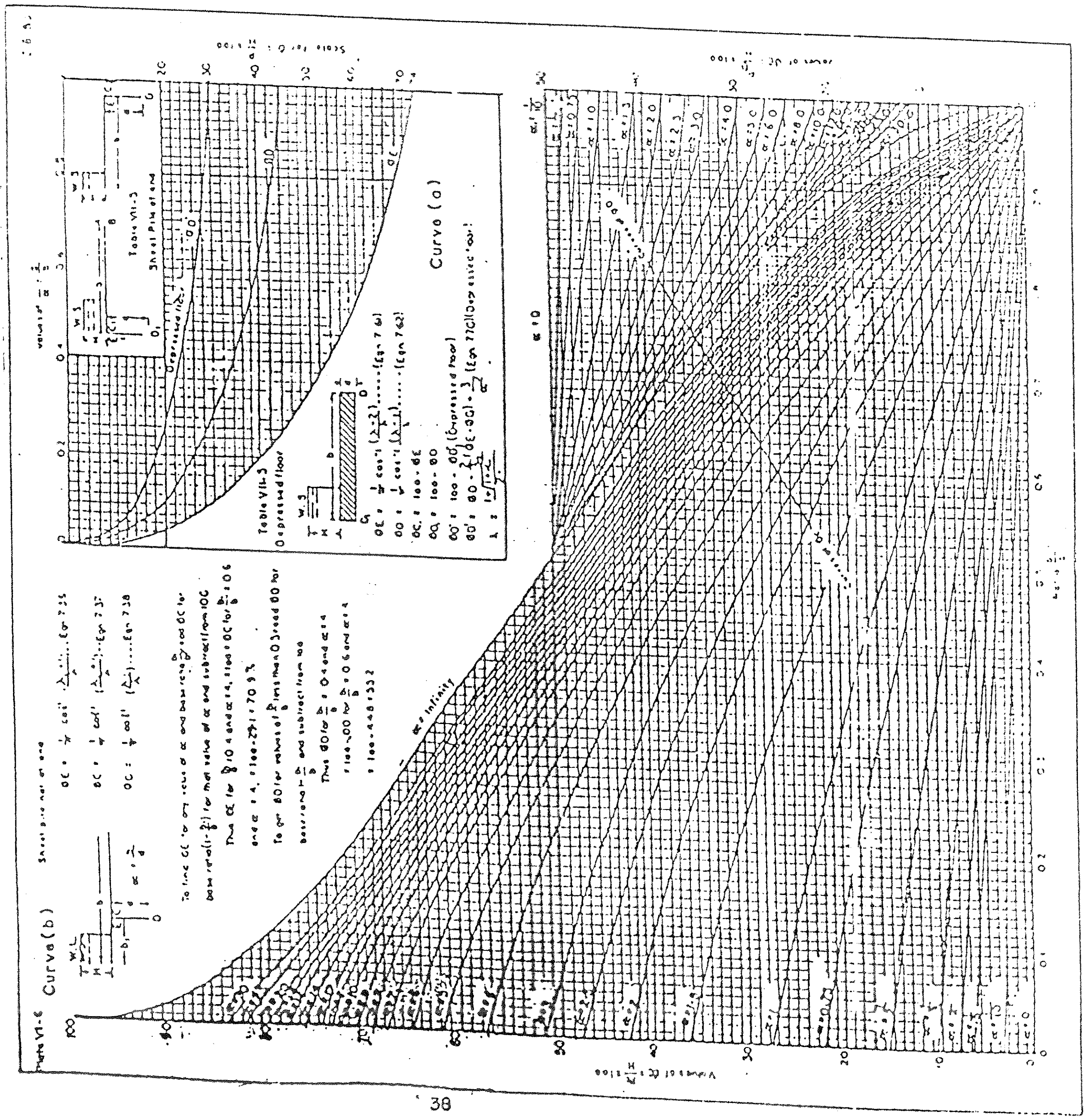
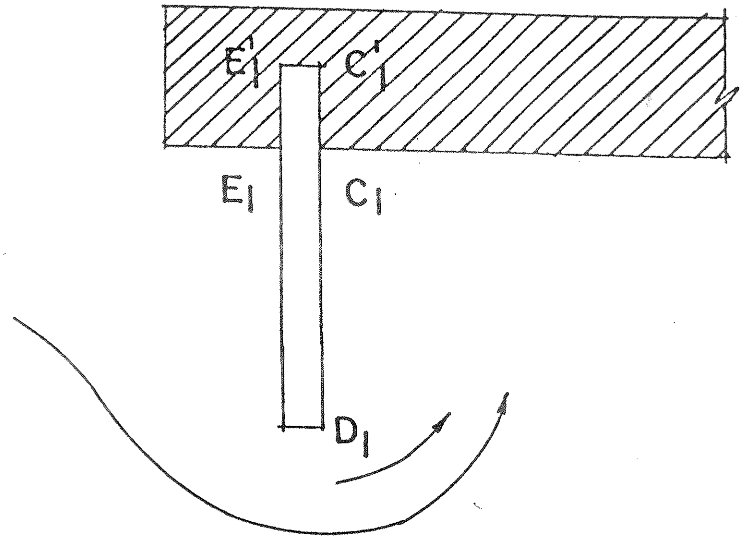


Figure 2.14



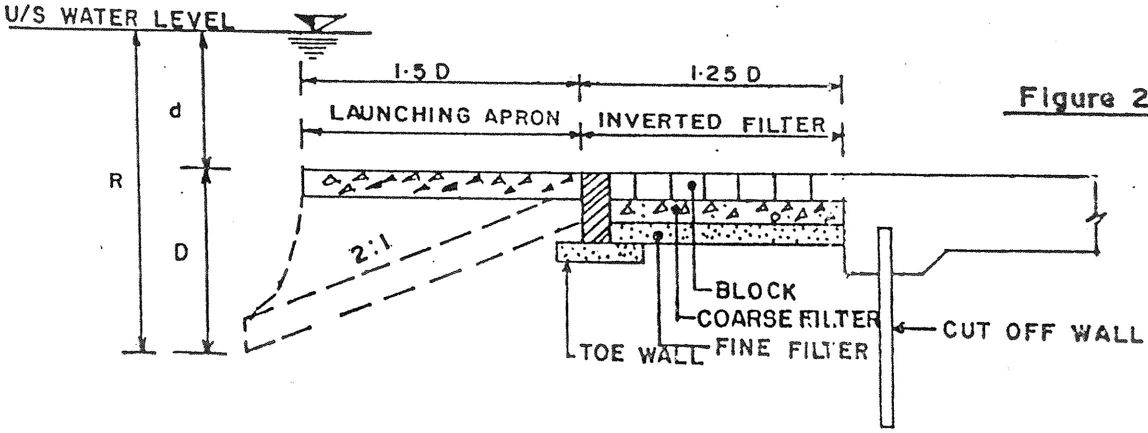
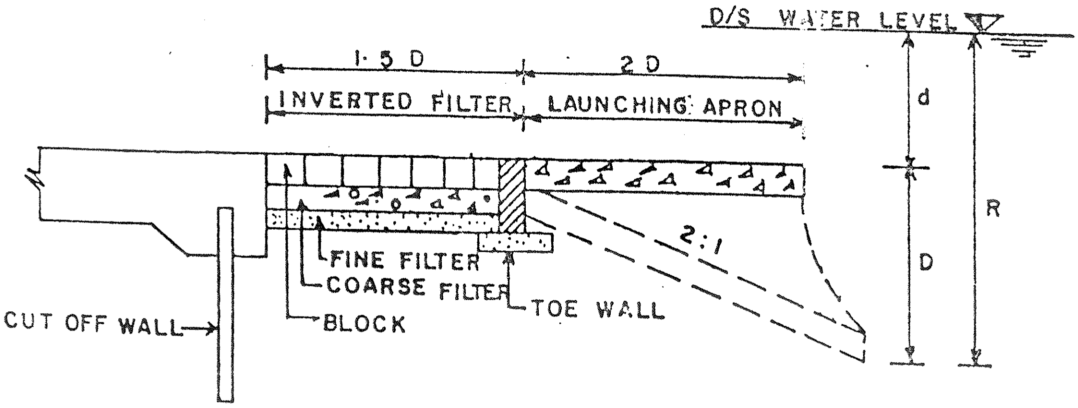


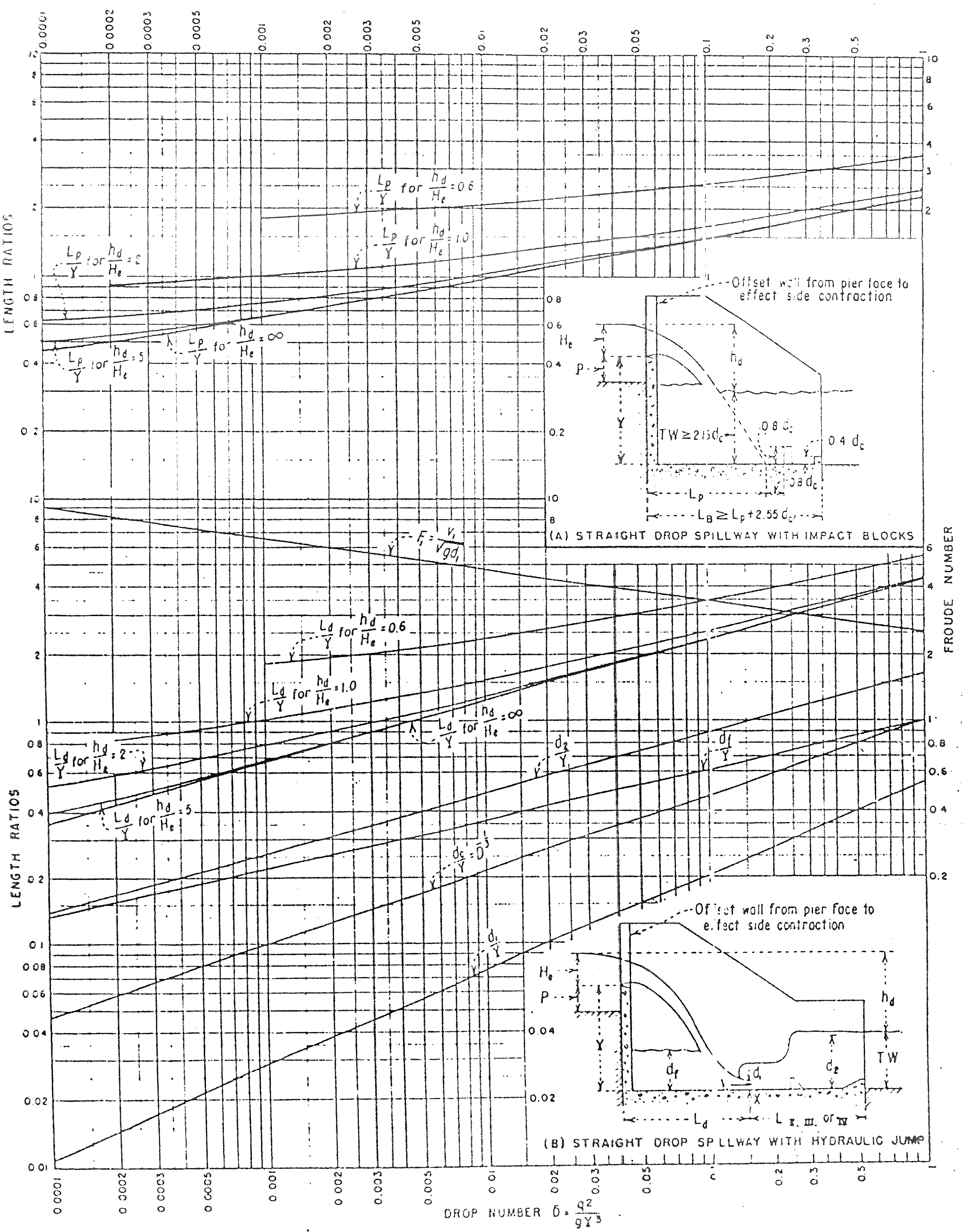
Figure 2.15

a) Details of Loose Protective Works (Upstream)



b) Details of Loose Protective Works (Downstream)

Fig. 2.16



Hydraulic characteristics of straight drop spillways with hydraulic jump or with impact blocks. 288-D-2437.

Table 2.1 Energy Loss in Hydraulic Jump

For explanatory diagram and equation used in derivation of this table refer Fig. 2.8

	0		0.1		0.2		0.3		0.4		0.5		0.6		0.7		0.8		0.9	
H/dc	d2/d1	d1/dc	d2/d1	d1/dc	d2/d1	d1/dc	d2/d1	d1/dc	d2/d1	d1/dc	d2/d1	d1/dc	d2/d1	d1/dc	d2/d1	d1/dc	d2/d1	d1/dc	d2/d1	d1/dc
0	1.0	1.0	2.07	.680	2.48	.614	2.81	.572	3.09	.541	3.35	.516	3.60	.494	3.82	.477	4.04	.461	4.24	.448
1	4.44	.436	4.64	.425	4.82	.415	5.00	.405	5.18	.397	5.36	.389	5.53	.381	5.69	.375	5.86	.368	6.02	.362
2	6.18	.356	6.33	.351	6.49	.345	6.64	.340	6.79	.336	6.94	.331	7.09	.327	7.23	.323	7.38	.319	7.52	.315
3	7.66	.311	7.80	.308	7.94	.304	8.07	.301	8.21	.298	8.34	.295	8.48	.292	8.61	.289	8.74	.286	8.87	.284
4	9.00	.281	9.13	.278	9.26	.276	9.39	.274	9.51	.271	9.64	.269	9.76	.267	9.89	.265	10.01	.263	10.13	.261
5	10.25	.259	10.38	.257	10.50	.255	10.62	.253	10.73	.251	10.85	.250	10.97	.248	11.09	.246	11.21	.244	11.32	.243
6	11.44	.241	11.55	.240	11.67	.238	11.78	.237	11.90	.235	12.01	.234	12.12	.233	12.24	.231	12.35	.230	12.46	.228
7	12.57	.227	12.68	.226	12.79	.225	12.90	.223	13.01	.222	13.12	.221	13.23	.220	13.34	.219	13.45	.218	13.56	.216
8	13.66	.215	13.77	.214	13.88	.213	13.98	.212	14.09	.211	14.19	.210	14.30	.209	14.41	.208	14.51	.207	14.61	.206
9	14.72	.205	14.82	.204	14.93	.203	15.03	.202	15.13	.202	15.23	.201	15.34	.200	15.44	.199	15.54	.198	15.64	.197
10	15.74	.197	15.84	.196	15.95	.195	16.05	.194	16.15	.193	16.25	.193	16.35	.192	16.45	.191	16.54	.191	16.64	.190
11	16.74	.189	16.84	.188	16.94	.187	17.04	.187	17.13	.186	17.23	.185	17.33	.185	17.43	.184	17.52	.183	17.62	.183
12	17.72	.182	17.81	.181	17.91	.181	18.01	.180	18.10	.180	18.20	.179	18.29	.178	18.39	.178	18.48	.177	18.58	.176
13	18.67	.176	18.77	.175	18.80	.175	18.95	.174	19.05	.174	19.14	.173	19.24	.173	19.33	.172	19.42	.171	19.52	.171
14	19.61	.170	19.70	.170	19.79	.169	19.89	.169	19.98	.168	20.07	.168	20.16	.167	20.25	.167	20.34	.166	20.44	.166
15	20.53	.165	20.62	.165	20.71	.164	20.80	.164	20.89	.164	20.98	.163	21.07	.163	21.16	.162	21.25	.162	21.34	.161
16	21.43	.161	21.52	.160	21.61	.160	21.70	.160	21.79	.159	21.88	.159	21.97	.158	22.05	.158	22.14	.157	22.23	.157
17	22.32	.157	22.41	.156	22.50	.156	22.58	.155	22.67	.155	22.76	.155	22.85	.154	22.93	.154	23.02	.154	23.11	.153
18	23.19	.153	23.28	.152	23.37	.152	23.45	.152	23.54	.151	23.63	.151	23.71	.151	23.80	.150	23.89	.150	23.97	.150
19	24.06	.149	24.14	.149	24.23	.148	24.31	.148	24.40	.148	24.49	.147	24.57	.147	24.66	.147	24.74	.146	24.83	.146
20	24.91	.146	24.99	.145	25.08	.145	25.16	.145	25.25	.145	25.33	.144	25.42	.144	25.50	.144	25.58	.143	25.67	.143
21	25.75	.143	25.83	.142	25.92	.142	26.00	.142	26.08	.141	26.17	.141	26.25	.141	26.33	.141	26.42	.140	26.50	.140
22	26.58	.140	26.66	.139	26.75	.139	26.83	.139	26.91	.139	26.99	.138	27.08	.138	27.16	.138	27.24	.138	27.32	.137
23	27.40	.137	27.48	.137	27.57	.136	27.65	.136	27.73	.136	27.81	.136	27.89	.135	27.97	.135	28.05	.135	28.13	.135
24	28.22	.134	28.30	.134	28.38	.134	28.46	.134	28.54	.133	28.62	.133	28.70	.133	28.78	.133	28.86	.132	28.94	.132
25	29.02	.132	29.10	.132	29.18	.131	29.26	.131	29.34	.131	29.42	.131	29.50	.131	29.58	.130	29.66	.130	29.74	.130
26	29.82	.130	29.89	.129	29.97	.129	30.05	.129	30.13	.129	30.21	.128	30.29	.128	30.37	.128	30.45	.128	30.52	.128
27	30.60	.127	30.68	.127	30.76	.127	30.84	.127	30.92	.127	31.00	.126	31.07	.126	31.15	.126	31.23	.126	31.31	.126
28	31.38	.125	31.46	.125	31.54	.125	31.62	.125	31.69	.125	31.77	.124	31.85	.124	31.93	.124	32.00	.124	32.08	.124
29	32.16	.123	32.23	.123	32.31	.123	32.39	.123	32.46	.123	32.54	.122	32.62	.122	32.69	.122	32.77	.122	32.85	.122
30	32.92	.121	33.00	.121	33.08	.121	33.15	.121	33.23	.121	33.31	.121	33.38	.120	33.46	.120	33.53	.120	33.61	.120
31	33.68	.120	33.76	.119	33.84	.119	33.91	.119	33.99	.119	34.06	.119	34.14	.119	34.21	.118	34.29	.118	34.36	.118
32	34.44	.118	34.51	.118	34.59	.118	34.66	.117	34.74	.117	34.81	.117	34.89	.117	34.96	.117	35.04	.117	35.11	.116
33	35.19	.116	35.26	.116	35.34	.116	35.41	.116	35.49	.116	35.56	.115	35.63	.115	35.71	.115	35.78	.115	35.86	.115
34	35.93	.115	36.00	.115	36.08	.114	36.15	.114	36.23	.114	36.30	.114	36.37	.114	36.45	.114	36.52	.113	36.59	.113
35	36.67	.113	36.74	.113	36.81	.113	36.89	.113	36.96	.112	37.03	.112	37.11	.112	37.18	.112	37.25	.112	37.33	.112
36	37.40	.112	37.47	.112	37.55	.111	37.62	.111	37.69	.111	37.76	.111	37.84	.111	37.91	.111	37.98	.111	38.05	.110
37	38.13	.110	38.20	.110	38.27	.110	38.34	.110	38.42	.110	38.49	.110	38.56	.109	38.63	.109	38.70	.109	38.78	.109
38	38.85	.109	38.92	.109	38.99	.109	39.06	.109	39.14	.108	39.21	.108	39.28	.108	39.35	.108	39.42	.108	39.49	.108
39	39.56	.108	39.64	.107	39.71	.107	39.78	.107	39.85	.107	39.92	.107	39.99	.107	40.06	.107	40.14	.107	40.21	.106
40	40.28	.106	40.35	.106	40.42	.106	40.49	.106	40.56	.106	40.63	.106	40.70	.106	40.77	.105	40.84	.105	40.91	.105

3.0 STRUCTURAL DESIGN OF REINFORCED CEMENT CONCRETE MEMBERS

3.1 General

Generally Working Stress Design Method will be followed for the design of RCC members. Ultimate Stress Design Method may also be followed. The permissible. stresses of concrete and steel are as given in the following sub-sections:

3.1.1 Concrete

Specified compressive strength (f_c') of concrete shall be 16.0 N/mm², 18.0 N/mm², 20.0 N/mm² and 22.0 N/mm² depending on the importance of the structure.

Modulus of elasticity of concrete

$$E_c = 4.73 \times 10^3 f'_c{}^{1/2} \text{ N/mm}^2 \quad (5.7 \times 10^4 f'_c{}^{1/2} \text{ psi})$$

3.1.2 Reinforcing Steel

Plain and deformed bars conforming to ASTM A575, A615 - Grade 40 & 60 (f_y = 276 N/mm² & 414 N/mm²) shall be used. For special cases high strength ribbed twisted bars conforming to Indian /Standard IS 1986 - 1979 or British Standard: BS 1144 & BS 4461 (1969) (f_y = 414 N/mm²) may also be used.

For small and light structures, mild steel with yield stress f_y = 248 N/mm² may be used.

Modulus of elasticity of steel

$$E_s = 2.00 \times 10^6 \text{ N/mm}^2 \quad (2.90 \times 10^7 \text{ psi})$$

3.1.3 Design Loads

I. Dead Loads

The unit weight of Dead Loads shall be as follows:

Plain Concrete	: 22.6 KN/m³	(144 lb/ft³)
Reinforced Cement Concrete	: 23.6 KN/m³	(150 lb/ft³)
Brick Masonry	: 18.9 KN/m³	(120 lb/ft³)
Dry Soil	: 15.7 KN/m³	(100 lb/ft³)
Compacted Soil	: 18.9 KN/m³	(120 lb/ft³)
Moist Soil	: 18.1 KN/m³	(115 lb/ft³)
Saturated Soil	: 19.6 KN/m³	(125 lb/ft³)
Water	: 9.8 KN/m³	(62.4 lb/ft³)

Timber:

(i) Hard	: 7.1-12.6 KN/m³	(45-80 lb/ft³)
(ii) Soft	: 4.7-7.1 KN/m³	(30-45 lb/ft³)

Stone	:	25.5 KN/m^3 (162 lb/ft^3)
Bitumen	:	13.7 KN/m^3 (87 lb/ft^3)
Steel	:	77.0 KN/m^3 (490 lb/ft^3)
Cement	:	14.1 KN/m^3 (90 lb/ft^3)

II. Live Loads

Bridge and culvert : The live loads on bridge and culvert shall be in accordance with the Article 9.1.7.2.

Buildings : The live loads on buildings: roof, floor and parts thereof shall be as follows:

Floor (Residential)	:	1.90 KN/m^2 (40 lb/ft^2)
Roof (Residential)	:	0.95 KN/m^2 (20 lb/ft^2)
Floor (Office)	:	3.80 KN/m^2 (80 lb/ft^2)
Roof (Office)	:	1.90 KN/m^2 (40 lb/ft^2)
Varandah, Balcony	:	3.80 KN/m^2 (80 lb/ft^2)
Stairs, landings	:	4.80 KN/m^2 (100lb/ft^2)

Hydraulic Structures : The live loads on Hydraulic Structures shall be as follows:

Road Deck Slab/Embankment :

Operating Platform :

- Without Stoplogs : 4.80 KN/m^2 (100 lb/ft^2)
- With Stoplogs : 7.20 KN/m^2 (150 lb/ft^2)

Construction Loading (when operating) : 12.00 KN/m^2 (250 lb/ft^2)

Live Loads for the Design of Bridge Decks Over Hydraulic Structures

Type of roadway	Type of crossing	Lane width	Loading
Village footpath	Footbridge or culvert	1.85 m (6 ft)	4.00 KN/m^2 (150 lb/ft^2)
Village road	Bridge or culvert	3.70 m (12 ft)	H_{10}
Feeder roads	Bridge or culvert	3.70 m (12 ft)	H_{15}
District Council and Highway	Bridge or culvert	3.70 m (12 ft) & 7.30 m (24ft)	H_{20} & H_{20}-S_{16}

Where H_{10}, H_{15} and H_{20} are as specified in AASHTC.

3.1.4 Design Factor of Safety

The following minimum factor of safety will be taken into consideration to stability of structure:

Overturning	:	Normal Case	:	1.50
		Extreme Case	:	1.30
Sliding	:	Normal Case	:	1.50
		Extreme Case	:	1.30

3.1.5 Clear Concrete Cover to Reinforcement

Description of Concrete Element	Cover (mm)	
	Normal Exposure	Saline Exposure
Wall and floor slab		
- Concrete with earth	60	75
- Exposed to weather & water	50	60
Piles		
- Cast-in-place	75	100
- Precast	40	50
Beam, Girder, Column	40	50
Building roof and floor slab	25	25
Bridge deck slab	40	40
Railing	25	25

3.1.6 Fillets

Fillets are used to provide increased strength or to relieve stress concentration at points of maximum stress. Table 3.1 gives the dimensions of fillets which are to be provided at the inside corners of box sections and at bases of vertical cantilever walls.

Table 3.1

Dimension of Fillet

Size of fillet	Size of box section	Clear vertical height of cantilever
50 mm x 50 mm 75 mm x 75 mm 100 mm x 100 mm 150 mm x 150 mm	0 to 1.25 m 1.25 m to 1.85 m 1.80 m to 2.45 m Over 2.45 m	- 0 to 2.45 m 2.45 m to 3.70 m Over 3.7 m
No reinforcement is needed for fillets upto 150 mm x 150 mm in size		

3.1.7 Backfill Soil Parameters

Special attention is to be given to the quantity of Backfill to wingwalls and returnwalls and where practicable sand-fill adopted.

Specified sand of Φ = 30° (for backfill of wingwall & returnwall)
In-situ soil of Φ = 20° (for Box conduit, abutment & headwalls)

Unit weight of soil : Moist soil = 18.10 KN/m^3
Saturated Soil = 19.60 KN/m^3

3.1.8 Earth Pressure

- Rankine's active earth pressure theory will be used for cantilever & counterfort retaining wall
- Coulomb's active earth pressure theory will be used for gravity wall
- The earth pressure on side slabs of conduits, (box culverts) shall be determined using earth pressure at rest (C_o).

3.1.9 Distribution of Truck Loading on Concrete Slab and Filled Earth

These will be as per following AASHTO design code of practice:

(i) The distribution of loads for wheels resting directly on the slab can be computed as follows:

$$E = (1.219 + 0.06S) \text{ with maximum of } 2.134 \text{ m.}$$

where,
E = distribution width (m) and
S = design span of slab (m)

ii) The distribution of wheel loads transmitted through fills are as follows:

(a) When the depth of fill is 0.60 m or more, concentrated loads shall be considered as uniformly distributed over a square, the sides of which are equal to 1.75 times the depth of fill. When such areas from several concentrations overlap, the total load shall be considered as uniformly distributed over the area defined by the outside limits of the individual areas.

(b) for single span the effect of live load may be neglected when the depth of fill is more than 2.44 m and exceeds the span length, for multiple spans it may be neglected when the depth of fill exceeds the distance between and supports or abutments.

(c) In no case shall the stresses for load as determined by the above be greater than those obtained where the wheel is applied directly on the structure.

(d) In no case shall the total width of distribution exceed the total width of supporting slab.

iii) The percentage impact (I) of vehicle on concrete slab should not exceed 30% and is given by the equation.

$$I = 15.2/(L+38)$$

Where, L = Span of the bridge deck (m)

iv) The percentage impact of vehicle on earth fill of various heights are as follows:

For fills	upto	0.30 m,	impact = 30%
"	"	0.30 m to 0.60 m,	impact = 20%
"	"	0.60 m to 0.90 m,	impact = 10%
"	"	0.90 m and over,	impact = 0%

(v) The depth of earth surcharge equivalent to truck loading shall be a maximum of 0.90m.

3.2 Working Stress Design Method

3.2.1 Concrete Stresses

(a) *Flexural Stress*

f_c = $0.40f'_c$, for bridges & Pump House.
= $0.45f'_c$, for other structures.

Where, f'_c = Specified compressive strength of concrete.

(b) *Shear Stress*

Flexural (without web reinf.)

$V_c = 9.10 \times 10^{-2} f'_c$ N/mm² ($1.1\ f'^{1/2}_c$ psi)

Flexural (with web reinf.)

$V_c = 0.42f'_c$ N/mm² ($5\ f'_c$ psi)

Punching = $0.17\ f'^{1/2}_c$ N/mm². ($2\ f'^{1/2}_c$ psi)

(c) *Bond Stress*

Top bar,

$$3.6\frac{f'^{1/2}_c}{d} \leq 1.10 N/mm^2 \ (1.7\frac{f'^{1/2}_c}{d} \leq 160 psi)$$

All other bars,

$$5.1\frac{f'^{1/2}_c}{d} \leq 1.10 N/mm^2 \ (2.4\frac{f'^{1/2}_c}{d} \leq 160 psi)$$

Where,
d = diameter of bar (mm or in)

(d) *Bearing Stress*

Load on full area:

= $0.25\ f'_c$ N/mm² ($0.25\ f'_c$ psi)

Load on one third area or less:

= $0.375\ f'_c$ N/mm² ($0.375\ f'_c$ psi)

3.2.2 Steel Stresses

ACI building Code

$$f_s = 0.45\ f_y$$

where, f_y = specified yield stress of steel.

3.2.3 Design Constants

$$n = \frac{E_s}{E_c}$$

$$r = \frac{f_s}{f_c}$$

$$k = \frac{f_c}{(f_s/n) + f_c}$$

$$j = 1 - k/3$$

$$R = 0.5 f_c k j$$

(Specific values for these stresses and constants are given at section 3.2.4 below)

3.2.4 Allowable Stresses in Design Constants

Allowable Stresses and Design Constants are given in the following Tables.

Table 3.2

Allowable Stresses and Design Constants
(for bridges & pump house)

f_y = 276 N/mm²

Design Parameters	Ultimate concrete strength (f'_c)			
	16.00 N/mm² 2320 psi	18.00 N/mm² 2610 psi	20.00 N/mm² 2900 psi	22.00 N/mm² 3190 psi
f_y	276 N/mm² 40000 psi	276 N/mm² 40000 psi	276 N/mm² 40000 psi	276 N/mm² 40000psi
f_s	124 N/mm² 18000 psi	124 N/mm² 18000 psi	124 N/mm² 18000 psi	124 N/mm² 18000 psi
$f_c = 0.40f'_c$	6.40 N/mm² 930 psi	7.20 N/mm² 1040 psi	8.00 N/mm² 1160 psi	8.80 N/mm² 1276 psi
n	11	10	9	9
r	19	17	16	14
k	0.362	0.367	0.367	0.389
j	0.879	0.878	0.877	0.870
R	1.02 N/mm² 148 psi	1.16 N/mm² 167 psi	1.29 N/mm² 187 psi	1.49 N/mm² 216 psi
E_s	2.0×10^5 N/mm² 2.9×10^7 psi	2.0×10^5 N/mm² 2.9×10^7 psi	2.0×10^5 N/mm² 2.9×10^7 psi	2.0×10^5 N/mm² 2.9×10^7 psi
E_c	1.89×10^4 N/mm² 2.74×10^6 psi	2.0×10^5 N/mm² 2.91×10^6 psi	2.12×10^4 N/mm² 3.08×10^6 psi	2.22×10^4 N/mm² 3.22×10^6 psi
V_c	0.364 N/mm² 53 psi	0.386 N/mm² 56 psi	0.407 N/mm² 59 psi	0.427 N/mm² 62 psi

Note: Figures in parentheses indicate equivalent stress in pounds per square inch, psi.

Table 3.3

Allowable stresses and Design Constants
(for Regulators/Culverts/Buildings)

f_y = 276 N/mm²

Design Parameters	Ultimate concrete strength (f'_c)			
	16.00 N/mm² 2320 psi	18.00 N/mm² 2610 psi	20.00 N/mm² 2900 psi	22.00 N/mm² 3190 psi
f_y	276 N/mm² 40000 psi	276 N/mm² 40000 psi	276 N/mm² 40000 psi	276 N/mm² 40000psi
f_s	124 N/mm² 18000 psi	124 N/mm² 18000 psi	124 N/mm² 18000 psi	124 N/mm² 18000 psi
f_c $=0.45f'_c$	7.20 N/mm² 1040 psi	8.10 N/mm² 1175 psi	9.00 N/mm² 1305 psi	9.90 N/mm² 1435 psi
n	11	10	9	9
r	17	15	14	13
k	0.389	0.395	0.395	0.418
j	0.870	0.868	0.868	0.861
R	1.22 N/mm² 176 psi	1.39 N/mm² 201 psi	1.54 N/mm² 224 psi	1.78 N/mm² 258 psi
E_s	2.0×10^5 N/mm² 2.9×10^7 psi	2.0×10^5N/mm² 2.9×10^7 psi	2.0×10^5N/mm² 2.9×10^7 psi	2.0×10^5N/mm² 2.9×10^7 psi
E_c	1.89×10^4N/mm² 2.74×10^6 psi	2.00×10^4N/mm² 2.91×10^6 psi	2.12×10^4N/mm² 3.08×10^6 psi	2.22×10^4N/mm² 3.23×10^6 psi
V_c	0.364 N/mm² 53 psi	0.386 N/mm² 56 psi	0.407 N/mm² 59 psi	0.427 N/mm² 62 psi

Table 3.4

Allowable stresses and Design Constants
(for Minor structures)

$f_y = 248\ N/mm^2$

Design Parameters	Ultimate concrete strength (f'_c)	
	16.00 N/mm^2 2320 psi	18.00 N/mm^2 2610 psi
f_y	248 N/mm^2 36000 psi	248 N/mm^2 36000 psi
f_s	112 N/mm^2 16200 psi	112 N/mm^2 16200 psi
$f_c = 0.40f'_c$	7.20 N/mm^2 1040 psi	8.10 N/mm^2 1175 psi
n	11	10
r	16	14
k	0.414	0.420
j	0.862	0.859
R	1.28 N/mm^2 186 psi	1.46 N/mm^2 212 psi
E_s	$2.0 \times 10^5\ N/mm^2$ 2.9×10^7 psi	$2.0 \times 10^5\ N/mm^2$ 2.9×10^7 psi
E_c	$1.89 \times 10^4\ N/mm^2$ 2.74×10^6 psi	$2.00 \times 10^5\ N/mm^2$ 2.91×10^6 psi
V_c	0.364 N/mm^2 53 psi	0.386 N/mm^2 56 psi

Table 3.5

Allowable stresses and Design constants

f_y = 414 N/mm²

Design Para meters	Ultimate concrete strength (f'_c)	
	20.00 N/mm² 2900 psi	22.00 N/mm² 3190 psi
f_y	414 N/mm² 60000 psi	414 N/mm² 60000 psi
f_s	186.21 N/mm² 27000 psi	186.21 N/mm² 27000 psi
f_c = $0.40f'_c$	9.00 N/mm² 1305 psi	~~9.90~~ N/mm² 8.8 1436 psi
n	9	9
r	21	19
k $= \frac{n}{n+f_s/f_c}$	0.303	0.324 0.298
j	0.899	0.892 0.901
R $= 0.5 f_c k j$	1.23 N/mm² 178 psi	1.~~43~~ N/mm² 1.18 208 psi
E_s	2.00×10^5 N/mm² 2.90×10^7 psi	2.00×10^5 N/mm² 2.90×10^7 psi
E_c	2.12×10^4 N/mm² 3.08×10^6 psi	2.22×10^5 N/mm² 3.22×10^6 psi
V_c	0.407 N/mm² 59 psi	0.427 N/mm² 62 psi

3.2.5 Reinforcement Bar

Plain mild steel reinforcement bars of following sizes and areas will be used.

Table 3.6

Reinforcement Bar Dia and X-sectional Area

Bar dia (in)	Equivalent Bar dia (mm)	Bar X-Sectional Area (mm^2)	Bar dia (mm)	Bar X'-Sectional Area (mm^2)
1/4	6.350	31.67	6	28.27
3/8	9.525	71.26	10	78.54
1/2	12.700	126.68	12	113.10
5/8	15.875	197.93	16	201.06
3/4	19.050	285.02	19 - 20	283.53 - 314.0
7/8	22.225	387.95	22	380.13
1	25.400	506.71	25	490.87
1-1/8	28.575	641.30	28	615.75
1-1/4	31.750	791.73	32	804.25

3.2.6 Maximum and Minimum Spacing of Reinforcement

Maximum spacing : 450 mm ($\geq$16 mm Φ bar)
: 300 mm ($\leq$12 mm ϕ bar)

The minimum clear distance between parallel bars or groups or bars (which must include consideration of lapped bars) shall be as follows:

For slab & beam : shall not be less than as follows or 50 mm whichever is the greater.

- Nominal bar dia
- 1.33 times the maximum size of coarse aggregate

For column : Not less than as follows; or 50 mm whichever is the greater.

- 1.5 times the bar
- 1.5 times the maximum size of coarse aggregate

3.2.7 Splice and Bond Length

For a fully stressed plain bar spliced in the tension zone, a splice length of 48 bar diameter is to be used. When spliced in compression zone, a splice length of 40 bar diameter is to be used. The bond or development length of bars should not be less than $f_s.d/4u$, where d is the diameter of bar and u is 0.8 times the allowable bond stress. The splice and bond lengths of different bars are given in Table 3.7 and Table 3.8 respectively.

Table 3.7

Splice Length for Plain Reinforcing Bars without Hooks

Bar dia (mm)	Splice length (mm)	
	Tension Zone	Compression Zone
10 (3/8")	480	400
12 (1/2")	580	480
16 (5/8")	770	640
19 (3/4")	910	760
22 (7/8")	1060	880
25 (1")	1200	1000
28 (1-1/8")	1350	1120
32 (1-1/4")	1550	1280

Table 3.8

Bond or development length (mm) of Plain Reinforcing Bar without Hook

Bar Dia	f'_c = 18.00 N/mm²		f'_c = 20 N/mm²	
	* Top Bar	Other Bar	* Top bar	Other Bar
10	352	352	352	352
12	423	423	423	423
16	653	564	619	564
19	921	669	873	669
22	1235	872	1171	826
25	1595	1126	1512	1067
28	2000	1412	1896	1339
32	2613	1844	2477	1749

* Top bar are horizontal bar with 300 mm or more concrete poured below the bar.

3.2.8 Minimum Reinforcement or Temperature Reinforcement

This minimum area of reinforcement is required to control the cracking, which occurs in the concrete due to temperature, shrinkage and creep. It enables the cracking to be uniformly distributed and therefore minimises individual drack widths.

Except for very small structures, the following criteria should be used to determine the cross-sectional area of temperature or minimum reinforcement required in hydraulic structures. The percentages indicated are based on the gross cross-sectional area, not including fillets, of the concrete to be reinforced. Where the thickness of the section exceeds 380 mm (15in), a thickness of 380 mm should be used in determining the temperature or minimum reinforcement.

(i) *Single layer Reinforcement.*

o	slabs not exposed to direct sun with spacing of joints not exceeding 9.0 m	0.25%
o	slabs exposed to direct sun with spacing of joints not exceeding 9.0 m	0.30%
o	slabs exceeding 9.0 m between joints and not exposed to direct sun	0.35%
o	slabs exceeding 9.0 m between joints and exposed to direct sun	0.40%

(ii) *Double layer Reinforcement (in each direction)*

o	face adjacent to earth with spacing of joints not exceeding 9.0 m	0.10%
o	face not adjacent to earth not exposed to direct sun and with spacing of joints not exceeding 9.0 m	0.15%
o	face not adjacent to earth but exposed to direct sun and with spacing of joints not exceeding 9.0 m	0.20%
o	if member exceeds 9.0 m in any direction parallel to reinforcement, add to the reinforcement requirement in that direction because of the increased length	+ 0.05%

Table 3.9 & 3.10 may be used to select the desired minimum reinforcement or temperature reinforcement.

Table 3.9

Minimum reinforcement of Temperature reinforcement
Member Length < 9.0 m

Member Thickness	Location of Reinforcement	Quantity of Reinf. (cm^2/m)	Dia and Spacing provided
30 cm	Adjacent to earth	0.0010x30x100 = 3.0	12 mm @ 300 c/c
	Not exposed to direct sun	0.0015x30x100 = 4.5	12 mm @ 250 c/c
	Exposed to direct sun	0.0020x30x100 = 6.0	12 mm @ 180 c/c or, 16 mm @ 300 c/c
35 cm	Adjacent to earth	0.0010x35x100 = 3.5	12 mm @ 300 c/c
	Not exposed to direct sun	0.0015x35x100 = 5.25	12 mm @ 200 c/c
	Exposed to direct sun	0.0020x35x100 = 7.0	12 mm @ 150 c/c or, 16 mm @ 280 c/c
38 cm or greater	Adjacent to earth	0.0010x38x100 = 3.8	12 mm @ 300 c/c
	Not exposed to direct sun	0.0015x38x100 = 5.7	12 mm @ 200 c/c
	Exposed to direct sun	0.0020x38x100 = 7.6	16 mm @ 250 c/c
N.B. These quantities correspond to double layer reinforcement			

Table 3.10

Minimum reinforcement or Temperature reinforcement (contd)
Member Length > 9.0 m

Member Thickness	Location of Reinforcement	Quantity of Reinforcement (cm^2/m)	Dia and Spacing Provided
30 cm	Adjacent to earth	0.0015x30x100 = 4.5	12 mm @ 250 c/c
	Not exposed to direct sun	0.0020x30x100 = 6.0	12 mm @ 180 c/c or 16 mm @ 300 c/c
	Exposed to direct sun	0.0025x30x100 = 7.5	12 mm @ 150 c/c or 16 mm @ 250 c/c
35 cm	Adjacent to earth	0.0015x35x100 = 5.25	12 mm @ 200 c/c
	Not exposed to direct sun	0.0020x35x100 = 7.0	12 mm @ 150 c/c or 16 mm @ 280 c/c
	Exposed to direct sun	0.0025x35x100 = 8.75	16 mm @ 220 c/c
38 cm or greater	Adjacent to earth	0.0015x38x100 = 5.7	12 mm @ 200 c/c or 16 mm @ 350 c/c
	Not exposed to direct sun	0.0020x38x100 = 7.6	16 mm @ 250 c/c
	Exposed to direct sun	0.0025x38x100 = 9.5	16 mm @ 200 c/c
N.B. These quantities correspond to double layer reinforcement			

3.2.9 Web Reinforcement

Vertical or inclined stirrups must be designed to resist the shear stress in excess of that which can be taken by the concrete alone i.e.

$$v' = (v - v_c)$$

where,

v'	=	excess shear stress for which web reinforcement has to be provided
v	=	developed shear stress = V/bd
v_c	=	allowable shear stress
V	=	total shear
b	=	width of the section
d	=	effective depth of the section

Longitudinal spacing S of the stirrups can be computed as follows:

For vertical stirrups:

$$s = \frac{A_v f_v}{v' b}$$

For inclined stirrups:

$$S = \frac{A_v f_v (\sin\alpha + \cos\alpha)}{v' b}$$

Where:

s	=	centre to centre spacing of the stirrups
A_u	=	cross-sectional area of the stirrups
f_u	=	allowable stress of the steel
v'	=	excess shear stress for which stirrups are to be provided.
b	=	width of the section
α	=	angle of inclination of the stirrups to the horizontal.

Where stirrups are required, under no circumstances should the spacing exceed d/2. Spacing should not exceed d/4 when

$$\frac{V}{bd} > 3\sqrt{f'_c}$$

Where web reinforcement is required the following conditions must be met:

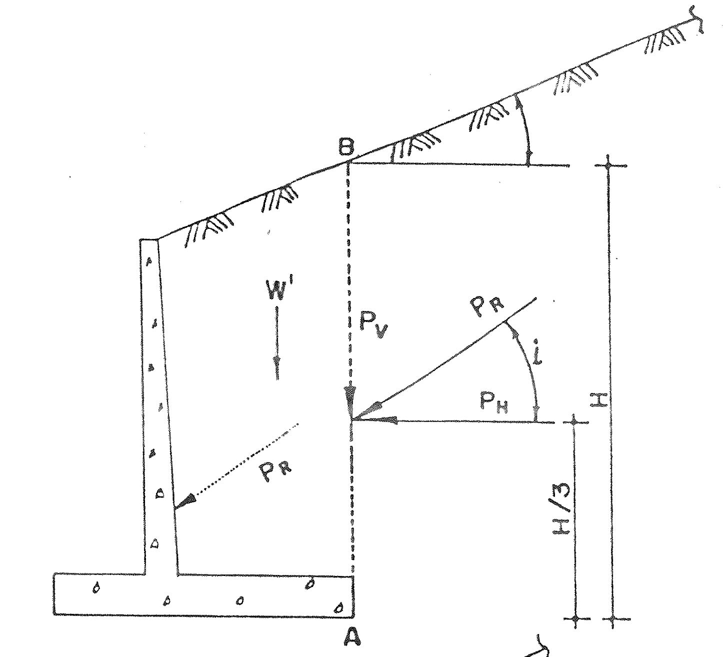
- A_v may not be less than 0.0015 bS for any vertical
- No single type of web reinforcement may resist a total shear force greater than 2v'/3 nor may the total shear stress exceed $5f'^{1/2}_c$.

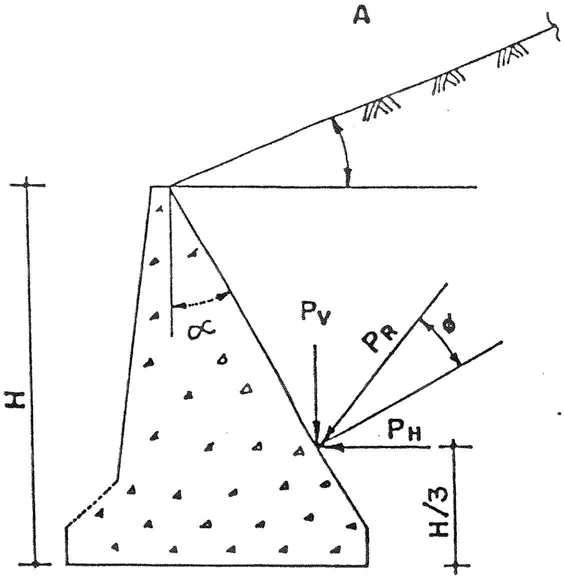
3.2.10 Minimum Member Thickness

Minimum Wall and Slab Thickness

Walls having two layers of reinforcement normally have a minimum thickness of 250 mm (10 in).Cantilever walls are to have a minimum thickness at the base equal to 25 mm (1 in) per 300 mm (12 in) of height, with absolute minimum of 120 mm (5 in) upto 2.45 m (8 ft). Above 2.45 m (8 ft) the minimum thickness at the base is to be 200 mm (8 in) plus 20 mm (3/4 in) for each 300 mm (1 ft) in height above 2.45 m (8 ft).

Fig. 3·1





P_R	Resultant soil pressure on vertical plane AB.
P_H	Horizontal component of Resultant P_R
P_V	Vertical component of Resultant P_R
H	Height of plane AB/Height of wall
i	Angle of backfill slope
Ø	Angle of internal friction of soil
Ø	Angle of friction between soil and wall
$\propto$	Angle of wall face with the vertical soil
W'	Weight of soil mass between plane AB and wall
ΣP_R	Total soil pressure on wall
	Resultant of P_R and W'
γ	Unit Weight of soil

RANKINE'S GENERAL FORMULA FOR EARTH PRESSURE

P_R = 1/2" $K\gamma H^2$

$$K = \text{Cos}\left[\frac{\text{Cos} - \sqrt{\text{Cos}^2 - \text{Cos}^2 Ø}}{\text{Cos} + \sqrt{\text{Cos}^2 - \text{Cos}^2 Ø}}\right]$$

When $i = 0$, $K = \dfrac{1 - \text{Sin}\,Ø}{1 + \text{Sin}\,Ø}$: When $i = Ø$, $K = \text{Cos}\,Ø$

COULOMB'S GENERAL FORMULA FOR EARTH PRESSURE

P_R = 1/2 $K\gamma H^2$

$$K = \frac{\text{Cos}^2(Ø - \propto)}{\text{Cos}^2\propto\ \text{Cos}(Ø + \propto)\left[1 + \sqrt{\dfrac{\text{Sin}(Ø + Ø)\,\text{Sin}(Ø - i)}{\text{Cos}(Ø + \propto)\,\text{Cos}(\propto - i)}}\right]^2}$$

3.3 Ultimate Strength Design Method

3.3.1 Concrete stress

Shear Stress :

Flexural (without web reinforcement)

$V_c = 0.166\Phi\sqrt{f'c}$ N/mm² $(2\Phi\sqrt{f'c}$ psi)

Pouncing : $0.332\Phi\sqrt{f'c}$ N/mm² $(4\Phi\sqrt{f'c}$ psi)

3.3.2 Load Factors

Factored Load Combinations for determining Required Strength U shall be as follows:

(a) *Basic Condition:*

Required strength U to resist dead load D and live load L shall be at least equal to

$$U = 1.4D + 1.7L$$

(b) *When Wind Forces W are considered:*

$$U = 0.75(1.4D + 1.7L + 1.7W)$$

Where load combinations shall include both full value and zero value of L to determine the more severe condition and

$$U = 0.9D + 1.3W$$

But for any combination of D, L and W, required strength U shall not be less than

$$U = 1.4D + 1.7L$$

(c) *When Earthquake loads or forces E are included in design:*

$$U = 0.75(1.4D + 1.7L + 1.87E)$$

and including consideration of L = 0

$$U = 0.9D + 1.43E$$

But for any combination of D, L and E required strength U shall not be less than

$$U = 1.4D + 1.7L$$

(d) *When Earth Pressure, H is considered:*

If effects H caused by earth pressure, ground water pressure or pressure caused by granular materials are included in design, the required strength equations become:

$$U = 1.4D + 1.7L + 1.7H$$

and where D or L reduce the effect of H

$$U = 0.9D + 1.7H$$

But for any combination of D,L or H U shall not be less than

$$U = 1.4D + 1.7L$$

(e) *When loading due to weight and pressure of fluids F is considered:*

For well defined fluid pressure, the required strength equations become:

$$U = 1.4D + 1.7L + 1.4F$$

and where D or L reduce the effect of F

$$U = 0.9D + 1.4F$$

But for any combination of D, L or F required strength shall not be less than

$$U = 1.4D + 1.7L$$

(f) *When Impact effect is taken into account in Design :*

If the Live Load is applied rapidly, as may be the case for parking structures, loading docks, warehouse floors, elevator shafts etc. impact effects should be considered.

In all equations substitute (L + I) for L when impact must be considered.

(g) *When effects of Differential Settlement, Creep, Shrinkage and Temperature changes are significant in Design :*

$$U = 0.75(1.4D + 1.7L + 1.4T)$$

But shall not be less than

$$U = 1.4(D + T)$$

3.3.3 Strength Reduction Factors

Kind of Strength	Strength Reduction Factor, Φ
Flexure, without axial load	0.90
Axial load, and axial load with flexure :	
(a) Axial tension, and axial tension with flexure	0.90
(b) Axial compression, and axial compression with flexure	
Members with spiral reinforcement	0.75
Other reinforced members	0.70
Except that for low values of axial load Φ may be increased in accordance with the following :	
For members in which f_y does not exceed 60,000 psi with symmetrical reinforcement and with $(h-d'-d_s)/h$ not less than 0.70, Φ may be increased linearly to 0.90 as ΦP_n decreases from $0.10f'_cA_g$ to zero.	
For other reinforced members, Φ may be increased linearly to 0.90 as ΦP_n decreases from $0.10f'_cA_g$ or ΦP_{nb}, whichever is smaller, to zero.	
Shear and torsion	0.85
Bearing on concrete	0.70

3.3.4 Development Length (l_d)

Development length of deformed bars and deformed wire in tension:

Development length l_d, in inches, for deformed bars and deformed wire in tension shall be computed as the product of the basic development length l_{db} in tension and the applicable modification factors of Table 3.11, but l_d shall not be less than 12".

(i) Basic Development Length (l_{db}) in tension :

$$l_{db} = \frac{0.04A_bf_y}{\sqrt{f'_c}}$$ for No.11 bars and smaller and deformed wire

$$l_{db} = \frac{0.085f_y}{\sqrt{f'_c}}$$ for No.14 bars.

$$l_{db} = \frac{0.125 f_y}{\sqrt{f'_c}} \quad \text{for No.18 bars.}$$

Where,

A_b = cross sectional area of the individual bar, in^2
f_y = Specified yield strength of non pre-stressed reinforcement, psi
f'_c = specified Compressive strength of concrete, psi.

It is to be noted here that values of $\sqrt{f'c}$ are not to be taken larger than 100 psi because of the lack of experimental evidence on bond strengths obtainable with concrete having compressive strength in excess of 10,000 psi.

Table 3.11

Modification factors to be applied to basic development length to obtain required development length l_d in tension

---

A Modification factors for bar spacing, cover, and transverse steel:

1. Bars satisfying any one of conditions (a), (b), (c), or (d) as described in the following 1.0

(a) Bars in beams or columns with minimum cover as required for concrete protection for reinforcement and with transverse reinforcement satisfying the requirements for compression members or minimum stirrup requirements along the development length for beams, and with clear spacing of not less than $3d_b$

(b) Bars in beams or columns with minimum cover as required for concrete protection for reinforcement and enclosed within transverse reinforcement A_{tr} along the development length of minimum area

$$A_{tr} \geq d_b sN/40$$

where,

A_{tr} = total cross-sectional area of transverse steel within a spacing s and perpendicular to the plane of bars being spliced or developed, in^2

d_b = diameter of the bar being developed, in.

s = spacing of stirrups or ties, in.

N = number of bars in layer being spliced or developed

(c) Bars with cover of d_b or less or with clear spacing $2d_b$ and with clear spacing not less than $3d_b$.

(d) Any bars with cover not less than $2d_b$ and with clear spacing not less than $3d_b$.

2. Bars with cover od d_b or less or with clear spacing $2d_b$ or less 2.0

3. Bars not included in 1 or 2 ... ... 1.4

4. For No. 11 bars and smaller with clear spacing not less than 5db and with cover from face of member to edge bar, measured in the plane of the bars, not less than 2.5 d_b, the factors of 1,2, or 3 may be multiplied by 0.8

5. For reinforcement enclosed withinspiral reinforcement not less than 1/4 in. diameter and not more than 4 in.pitch, within No. 4 or larger circular ties spaced at not more than 4 in. on center, or within No. 4 or larger ties or stirrups spaced not more than 4 in. on center and arranged such that alternate bars shall have support provided by the corner of a ties or hoop with an included angle of not more than 135 degrees, the factors of 1,2 or 3 may be multiplied by 0.75

6. The basic development length, multiplied by the applicable factor of 1, 2, or 3, with modifiers of 4, 5 or both, shall not be less than $0.03d_bf_y/\sqrt{f'_c}$.

B. Other modification factors:

The basic development length l_{db} as modified by the factors of part A shall also be multiplied by applicable factor or factors as follows:

1. Top reinforcement, i.e., horizontal reinforcement so placed that more than 12 in. of fresh concrete is cast in the member below the development length or splice .. 1.3

2. Lightweight aggregate concrete 1.3
or when f_{ct} is specified ... ... $6.7\sqrt{f'_c}/f_{ct}$
... ... but not less than 1.0

3. Epoxy-coated reinforcement

(a) Bars with cover less than $3d_b$ or clear spacing between bars less than $6d_b$ 1.5

(b) All other conditions 1.2

(c) The product of the factor for top reinforcement and for epoxy-coated reinforcement need not be taken greater than 1.7

4. Excess reinforcement

Development length may be reduced where reinforcement in a flexural member is in excess of that required by analysis, except where anchorage or development for f_y is spcifically required or the reinforcement is designed for a region of high seismic risk, by the factor (A_s required/A_s provided) *(ii) Development length of deformed bars in compression*

Development length l_d, in inches, for deformed bars in compression shall be computed as the product of the basic development length l_{db} in compression and the applicable modification factors of Table 3.12, but l_d shall not be less than 8".

Basic development length in compression is :

$$l_{db} = \frac{0.02 d_b f_y}{\sqrt{f'_c}} \quad but \geq 0.0003 d_b f_y$$

Table 3.12

Development Length

(i) Reinforcement in excess of that required by analysis	$\frac{\text{As required}}{\text{As provided}}$
(ii) Reinforcement enclosed within spiral reinforcement not less than 1/4" diameter and not more than 4" pitch or within No. 4 ties spaced at not more than 4" on centres	0.75

(iii) Development lengths for hooked deformed bars in tension

A.	Basic development length lhb for hooked bars with fy = 60,000 psi	$\frac{1200 d_b}{\sqrt{f'_c}}$
B.	Modification factors to be applied to l_{hb}	
	Bars with fy other than 60,000 psi	$\frac{f_y}{60,000}$
	For No.11 bars and smaller, side cover not less than 2½ in. and for 90° hook, cover on bar extension beyond hook not less than 2 in.	0.7
	For No.11 bars and smaller, hook enclosed vertically or horizontally within ties or stirrup-ties spaced along the full development length ldh with spacing not greater than 3db	0.8
	Reinforcement in excess of that required by analysis	$\frac{A_s \text{ required}}{A_s \text{ provided}}$
	Lightweight aggregate concrete	1.3

3.3.5 Maximum allowable computed deflections

<table>
<tr><th>Type of member</th><th>Deflections to be considered</th><th>Deflection Limitation</th></tr>
<tr><td>Flat roofs not supporting or attached to nonstructural elements likely to be damaged by large deflections</td><td>Immediate deflection due to the live load L</td><td>1/180</td></tr>
<tr><td>Floors not supporting or attached to nonstructural elements likely to be damaged by large deflections.</td><td>Immediate deflection due to the live load L.</td><td>1/360</td></tr>
<tr><td>Roof or floor construction supporting or attached to nonstructural elements likely to be damaged by large deflections</td><td rowspan="2">That part of the total deflection which occurs after attachment of the nonstructural elements, the sum of the long-time deflection due to all sustained loads, and the immediate deflection due to any additional live load.</td><td>1/480</td></tr>
<tr><td>Roof or floor construction supporting or attached to nonstructural elements not likely to be damaged by large deflections</td><td>1/240</td></tr>
</table>

3.3.6 **Minimum depth of beams or one-way slabs unless deflections are computed**

<table>
<tr><th rowspan="3">Member</th><th colspan="4">Minimum thickness, h</th></tr>
<tr><th>Simply supported</th><th>One end continuous</th><th>Both ends continuous</th><th>Cantilever</th></tr>
<tr><td colspan="4">Members not supporting or attached to partitions or other construction likely to be damaged by large deflections</td></tr>
<tr><td>Solid one-way slabs</td><td>1/20</td><td>1/24</td><td>1/28</td><td>1/10</td></tr>
<tr><td>Beams or ribbed one-way slabs</td><td>1/16</td><td>1/18.5</td><td>1/21</td><td>1/8</td></tr>
</table>

3.3.7 Tolerable crack widths for reinforced concrete

Exposure condition	Tolerable crack width in	Tolerable crack width mm
Dry air or protective membrane	0.016	0.41
Humidity, moist air, soil	0.012	0.30
Deicing chemicals	0.007	0.18
Seawater and seawater spray; wetting and drying	0.006	0.15
Water-retaining structures, excluding nonpressure pipes	0.004	0.10

3.3.8 Slab

(i) One way slab : $\frac{Length}{width} > 2$

(ii) Two way slab : $\frac{Length}{width} \leq 2$

(i) One way slab:

Slab may be supported on two opposite sides only in which case the structural action of the slab is essentially one way, the loads being carried by the slab in the direction perpendicular to the supporting beams or wall. There may be beams on all four sides. If the ratio of length to width on one slab panel is larger than about 2, most of the load is carried in the short direction to the supporting beams or walls and one-way action is obtained in effect, even through supports are provided on all sides.

The minimum thickness, t of non prestressed slabs of normal weight concrete (w_c = 145 pcf) using Grade 60 reinforcement is shown in Table 3.13, provided that the slab is not supporting or attached to construction that is likely to be damaged by large deflections.

Table 3.13

Minimum thickness t of nonprestressed one-way slabs

simply supported	1/20
One end continuous	1/24
Both ends continuous	1/28
Cantilever	1/10

Lesser thickness may be used if calculation of deflections indicates no adverse effects. For concretes having unit weight w_c in the range from 90 to 120 pcf, the tabulated values should be multiplied by $(1.65 - 0.005w_c)$, but not less than 1.09. For reinforcement having a yield stress fy other than 60,000 psi, the tabulated values should be multiplied by (0.40 + fy/100,000).

Shear will seldom control the design of one-way slabs, particularly if low tensile steel ratios are used. It will be found that the shear capacity of the concrete, ΦVc, will almost without exception be well above the required shear strength Vu at factored loads.

The total slab thickness t is usually rounded to the next higher ¼" for slabs upto 6" thickness and to the next higher ½" for thicker slabs. The concrete protection below the reinforcement should be ¾" below the bottom of the steel. In a typical slab, 1" below the centre of the steel may be assumed. The lateral spacing of the bars, except those used only to control shrinkage and temperature cracks, should not exceed 3 times the thickness t or 18", whichever is less. Generally, bar size should be selected so that the actual spacing is not less than about 1.5 times the slab thickness, to avoid excessive cost for bar fabrication and handling.

(ii) Two-way slabs

The minimum thickness t of two-way slab is:

t = perimeter/180

perimeter should be calculated using panel dimensions

Temperature and shrinkage Reinforcement:

Table 3.14

Minimum ratios of temperature and shrinkage reinforcement in slabs

Slabs where Grade 40 or 50 deformed bars are used	0.0020
Slabs where Grade 60 deformed bars or welded wire fabric (smooth or deformed) are used	0.0018
Slabs where reinforcement with yield strength exceeding 60,000 psi measured at yield strain of 0.35 percent is used.	$\frac{0.0018 \times 60,000}{f_y}$

Reinforcement for shrinkage and temperature stresses normal to the principal reinforcement should be provided in a structural slab in which the principal reinforcement extends in one direction only. The minimum ratios of reinforcement area to gross concrete area are shown in Table 3.14 but in no case shall such reinforcing bars be placed further apart than 5 times the slab thickness or more than 18 inch. In no case, the steel ratio to be less than 0.0014.

The steel required for shrinkage and temperature crack control also represents the minimum permissible reinforcement in the span direction of one-way slabs. The usual minimum ratio $200/f_y$ does not apply.

3.3.9 Design of web reinforcement

The design of beams for shear is based on the relation:

$$V_u \leq \Phi V_n$$

Where,

V_u = Total shear force applied at a given section of the beam due to factored loads.

V_n = Nominal shear strength
= $V_c + V_s$

V_c = Nominal shear strength provided by concrete.

V_s = Nominal shear strength provided by reinforcement.

Φ = Strength reduction factor = 0.85 for shear.

For members subject to shear and flexure only, the concrete contribution to shear strength is

$$V_c = 2\sqrt{f'_c}\,b_w d$$

Where,

b_w = web width in inch

d = distance from extreme compression fibre to centroid of longitudinal tension reinforcement.

f'_c = specified compressive strength of concrete.

If V_u, the shear force at factored loads, is no larger than ϕV_c then theoretically no web reinforcement is required. Even in such a case, however, ACI Code requires provision of at least a minimum area of web reinforcement equal to

$$A_v = \frac{50 b_w s}{f_y}$$

Where,

A_v = total cross-sectional area of web steel within distance s, in²
f_y = yield strength of web steel, psi
s = longitudinal spacing of web reinforcement, inch

This provision holds unless V_u is only half or less of the design shear strength ϕV_c provided by the concrete. specific exceptions to this requirement for minimum web steel are made for slabs and footings, for concrete joist floor construction, and for beams with total depth not greater than 10 inch, 2.5 times the thickness of the flange, or one half the web width (whichever is greatest).

Spacing of Web Reinforcement:

Required spacing of web reinforcement is,

For Vertical stirrup, $$s=\frac{\phi A_v f_y d}{V_u-\phi V_c}$$

For inclined stirrups or bent bars,

$$s=\frac{\phi A_v f_y d(\sin\alpha+\cos\alpha)}{V_u-\phi V_c}$$

Where,

f_y = specified yield strength of reinforcement
α = inclination of the bars with respect to the beam axis

For usual case of vertical stirrups, with $V_s \le 4\sqrt{f'_c}b_w d$, the maximum spacing of stirrups is the smallest of

$$s_{max} = \frac{A_v f_y}{50 b_w} \quad \dots\dots (1)$$

$$s_{max} = d/2 \quad \dots\dots (2)$$

$$s_{max} = 24 \text{ inch} \quad \dots\dots (3)$$

For longitudinal bars bent at an angle 45° , equation (2) is replaced by $s_{max} = 3d/4$. For $V_s > 4\sqrt{f'_c}b_w d$, these maximum spacings are halved.

To avoid excessive crack width in beam webs, the ACI Code limits the yield strength of the reinforcement to $f_y = 60,000$ psi or less. But in no case V_s to exceed $8\sqrt{f'_c}b_w d$ regardless of the amount of web steel used.

References:

1) Nilson Arthur H., Winter George, Design of Concrete Structures, Eleventh edition.

2) Building Code Requirements for Reinforced concrete (ACI 318-89) (Revised 1992) and Commentary - ACI 318R-89 (Revised 1992)

4.0 PRESTRESSED CONCRETE

Reinforced concrete in which internal stresses have been introduced to reduce potential tensile stresses in concrete resulting from loads. In reinforced concrete members the prestress is commonly introduced by tensioning the steel reinforcement.

Pretensioning : Method of prestressing in which tendons are tensioned before concrete is placed.

Post tensioning : Method of prestressing in which tendons are tensioned after concrete has hardened.

Tendon : Steel element such as wire, cable, bar, rod or strand, or a bundle of such elements, used to impart prestress to concrete.

Transfer : Act of transferring stress in prestressing tendons from jacks or pretensioning bed to concrete member.

Anchorage : In pretensioning, a device used to anchor tendon during hardening of concrete. In post tensioning, a device used to anchor tendon to concrete member.

Bonded Tendon : Prestressing tendon that is bonded to concrete either directly or through grouting.

Effective prestress : Stress remaining in prestressing tendons after all losses have occurred, excluding effects of dead load and superimposed load.

Jacking force : In prestressed concrete, temporary force exerted by device that introduces tension into prestressing tendons.

4.1 Design Code and Practice

The AASHTO specifications cover the design of pretensioned and post tensioned structures which requires that all critical sections be designed for ultimate strength and allowable stresses. These design specifications are generally intended for use with concrete strengths of 5000-6000 psi (34.5 N/mm^2 - 41.4 N/mm^2). It is reasonable, however, to use the same specifications for higher strength concrete if caution is exercised in design, and quality control in the fabrication of prestressed beams.

Prestress losses for pretensioned precast beams are covered in AASHTO Article 1.6.7(B)(1) and (2). For normal design it is proper to use the lump-sum losses provided in (2) when the design concrete strengths are within 500 psi, (±), of the 5000 psi strength given For strengths greater than $f'_c = 5500$ psi the losses should be computed by formulae for the individual loss sources given in (1). It is proper to subtract the elastic shortening loss from the initial prestress force when computing initial stresses. When using the lump sum losses a reasonable approximation of the elastic shortening loss is 0.07 X 270 ksi = 18.9 ksi.

Strength at transfer of prestress usually range from 4000 to 5000 psi. 1/2" diameter seven- wire, 270^k strand is commonly used today as the major prestressing reinforcement. In many areas strands are bundled near the midspan area of beams to obtain a greater eccentricity.

The normal design procedure for precast /prestressed concrete is to design for allowable working stresses and to check initial stresses and ultimate moment capacity. Most designs for this type of section utilize pretensioned prestressing in which the strands are stressed before the concrete is placed around the strands. When the concrete has cured to sufficient strength the strands are cut and the prestressing is transferred to the beams. Some post-tensioning is used on occasion with pre-cast sections. The ducts are cut in the section and strands are installed after concrete placement and curing. After the concrete cures, the strands are jacked, anchored and grouted.

4.2 Materials

4.2.1 Concrete

In the present practice, compressive strength, f'_c (28 day cylinder strength) between 4000 to 6000 psi (28 to 41 N/mm²) is commonly specified.

AASHTO design specifications are generally intended for use with concrete strengths of 5000-6000 psi. It is reasonable, however, to use the same specifications for higher strength concrete if caution is exercised in design and quality control in the fabrication of prestressed beam.

Strength at transfer of prestress usually range from 4000 to 5000 psi (28 to 34 N/mm²).

4.2.2 Steel for Prestressing

High tensile steel for prestressing usually takes one of three forms: wires, strands, or bars.

For post tensioning, wires are widely employed. They are grouped, in parallel, into cables. Strands are fabricated in the factory by twisting wires together, thus decreasing the number of units to be handled in the tensioning operations. Strands, as well as high tensile roads are also used for post tensioning.

For pretensioning, 7 wires strands are almost exclusively used in the USA and in other countries. Although strands cost slightly more than wires of the same tensile strength its better bonding characteristics make it specially suitable for pretensioning.

4.2.2.1 Steel Wires

Wires for prestressing generally conform to ASTM specification A-421 for "Uncoated stress-relieved wire for prestressed concrete". The tensile strength and the minimum yield strength (measured by the 1.0% total elongation method) are shown in Table 4.1 for the common sizes of wires.

Table 4.1

Properties of Uncoated Stress-Relieved Wire (ASTM A 421)

Nominal Diameter in. (mm)	Minimum Tensile Strength psi (N/mm²)		Minimum Stress at 1% Extension (N/mm²)	
	Type BA[b]	Type WA	Type BA[b]	Type WA
0.192 (4.88)	[a]	250,000 (1725)	[a]	200,000 (1380)
0.196 (4.98)	240,000 (1655)	250,000 (1725)	192,000 (1325)	200,000 (1380)
0.250 (6.35)	240,000 (1655)	240,000 (1655)	192,000 (1325)	192,000 (1325)
0.276 (7.01)	[a]	235,000 (1622)	[a]	188,000 (1295)

[a] These sizes are not commonly furnished in Type BA wire.

[b] Type BA wire is used for applications in which cold-end deformation is used for anchoring purposes (button anchorage), and type WA is used for applications in which the ends are anchored by wedges and no cold-end deformation of the wire is involved (wedge anchorage). Examples of tendons with button anchorages, more common in United States practice.

4.2.2.2 Steel Strands

Strands for prestressing generally conform to ASTM specification A-416 for "Uncoated seven-wire stress relieved for prestressed concrete". Two grades are available, 250 ksi and 270 ksi (1724 N/mm2 and 1862 N/mm2), where the grades indicates minimum guaranteed breaking stress. While these specifications were intended for pretensioned, bonded, prestressed concrete construction, they are also applicable for post tensioned construction, whether of the bonded or the unbonded type. These seven wire strands all have a centre wire slightly larger than the six wires which enclose it tightly in a helix with a uniform pitch between 12 and 16 times the nominal diameter of the strand.

** 1/2" diameter seven wire, 270[k] strand is commonly used today as the major prestressing reinforcement (both for pretensioned and post tensioned structures).

Table 4.2

Properties of Uncoated Seven-Wire Stress-Relieved Strand (ASTM A-416)

Nominal Diameter in. (mm)	Breaking Strength lb (kN)	Nominal Area of Strand in2. (mm2)	Minimum Load at 1% Extension lb (kN)
Grade 250			
0.250 (6.35)	9000 (40.0)	0.036 (23.22)	7650 (34.0)
0.313 (7.94)	14,500 (64.5)	0.058 (37.42)	12,3000 (54.7)
0.375 (9.53)	20,000 (89.0)	0.080 (51.61)	17,000 (75.6)
0.438 (11.11)	27,000 (120.1)	0.108 (69.68)	23,000 (102.3)
0.500 (12.70)	36,000 (160.1)	0.144 (92.90)	30,600 (136.2)
0.600 (15.24)	54,000 (240.2)	0.216 (139.35)	45,900 (204.2)
Grade 270			
0.375 (9.53)	23,000 (102.3)	0.085 (54.84)	19,550 (87.0)
0.438 (11.11)	31,000 (137.9)	0.115 (74.19)	26,350 (117.2)
0.500 (12.70)	41,300 (183.7)	0.153 (98.17)	35,100 (156.1)
0.600 (15.24)	58,600 (260.7)	0.217 (140.00)	49,800 (221.5)

4.2.2.3 Steel Bars

Alloy steel bars are available in diameters ranging from 1/2" to 1 3/8" as shown Table 4.3 and in two grades, Grade 145 and Grade 160 corresponding to the minimum ultimate strengths of 145,000 and 160,000 psi (1000 and 1100 N/mm^2) respectively.

Table 4.3

Properties of Alloy Steel Bars

Nominal Diameter in. (mm)	Nominal Area of Bar in^2. (mm^2)	Breaking Strength lb (kN)	Minimum Load at 0.7% Extension lb/kN
Grade 145			
½ (12.70)	0.196 (127)	28,000 (125)	25,000 (111)
⅝ (15.88)	0.307 (198)	45,000 (200)	40,000 (178)
¾ (19.05)	0.442 (285)	64,000 (285)	58,000 (258)
⅞ (22.23)	0.601 (388)	87,000 (387)	78,000 (347)
1 (25.40)	0.785 9507)	114,000 (507)	102,000 (454)
1⅛ (28.58)	0.994 (642)	144,000 (641)	129,000 (574)
1¼ (31.75)	1.227 (792)	178,000 (792)	160,000 (712)
1⅜ (34.93)	1.485 (958)	215,000 (957)	193,000 (859)
Grade 160			
½ (12.70)	0.196 (127)	31,000 (138)	27,000 (120)
⅝ (15.88)	0.307 (198)	49,000 (218)	43,000 (191)
¾ (19.05)	0.442 (285)	71,000 (316)	62,000 (276)
⅞ (22.23)	0.601 (388)	96,000 (427)	84,000 (374)
1 (25.40)	0.785 (507)	126,000 (561)	110,000 (490)
1⅛ (28.58)	0.994 (642)	159,000 (708)	139,000 (619)
1¼ (31.75)	1.227 (792)	196,000 (872)	172,000 (765)
1⅜ (34.93)	1.485 (958)	238,000 (1059)	208,000 (926)

4.3 Permissible Stresses for Flexural Members (ACI code)

4.3.1 Steel stresses - not more than the following values:

a. Due to tendon jacking force, $0.80f_{pu}$ or $0.94\ f_{py}$, whichever is smaller, but not greater than maximum value recommended by manufacturer of prestressing tendons or anchorages.

Here, f_{pu} = Specified tensile strength of prestressing tendons, psi

f_{py} = Specified yield strength of prestressing tendons, psi.

b. Pretensioned tendons immediately after transfer of prestress or post tensioned tendons after anchorage, $0.70f_{py}$

4.3.2 Concrete stresses - not more than the following values:

a. Immediately after transfer of prestress (before losses), extreme fibre stress:

Compression - 0.60 f'_c
Tension - 3 $\sqrt{f'_c}$ (except at ends of simply supported members where $6\sqrt{f'_c}$ is permitted)

b. At service load after allowance for all prestress losses:

Compression - 0.45 f'_c
* Tension - 6 $\sqrt{f'_c}$

* When analysis based on cracked sections and bilinear moment deflection relationships show that immediate and long time deflections satisfy code limits, maximum tendon is 12 $\sqrt{f'_c}$.

4.4 Ultimate load

Structures designed on the basis of working stresses may not always possess a sufficient margin for over loads. Since it is required that a structure possess a certain minimum factored load capacity , it is necessary to determine its ultimate strength. In general, the ultimate strength of structure is defined by the maximum load it can carry before collapsing. Ultimate strength is commonly accepted as a criterion for design in prestressed concrete as with other structural systems.

Table - 4.4

Load factors and Required strength under ACI Code

Summarizes the basic ACI Code strength requirements:

ACI Code load factors

$U = 1.4D + 1.7L$ (ACI 9 - 1) Basic Required Strength

where wind is included, see if one of the following is greater.

$U = 0.75(1.4D + 1.7L + 1.7W)$ (ACI 9 -2)

or

$U = 0.9D + 1.3W$ (ACI 9 -3)

Where, U = Required strength
D = Dead load
L = Live load
W = Wind load.

Required strength	$\leq$	Design Strength of Member
Required strength	$M_u \leq \Phi M_n$	Member design strength is the
from factored loads	$P_u \leq \Phi P_n$	strength reduction factor, Φ,
	$V_u \leq \Phi V_n$	times the best estimate of member strength (nominal strength)

$\Phi = 0.90$ - flexure, M
$\Phi = 0.85$ - shear, V
$\Phi = 0.75$ - spiral column, P

4.5 Loss of Prestress

To determine effective prestress f_{se}, allowance for the following sources of loss of prestress shall be considered:

(a) Anchorage seating loss
(b) Elastic shortening of concrete
(c) Creep of concrete
(d) Shrinkage of concrete
(e) Relaxation of tendon stress.
(f) Friction loss due to intended or curvature in post-tensioning tendons.

4.5.1 Anchorage Seating Loss

Loss of Prestress due to Anchorage Take-up and anchorage deformation Δ_a is

$$ANC = \frac{\Delta_a E_s}{L}$$

Where,

ANC = Loss of prestress due to anchorage of prestressing steel

Δ_a = anchorage deformation

Es = Modulus of elasticity of steel.

L = Tendon length.

4.5.2 Loss of Prestress due to Elastic Shortening of Concrete

Loss of prestress in the steel due to elastic shortening of concrete is:

$$ES = K_{es} . E_s . \frac{f_{cir}}{E_{ci}}$$

Where,

ES = Loss of prestress in the steel due to elastic shortening of concrete.

K_{es} = 1.00 for pre-tensioned member.

K_{es} = 0.5 for post-tensioned member when tendons are in sequential order to the same tension.

f_{cir} = stress in the concrete at c.g.s (i.e. at the level of the steel centroid) due to prestress force Fo which is effective immediately after prestress has been applied to concrete.

F_o = 0.9 F_1 (Pre-tensioned member).

F_1 = Initial Prestress force.

$$f_{cir} = \frac{F_o}{A} + \frac{F_o . e^2}{I} - \frac{M_G . e}{I}$$

Where,

A = Area of gross concrete section.
e = eccentricity of steel centroid with respect to concrete centroid.
I = Moment of Inertia of concrete section.
M = Moment due to self wt. of the member.

4.5.3 Loss of prestress due to creep of concrete (CR)

Loss of prestress due to creep of concrete is computed for bonded members from the following expression (for normal weight concrete):

$$CR = K_{cr} \frac{E_s}{E_c} (f_{cir} - f_{cds})$$

where,

K_{cr} = 2.00 for pre-tensioned members.

K_{cr} = 1.60 for post-tensioned members.

E_s = Modulus of elasticity of prestressing tendons.

E_c = Modulus of elasticity of concrete at 28 days corresponding to f'_c.

f_{cir} = The concrete stress at the level of steel centroid (c.g.s) immediately after transfer of prestress.

f_{cds} = Stress in concrete at c.g.s (i.e. at steel centroid) of tendons due to all super-imposed dead loads that are applied to the member after it is prestressed.

For unbonded tendons the average compressive stress is used to evaluate losses due to elastic shortening and creep of concrete losses. The losses in the unbonded tendon are related to the average member strain rather than strain at the point of maximum moment.

Thus,

$$CR = K_{cr} \cdot \frac{E_s}{E_c} \cdot f_{cpa}$$

Where,

f_{cpa} = average compressive stress in the concrete along the member length at the c.g.s of the tendon.

4.5.4 Loss of Prestress due to shrinkage of Concrete

The loss of prestress due to shrinkage is the product of the effective shrinkage ϵ_{sh} and the modulus of elasticity of prestressing steel.

The loss of prestress due to shrinkage of concrete is:

$$SH = K_{sh} \cdot \epsilon_{sh} \cdot E_s$$
$$= K_{sh} \times 8.2 \times 10^{-6} (1 - 0.06V/S) (100 - RH).Es$$

Where,

K_{sh} = coefficient.

K_{sh} = 1.00 for pretensioned members

V/S = volume to surface ratio

RH = relative humidity.

Es = modulus of elasticity of prestressing steel.

Values of K_{sh} for post-tensioned member

Time after end of moist curing to application of prestress

Days	1	3	5	7	10	20	30	60
Ksh	0.92	0.85	0.80	0.77	0.73	0.64	0.58	0.45

4.5.5 Loss of prestress due to Relaxation of Tendon Stress

Loss of steel stress resulting from relaxation

$$RE = f_{pi}\,\frac{\log t}{10}\left(\frac{f_{pi}}{f_{py}} - 0.55\right)$$

Where,

f_{pi} = initial stress

f_{py} = effective yield stress

t = time in hours after stressing

logt is to the base 10 and f_{pi}/f_{py} exceeds 0.55

For step-by-step loss analysis, the loss increment in any time interval resulting from steel relaxation :

$$RE = f_{pi}\left(\frac{\log t - \log t1}{10}\right)\left(\frac{f_{pi}}{f_{py}} - 0.55\right)$$

Where,

t1 = the time at the beginning of the interval.
t = time at the end of the interval.

The same equation is useful in estimating relaxation loss for pretensioned members for which the relaxation loss takes place before the concrete is cast should be subtracted form the total relaxation loss to obtain the change in steel stress as the beam ages from the initial to the final condition.

Relaxation loss will be diminished because of the effects of concrete shrinkage and creep, which reduce the stress intensity in the steel. This interaction may be accounted for in an approximate way be substituting $0.90f_{pi}$ for f_{pi} in the above equations.

Alternatively,

According to ACI - ASCE Committee loss due to relaxation of steel:

$$RE = [K_{re} - J(SH + CR + ES)]C$$

Where,

SH = loss due shrinkage of concrete.
CR = loss die to creep of concrete.
ES = loss due to Elastic shortening of concrete.

Values of K_{re}, J and C are given in Table 4.5 and Table 4.6

Table 4.5

Values of K_{re} and J

Type of Tendon *	K_{re}	J
270 Grade stress-relieved strand or wire	20,000	0.150
250 Grade stress-relieved strand or wire	18,500	0.140
240 or 235 Grade stress-relieved wire	17,600	0.130
270 grade low-relaxation strand	5,000	0.040
250 Grade low-relaxation wire	4,630	0.037
240 or 235 Grade low-relaxation wire	4,400	0.035
145 or 160 Grade stress-relieved bar	6,000	0.050

* In accordance with ASTM A416-74, ASTM A421-76, or ASTM A722-75

Table 4.6

Values of C

$f_{pi}f_{pu}$	Stress relieved strand or wire	Stress-relieved bar or low-relaxation strand or wire
0.80		1.28
0.79		1.22
0.78		1.16
0.77		1.11
0.76		1.05
0.75		1.00
0.74		0.95
0.73		0.90
0.72		0.85
0.71		0.80
0.70	1.45	0.75
0.69	1.36	0.70
0.68	1.27	0.66
0.67	1.18	0.61
0.66	1.09	0.57
0.65	1.00	0.53
0.64	0.94	0.49
0.63	0.89	0.45
0.62	0.83	0.41
0.61	0.78	0.37
0.60	0.73	0.33
	0.68	
	0.63	
	0.58	
	0.53	
	0.49	

4.5.6 Loss of Friction due to intended or curvature in post-tensioning tendons

Effect of friction loss in post-tensioning tendons shall be computed by:

$$P_s = P_x e^{(klx + \mu\alpha)}$$

When $(klx + \mu\alpha)$ is not greater than 0.3. effect of friction loss may be computed by :

$$P_s = P_x \ (1 + kl_x + \mu\alpha)$$

Where,

P_s = Prestressing tendon force at jacking end
P_x = Prestressing tendon force at any point X
k = Wobble friction coefficient per foot prestressing tendon
l_x = Length of prestressing tendon element from jacking end to any point X ft
μ = Curvature friction coefficient

α = Total angular change of prestressing tendon profile in radius from tendon jacking end to any point X

Friction loss shall be based on experimentally determined Wobble K and curvature μ friction coefficients and shall be verified during tendon stressing operations.

Friction coefficients for post-Tensioned Tendons

			Wobble coeff. K per foot	Curvature coeff. μ
Grouted tendons in metal Sheathing		Wire tendons	0.0010 - 0.0015	0.15 - 0.25
		High strength bars	0.0001 - 0.0005	0.08 - 0.30
		7-wire strand	0.0005 - 0.0020	0.15 - 0.25
Unbonded tendons	Mastic coated	Wire tendons	0.0010 - 0.0020	0.05 - 0.15
		7-wire strand	0.0010 - 0.0020	0.05 - 0.15
	Pre-greased	Wire tendons	0.0003 - 0.0020	0.05 - 0.15
		7-wire strand	0.0003 - 0.0020	0.05 - 0.15

4.6 Shear Design Criteria

[a] Design Basis

The design of cross sections subject to shear is to be based on the relation

$$V_u \leq \Phi V_n \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

Where,

V_u = factored shear force at section.
V_n = nominal shear strength of the section.
Φ = strength reduction factor taken equal to 0.85 for shear.

The nominal shear strength, V_n is calculated from the equation

$$V_n = V_c + V_s \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2)$$

Where,

V_c = nominal shear strength provided by concrete.
V_s = nominal shear strength provided by shear reinforcement.

The value of V_c is to be calculated according to section below.

The first critical section for shear is assumed to be at a distance h/2 from the face of a support and sections located less than a distance h/2 from the face of support may be designed for the same shear V_u as that computed at a distance h/2 (h = overall thickness of member i.e. total beam depth). This provision recognizes, the beneficial effect of vertical compression in the concrete caused by the reaction. In special circumstances, those benefits do not obtain and the shear at the support face may become critical.

[b] *Nominal shear strength provided by concrete:*

The value of V_c in equation (2) is to be taken equal to smaller of V_{ci} and V_{cw}, determined by flexure shear cracking respectively.

V_{ci} = nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment.

V_{cw} = nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in web.

$$V_{ci} = 0.6\sqrt{f'_c}\,b_w d + V_d + \frac{V_i M_{cr}}{M_{max}} \qquad \ldots\ldots\ldots (3)$$

but V_{ci} need not be taken less than $1.7\sqrt{f'_c}b_w d$

where,

b_w = web width, or diameter of circular section.

d = distance from extreme compression fibre to centroid of prestressed reinforcement or 0.80h whichever is greater.

V_d = shear force at section due to unfactored dead load.

V_i = factored shear force at section due to externally applied loads (dead load and live load) occurring simultaneously with Mmax.

M_{max} = Maximum factored moment at section due to externally applied loads.

M_{cr} = Moment causing flexural cracking at section due to externally applied loads.

$$M_{cr} = \frac{I}{Y_t}(6\sqrt{f'_c} + f_{pe} - f_d)$$

where,

I = moment of inertia of section resisting externally applied factored loads.

Y_t = distance from centroidal axis of gross section, neglecting reinforcement, to extreme fibre in tension.

f_{pe} = Compressive stress in concrete due to effective prestress forces only (after allowance for all prestress losses) at extreme fibre of section where tensile stress is caused by externally applied loads, psi.

f_d = stress due to unfactored dead load at extreme fibre of section where tensile stress is caused by externally applied loads, psi.

Values of M_{max} and V_i shall be computed from the load combination causing maximum moment to occur at the section.

The nominal shear strength corresponding to web-shear cracking is computed from the equation:

$$V_{cw} = (3.5\sqrt{f'_c} + 0.3f_{pc})b_w d + V_p \quad \ldots\ldots\ldots\ldots (4)$$

Where,

f_{pc} = compressive stress in concrete (after allowance for all prestress losses) at centroid of cross section resisting externally applied loads or at junction of web and flange when the centroid lies within the flange, psi. (In a composite member, f_{pc} is resultant compressive stress at centroid of composite section or at junction of web and flange when the centroid lies within the flange, due to both prestress and moments resisted by precast member acting alone).

d = distance from extreme compression fibre to centroid of prestressed reinforcement or 0.8h whichever is greater.

V_p = vertical component of the effective prestress force at the section.

= $P_e.\sin\theta$

P_e = effective prestress force at the section.

θ = slope of the tendon centroid line at the section.

Alternatively, Vcw may be computed as the shear force corresponding to dead load plus live load that results in a principal tensile stress of $4\sqrt{f'_c}$ at the centroidal axis of the member, or at the intersection of flange and web when the centroidal axis is in the flange. In composite members, the principal tensile stress shall be computed using the cross section that resists live load.

For members with an effective prestress force not less than 40 percent of the tensile strength of the flexural reinforcement, an alternative to the use of equation (3) and (4) is permitted by ACI code. The shear strength Vc provided by concrete may be computed by

$$V_c = (0.6\sqrt{f'c} + \frac{700 V_u d}{M_u}) b_w d \quad \ldots\ldots\ldots (5)$$

but V_c need not be taken less than $2\sqrt{f'_c}b_w d$ and must not be taken greater than $5\sqrt{f'_c}b_wd$.

In this equation

V_u = factored shear force at section due to all loads.

M_u = factored moment at section due to all loads.

The quantity V_ud/M_u shall not be taken greater than 1.0. In this equation

d = distance from extreme compression fibre to the centroid of prestressed reinforcement; the lower bound of d = 0.8h used elsewhere does not apply here.

Equation (5) is appealing in that it is simple to use compared with the more accurate equation (3) and (4), but it may give very conservative and uneconomical results for certain classes of members.

[c] Required Area of web Reinforcement

When shear reinforcement perpendicular to the axis of the member is used, its contribution to shear strength is

$$\frac{A_v f_y d}{s}$$

but the value of Vs is not to be taken greater than $8\sqrt{f'c}bwd$.

Where,

A_v = area of shear reinforcement within a distance s.

The total nominal shear strength V_n is found by summing the contributions of the steel and the concrete;

$$V_n = \frac{A_v f_y d}{s} + V_c$$

From equation (1), in the limiting case and equation (2);

$$V_u = \Phi V_n$$

$$= \Phi (V_s + V_c)$$

$$= \Phi \left(\frac{A_v f_y d}{s} + s \right)$$

Hence,

$$A_v = \frac{(V_u - \Phi V_c) s}{\Phi f_y d}$$

Hence stirrup spacing,

$$s = \frac{\Phi A_v f_y d}{V_u - \Phi V_c} \quad \text{.....................} \quad (6)$$

If the spacing determined for the trial stirrup size is too close for placement, economy or practically, or if it is so large that maximum spacing requirements control over too great a part of the beam span, then a revised bar size is selected and the calculation repeated.

[d] *Minimum web Reinforcement*

At least a certain minimum area of shear reinforcement is to be provided in all reinforced concrete flexural members (prestressed and nonprestressed) where factored shear force V_u exceeds one half the shear strength ΦV_c provided by the concrete except:

1. Slabs and footings
2. Concrete joist construction (including ribbed members such as double tee beams)
3. Beams with a total depth not greater than the largest of 10", 2½ times the thickness of the flange and 1/2 the web width.

The minimum area of shear reinforcement to be provided in all other cases is to be taken equal to the smaller of the following values:

$$A_v = 50\ b_w s/f_y \quad \text{... ... .} \quad (7)$$

$$A_v = \frac{A_{ps}}{80} \cdot \frac{f_{pu}}{f_y} \cdot \frac{s}{d}\sqrt{\frac{d}{b_w}} \quad \text{.....} \quad (8)$$

where,

A_{ps}	=	area of prestressed reinforcement in tension zone, in^2.
b_w	=	web width or diameter of circular section, inch.
s	=	spacing of shear reinforcement in direction parallel to the longitudinal reinforcement, inch.
f_y	=	yield stress of stirrup steel.
f_{pu}	=	specified tensile strength of prestressing tendons, psi. (i.e. ultimate tensile strength).
d	=	distance from extreme compression fibre to centroid of longitudinal tension reinforcement, but need not be less than 0.8h for prestressed members, inch.

Equation (7) will generally require greater minimum web steel than equation (8). Thus equation (8) generally controls. However, equation (8) may be applied only if the effective prestress force is not less than 40 percent of the tensile strength of the flexural reinforcement.

In addition, certain restrictions are imposed on the maximum spacing of web reinforcement by ACI code to ensure that any potential diagonal crack will be crossed by at least a minimum amount of web steel. For prestressed members this maximum spacing of shear reinforcement placed perpendicular to the axis of the member shall not exceed the smaller of;

3/4 h

24 inch.

If the value of Vs exceeds $4\sqrt{f'_c}b_wd$ these limits (3/4h, 24 inch) are reduced by one half.

Contribution of shear reinforcement to shear strength V_s shall not be taken greater than $8\sqrt{f'_c}b_wd$.

Design yield strength of shear reinforcement shall not exceed 60,000 psi. Limiting the design yield strength of shear reinforcement to 60,000 psi provides a control on diagonal crack width.

4.7 Anchorage Zone Design

In prestressed Concrete beams, the prestressing force is introduced as a load concentration, often acting over a relatively small fraction of the total member depth.

For post-tensioned beams with mechanical anchorages, the load is applied at the end face, while for pre-tensioned beams it is introduced somewhat more gradually over the transfer length.

Pre-tensioned members:

The total stirrup tension S is expressed in terms of the total longitudinal prestress force P by the relation;

$$\frac{S}{P} = 0.0106\frac{h}{l_t}$$

where;

h	=	total beam depth
l_t	=	transfer length

The stirrup stress varies approximately linearly from maximum close to the end face to zero near the end of the crack. Thus, if f_s is a permissible stirrup stress when the initial prestress force P_i is applied, the average stress in the stirrups can be taken as $f_s/2$ and the total cross sectional area of stirrups necessary, A_t, is

$$\frac{A_t f_s}{2P_i} = 0.0106\frac{h}{l_t}$$

$$\text{or} \quad A_t = 0.021\frac{P_i h}{f_s l_t} \qquad \text{(a)}$$

An allowable stress f_s = 20,000 psi has been found in tests to produce acceptably small crack widths.

The transfer length l_t from the end of the member equal to ;

$$l_t = (f_{pe}/3)\ d_b$$

where;

l_t is given in inches.

d_b = nominal strand diameter in inches.
f_{pe} = effective prestress in ksi.

Alternatively l_t may be assumed to equal 50 times the nominal diameter of the strand.

The required reinforcement having total area A_t should be distributed uniformly over a length equal to h/5 measured from the end face of the beam, and for most efficient crack control the first stirrup should be placed as close to the end face as possible.

It is recommended that vertical reinforcement according to equation (a) be provided for all pretentioned members unless tests or experience indicate that cracking does not occur at service or overload stages.

Post-tensioned members:

Equation (a) which is experimentally derived, does not apply to post-tensioned members, for which the prestressing force is applied at or near the end face rather than by bond stress over a transfer length. For post tensioned members, Gergely and Sozen have developed a method based on equilibrium conditions after cracked anchorage zone, with the objective of linking the length and width of the horizontal crack.

Knowing the maximum bending moment to be resisted, the forces T and C can be calculated if the distances between those forces can be estimated.

Where:

T = Tensile force from the end zone reinforcement ,
C = Compressive resultant force from the concrete.

For post-tensioned beams, stirrups should be provided within a distance h/2 from the end face to resist T, and the centre of gravity of those stirrup forces is easily found. The location of C must be estimated. It is usually assumed to be at a distance h from the end face. Accordingly, the tensile force to be resisted by the end stirrups is

$$T = \frac{M_{max}}{h-x}$$

Where,

M_{max} = maximum bending moment to be resisted,
x = distance from the end face to the centroid of the vertical steel within the distance h/2.

The total required area of steel reinforcement is

$$A_t = \frac{T}{f_s}$$

Where:

f_s = allowable stress chosen on the basis of crack centrally value of f_s=20,000 psi has been found to be satisfactory.

In addition to vertical tensile stress causing spitting end zone distress may be caused in post-tensioned beams by the high concentration of longitudinal compression under the bearing plates of the anchorages. The bearing stress on the concrete caused by the post-tensioning anchorages when the initial force P_i acts should not exceed

$$f_{cp} = 0.6 f'_{ci} \left(\frac{A_2}{A_1}\right)^{1/3}$$

and should not exceed f'_{ci}.

Where:

A_1 = bearing area of the anchor plate,
A_2 = maximum area of the portion of the anchorage surface that is geometrically similar to and concentric with the area of the anchor plate of the post tensioning steel.

f'_{ci} = Compressive strength of the concrete at the time of initial prestress.

4.8 Section Properties

Table 4.7

Section Properties of AASHTO Bridge Girders

Type	h in.	A_c in^2	I_c in^4.	c_1 in.	c_2 in.	r_2 in^2.	w_o plf
I	28	276	22,750	15.41	12.59	82	288
II	36	369	50,979	20.17	15.83	138	384
III	45	560	125,390	24.73	20.27	224	583
IV	54	789	260,741	29.27	24.73	330	822
V	63	1013	521,180	31.04	31.96	514	1055
VI	72	1085	733,320	35.62	36.38	676	1130

4.9 DESIGN AIDS

Design aids are given in Table 4.8 to Table 4.14

Table 4.8

Properties of Prestressing Steels

Seven-Wire Strand Grade 250						
Nominal diameter, in.	1/4	5/16	3/8	7/16	1/2	0.600
Area, Ap, in^2	0.036	0.058	0.080	0.108	0.144	0.215
Weight, plf	0.120	0.200	0.27	0.370	0.490	0.74
$0.7f_{pu}Ap$, kips	6.300	10.20	14.0	18.90	25.20	37.6
$0.8f_{pu}Ap$, kips	7.200	11.60	16.00	21.60	28.8	43.0
$f_{pu}Ap$, kips	9.000	14.50	20.00	27.00	36.0	54.0

Table 4.9

Properties of Prestressing Steels

Seven-Wire Strand Grade 270				
Nominal diameter, in.	3/8	7/16	1/2	0.600
Area, Ap, in2.	0.085	0.115	0.153	0.217
Weight, plf.	0.290	0.390	0.520	0.740
$0.7f_{pu}Ap$, kips	16.10	21.70	28.90	41.00
$0.8f_{pu}Ap$, kips	18.40	24.80	33.00	46.90
$f_{pu}Ap$, kips	23.00	31.00	41.30	58.60

Table 4.10

Properties of Prestressing Steels

Round wire				
Diameter, in.	0.192	0.196	0.250	0.276
Area, Ap, in^2	0.0289	0.0302	0.0491	0.0598
Weight, plf	0.0980	0.1000	0.1700	0.2000
Ultimate strength, fpu, ksi	250.00	250.00	240.00	235.00
$0.7f_{pu}Ap$, kips	5.0500	5.2800	8.2500	9.8400
$0.8f_{pu}Ap$, kips	5.7800	6.0400	9.4200	11.240
$f_{pu}Ap$, kips	7.2200	7.5500	11.780	14.050

Table 4.11

Properties of Prestressing Steels

Round Bars Grade 145						
Nominal diameter, in.	3/4	7/8	1	1⅛	1¼	1⅜
Area, Ap, in^2	0.442	0.601	0.785	0.994	1.227	1.485
Weight, plf	1.500	2.040	2.670	3.380	4.170	5.050
$0.7f_{pu}Ap$, kips	44.90	61.00	79.70	100.9	124.5	150.7
$0.8f_{pu}Ap$, kips	51.30	69.70	91.00	115.3	142.3	172.2
$f_{pu}Ap$, kips	64.10	87.10	113.8	144.1	177.9	215.3

Table 4.12

Properties of Prestressing Steels

Round Bars Grade 160						
Nominal diameter, in.	3/4	7/8	1	1⅛	1¼	1⅜
Area, Ap, in2.	0.442	0.601	0.785	0.994	1.227	1.485
Weight, plf.	1.500	2.044	2.670	3.380	4.170	5.050
0.7fpuAp, kips.	49.50	67.30	87.90	111.3	137.4	166.3
0.8fpuAp, kips	56.60	77.00	100.5	127.2	157.0	190.1
fpuAp, kips.	70.70	96.20	125.6	159.0	196.3	237.6

Table 4.13

Properties of Prestressing Steels

Deformed Bars							
Nominal diameter, in.	⅝	⅝S	1	1	1¼	1¼	1¼
Area, Ap, in2.	0.28	0.31	0.85	0.85	1.25	1.25	1.56
Weight, plf.	0.98	1.09	3.01	3.01	4.39	4.39	5.56
Ultimate strength, f_{pu}, ksi	157	230	150	160	150	160	150
$0.7f_{pu}$Ap, kips	30.5	49.5	89.5	95.4	131.0	140.0	163.8
$0.8f_{pu}$Ap, kips	34.8	56.5	102.2	109.1	150.0	160.0	187.2
f_{pu}Ap, kips	43.5	70.5	127.8	136.3	187.5	200.0	234.0

Table 4.14

Designations, Areas, Perimeters, and Weights of Reinforcing Bars

Bar No.[a]	Diameter, (in.)	Cross-Sectional, A (in^2.)	Perimeter (in.)	Unit Weight, per foot, lb
2	¼ = 0.250	0.05	0.79	0.167
3	⅜ = 0.375	0.11	1.18	0.376
4	½ = 0.500	0.20	1.57	0.668
5	⅝ = 0.625	0.31	1.96	1.043
6	¾ = 0.750	0.44	2.36	1.502
7	⅞ = 0.875	0.60	2.75	2.044
8	1 = 1.000	0.79	3.14	2.670
9	1⅛ = 1.128[b]	1.00	3.54	3.400
10	1¼ = 1.270[b]	1.27	3.99	4.303
11	1⅜ = 1.410[b]	1.56	4.43	5.313
14	1¾ = 1.693[b]	2.25	5.32	7.650
18	2¼ = 2.257[b]	4.00	7.09	13.60

a. Based on the number of eighths of an inch included in the nominal diameter of the bars. The nominal diameter of a deformed bar is equivalent to the diameter of a plain bar having the same weight per foot as the deformed bar. Bar No. 2 is available in plain rounds only. All others are available in deformed rounds.

b. Approximate to the nearest 1/2 in.

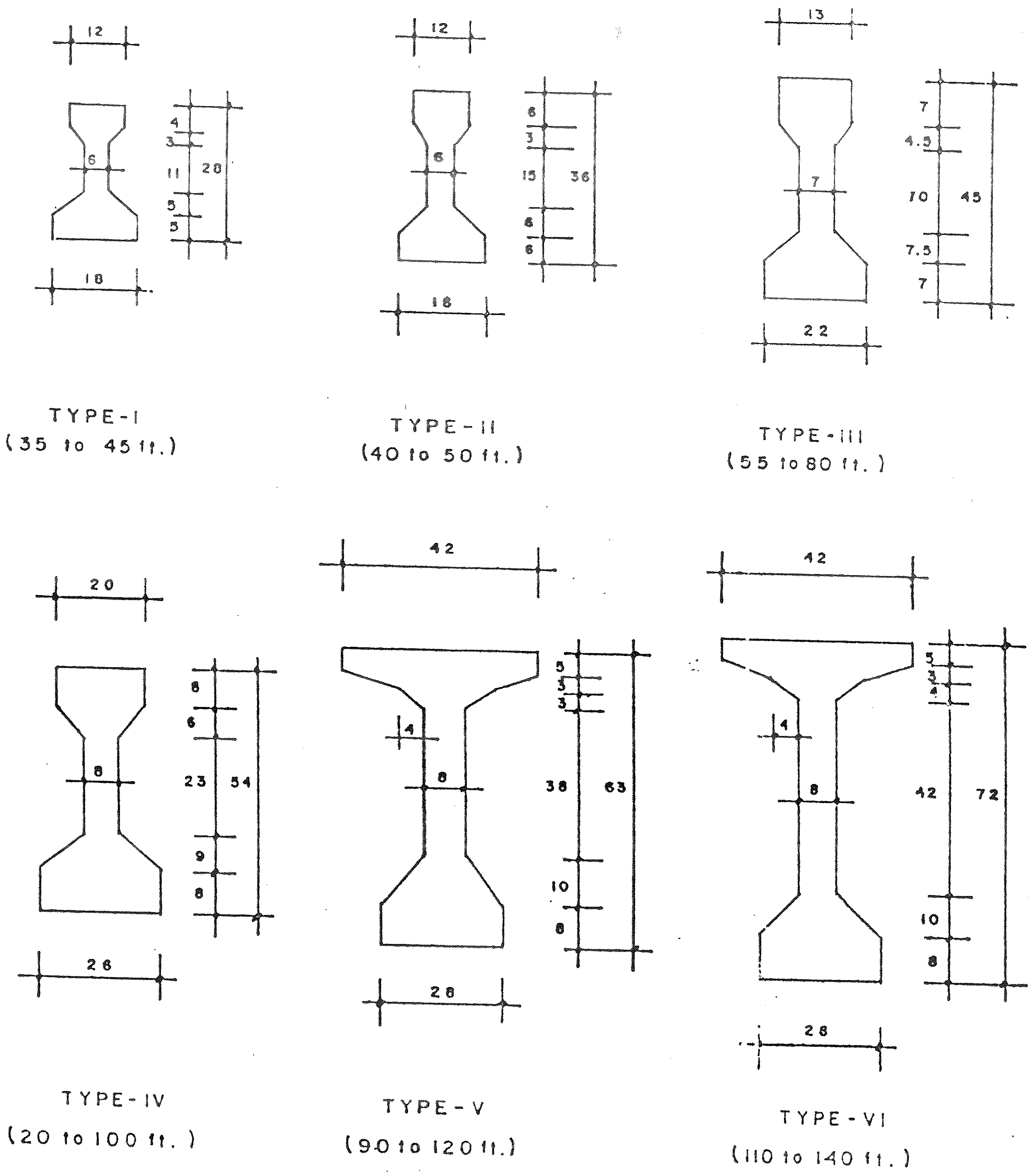
References:

1) Lin T. Y., Prestressed Concrete Structures, 3rd edition

2) Nilson Arthur H., Prestressed Concrete

3) Building Code Requirements for Reinforced concrete (ACI 318-89) (Revised 1992) and Commentary - ACI 318R-89 (Revised 1992)

Fig. 4.1



Sections of AASHTO Bridge Girders

5.0 GENERAL FOUNDATION DESIGN

5.1 Basic Requirements

The foundation design is to be based on satisfying the basic two fundamental requirements i.e. (i) factor of safety against bearing capacity failure and (ii) allowable settlement limit.

5.1.1 Factor of safety (F.S) against Bearing capacity failure

The factor of safety with respect to ultimate bearing capacity failure shall be as follows:

(a) Cohesionless soils

F.S = 2, Normal and extreme loading condition

(b) Cohesive soils

F.S = 3, Normal loading condition

F.S = 2, Normal and extreme loading (with live load)

5.1.2 Allowable Settlement

The settlement of foundations should be within the following acceptable limits.

Type of Foundation	Total Settlement	Differential settlement
Isolated footing	25 mm	19 mm
Raft or Combined footing	50 mm	25 mm
Pile	As above depending upon whether pile cap is a footing or a raft	
Well/Caisson	Not critical if factor of safety against bearing capacity failure is adequate	

5.2 Bearing Capacity

The supporting power of a soil or rock is referred to as its bearing capacity or in other words the bearing capacity is defined as the load or pressure developed under the foundation without introducing damaging movements in the foundation and in the superstructure supported on the foundation. The general equation for concentrically loaded shallow foundation ($D_f <= B$) suggested by Terzaghi can be used for computation of ultimate bearing capacity of a foundation soil

$$Q_{ult} = cN_c s_c + \gamma D_f N_q s_q + 0.5\gamma B N_\gamma s_\gamma$$

Where

c = cohesion
γ = Unit weight of soil,
B = Minimum width of foundation,
D_f = Effective depth of foundation or surcharge depth,

s_c, s_q, s_γ = shape factors,

N_c, N_q, N_γ = Bearing capacity factors.

When using the bearing capacity equation the following points must be considered :

1. The equation is a general one for both cohesive and cohesionless soils. When using for cohesive soils only, the first term (with $N_c = 5.14$) and second term (with $N_q=1$), only will be applicable.

2. Influence of ground water has a significant effect on bearing capacity. The influence of ground water shall be accounted as follows:

   a. No correction is needed for water table at a depth equal to or greater than depth B below the footing base.

   b. For water table above the level of base of footing, substitute γ by submerged unit weight γ' in the term $0.5\gamma BN_\gamma s_\gamma$. Surcharge γD_f is to be estimated as effective surcharge. That means the unit weight of soil below the ground water shall be γ'.

   c. For water table below the base of the footing but within depth B from footing base, replace γ in the term $0.5\gamma BN_\gamma s_\gamma$ by $\gamma' + (Z_d/B)(\gamma - \gamma')$, where Z_d is the depth of ground water below the base of footing.

3. For eccentrically loaded foundations and retaining walls the effective width (B'), defined as the actual least dimension of the footing B minus twice the eccentricity (e) i.e. $B' = B - 2e$, has to be considered for the bearing capacity computation.

4. In all of the following SPT value N should be corrected for fine sand and silt under ground water submergence when the SPT values exceed 15. Then,

$$N' = 15 + 1/2(N - 15)$$

   The SPT values need to be corrected for effective overburden pressure. Corrected SPT values be used for design.

5. If values of the angle of the internal friction ϕ for cohesionless soil from test data are not available, ϕ can be interpreted from the N-ϕ relationship given in Table 5.1. The ϕ value of a cohesionless soil changes slightly with saturation, this change need not be considered in the design computation.

6. Where test results of c and q_u are not available (for minor structure), q_u can be read from the N-q_u relationship given in Table 5.2. It should be noted that this relationship is very approximate for cohesive soils. For major structures, c and q_u values from test data are mandatory.

7. The shear strength parameter c of cohesive soils may be taken as equal to half the unconfined compressive strength q_u.

8. The soil and its design parameters within a depth equal to a maximum of 2B below the foundation level, where B is the base width, should be considered for bearing capacity computations.

9. Shape factors and bearing capacity factors may be taken from Table 5.3 and Table 5.4 respectively.

Table 5.1

Relative Density of Granular Soils

Degree of Compaction	Relative Density (%)	SPT Values (N)	Angle of Internal Friction (ϕ)	Field Identification
Very loose	0-15	0-4	0-28°	Reinforcing rod can be pushed into soil several feet
Loose	15-35	4-10	28°-30°	
Medium Dense	35-65	10-30	30°-36°	
Dense	65-85	30-50	36°-41°	Difficult to drive 2"x 4" stake with a sledge hammer
Very Dense	85-100	50 and above	41° and above	

Table 5.2

Consistency of Cohesive Soil

Consistency	Unconfined Compressive Strength, q_u (tons/sft)	SPT Value (N Blows/ft)	Field Identification
Very Soft	Less than 0.25	Below 2	Easily penetrated several inches by fist
Soft	0.25-0.50	2-4	Easily penetrated by thumb
Medium stiff	0.50-1.00	4-8	Can be penetrated by thumb with moderate effort
Stiff	1.00-2.00	8-16	Readily indented by thumb but penetrated only with great effort
Very Stiff	2.00-4.00	16-32	Readily indented by thumb nail
Hard	over 4.00	over 32	Indented with difficulty by thumb nail.

Table 5.3

Shape Factors

Shape of Footing	s_c	s_q	s_γ
Rectangle	$1+(B/L)(N_q/N_c)$	$1+(B/L)\tan\phi$	1-0.4(B/L)
Square(side=B)	$1+(N_q/N_c)$	$1+\tan\phi$	0.80
Circle(dia=B)	$1+(N_q/N_c)$	$1+\tan\phi$	0.60
Strip	1.00	1.00	1.00

Table 5.4
Bearing Capacity Factors (For shallow foundations)

ϕ	N_c	N_q	N_γ	N_q/N_c	tan ϕ
0	5.14	1.00	0.00	0.20	0.00
1	5.38	1.09	0.07	0.20	0.02
2	5.63	1.20	0.15	0.21	0.03
3	5.90	1.31	0.24	0.22	0.05
4	6.19	1.43	0.34	0.23	0.07
5	6.49	1.57	0.45	0.24	0.09
6	6.81	1.72	0.57	0.25	0.11
7	7.16	1.88	0.71	0.26	0.12
8	7.53	2.06	0.86	0.27	0.14
9	7.92	2.25	1.03	0.28	0.15
10	8.35	2.47	1.22	0.30	0.18
11	8.80	2.71	1.44	0.31	0.19
12	9.28	2.97	1.69	0.32	0.21
13	9.81	3.26	1.97	0.33	0.23
14	10.37	3.59	2.29	0.35	0.25
15	10.98	3.94	2.65	0.36	0.27
16	11.63	4.34	3.06	0.37	0.29
17	12.34	4.77	3.53	0.39	0.31
18	13.10	5.26	4.07	0.40	0.32
19	13.93	5.80	4.68	0.42	0.34
20	14.83	6.40	5.39	0.43	0.36
21	15.82	7.07	6.20	0.45	0.38
22	16.88	7.82	7.13	0.46	0.40
23	18.05	8.66	8.20	0.48	0.42
24	19.32	9.60	9.44	0.50	0.45
25	20.72	10.66	10.88	0.51	0.47
26	22.25	11.85	12.54	0.53	0.49
27	23.94	13.20	14.47	0.55	0.51
28	25.80	14.72	16.72	0.57	0.53
29	27.86	16.44	19.34	0.59	0.55
30	30.14	18.40	22.40	0.61	0.58
31	32.67	20.63	25.99	0.63	0.60
32	35.49	23.18	30.22	0.65	0.62
33	38.64	26.09	35.19	0.68	0.65
34	42.16	29.44	41.06	0.70	0.67
35	46.12	33.30	48.03	0.72	0.70
36	50.59	37.75	56.31	0.75	0.73
37	55.63	42.92	66.19	0.77	0.75
38	61.35	48.93	78.03	0.80	0.78
39	67.87	55.96	92.25	0.82	0.81
40	75.31	64.20	109.41	0.85	0.84
41	83.86	73.90	130.22	0.88	0.87
42	93.71	85.38	155.55	0.91	0.90
43	105.11	99.02	186.54	0.94	0.93
44	118.37	115.31	224.64	0.97	0.97
45	133.88	134.88	271.76	1.01	1.00
46	152.10	158.51	330.35	3.04	1.04
47	173.64	187.21	403.67	1.08	1.07
48	199.26	222.31	496.01	1.12	1.11
49	229.93	265.51	613.16	1.15	1.15
50	266.89	319.07	762.89	1.20	1.19

Note : Bearing capacity factors recommended in Table 5.4 are for shallow foundations for general shear failure cases. For local shear (as well as punching shear) Terzaghi proposed reduced shear strength parameters and hence reduced bearing capacity factors $c'=0.67c$ and $\phi' = \tan^{-1}(0.67 \tan\phi)$, where c and ϕ are strength parameters as determined from test and as used for general shear failure cases.

5.3 Bearing Capacity for Shallow Foundations (Hansen's Equation)

In addition to the parameters described above the depth factors and load inclination factors are accounted for estimating the bearing capacity of a foundation. With the inclusion of these factors for foundations placed on horizontal surfaces the general equation for bearing capacity for both shallow and deep foundations as proposed by Hansen is:

$$Q_{ult} = cN_c s_c d_c i_c + \gamma D_f N_q s_q d_q i_q + 0.5\gamma B N_\gamma s_\gamma d_\gamma i_\gamma$$

$$N_q = e^{\pi \tan\Phi} \tan^2(45° + \Phi/2)$$

$$N_c = (N_q - 1)\cot\Phi$$

$$N\gamma = 1.5(N_q - 1)\tan\Phi$$

For above bearing capacity factors Table 5.5 may also be used:

Table 5.5

Bearing capacity factors for Hansen's equations

Φ	N_c	N_q	$N\gamma$
0	5.14	1.0	0.0
5	6.49	1.6	0.1
10	8.34	2.5	0.4
15	10.97	3.9	1.2
20	14.83	6.4	2.9
25	20.71	10.7	6.8
26	22.25	11.8	7.9
28	25.79	14.7	10.9
30	30.13	18.4	15.1
32	35.47	23.2	20.8
34	42.14	29.4	28.7
36	50.55	37.7	40.0
38	61.31	48.9	56.1
40	75.25	64.1	79.4
45	133.73	134.7	200.5
50	266.50	318.5	567.4

Table 5.6

Depth factors

d_c	d_q	$d\gamma$
$1+0.4D/B$ for $D/B \le 1$ $1+0.4\tan^{-1}D/B$ for $D/B>1$	$1+2\tan\Phi(1-\sin\Phi)^2 D/B$ for $D/B \le 1$ $1+2\tan\Phi(1-\sin\Phi)^2 \tan^{-1} D/B$ for $D/B>1$	1.0 for all Φ

For shape factors S_c, S_q and S_γ Table 5.3 may be used.

For most practical problems the load remains vertical and hence load inclination factors $i_c=i_q=i_\gamma=1$.

Note: For inclined loading & more detailed discussion "**FOUNDATION ANALYSIS AND DESIGN**" by Joseph E. Bowles, Fourth Edition may be consulted.

5.4 Bearing capacity of cohesive soil (clay and plastic silt)

The net ultimate bearing pressure for vertical load on clay soils is normally computed as a simplification of Hansen's equation;

$$q_{ult} = cN_c s_c d_c + qN_q s_q d_q - q$$

which is often written (and dropping s_q, d_q) as,

$$q_{ult} = cN_c s_c d_c + q(N_q - 1)$$

When $\Phi = 0$ and $N_q = 1.0$, we have

$$q_{ult} = cN_c s_c d_c$$

The combined effect of $N_c s_c d_c$ has a limiting value of about 9.0 for square and round footings and 7.6 for a strip footing for D/B ≥5. The value of 9.0 has both a mathematical and experimental basis. Table 5.7 shows values of $N_c s_c d_c$ for different values of D/B.

Table 5.7

Values of $(N_c s_c d_c)$ for several D/B ratio (after Skempton (1951))]

Footing	D/B=0	1	2	5	10	50
Round or square	6.2	7.6	8.4	9.0	9.0	9.0
Strip	5.14	6.4	7.2	7.6	7.6	7.6

Actual values seem to range from about 7.5 to 12.0, so in the absence of specific local data indicating otherwise, a value of 9.0 should be used when D/B ≥ 5 and the foundation base is round or square.

For small jobs where a better economy can be achieved by using a conservative design based on simple test results, the Standard Penetration Test is used. The N values of SPT should be adjusted for overburden pressure.

Approximate value of unconfined compressive strength of clayey soil can also be obtained from the following empirical relationship:

$$q_u = N/7.5 \quad \text{Kg/cm}^2$$

$$= 13\ N \quad \text{KN/m}^2$$

5.5 Settlement in Cohesive Soils

The computation of total settlement S due to consolidation of soil, using Terzaghi's method, is given below for normally consolidated, overconsolidated and underconsolidated soils.

5.5.1 Settlement of Normally Consolidated Sub-soil

Soil which has not been subjected to pressure in excess of the present overburden pressure is considered normally consolidated soil, For such soils, the settlement S is given by

$$S = \frac{C_c H}{1 + e_o} \log_{10} \frac{P_o + \Delta P}{P_o}$$

where:

H	=	thickness of layer
e_o	=	natural void ratio of the soil in place
P_o	=	original effective overburden pressure at height of consolidating layer before construction of structure.
ΔP	=	increase in pressure at mid-height of the cohesive layer caused by construction of the structure.
C_c	=	compression index to be determined from the results of a consolidation test.

5.5.2 Settlement of Overconsolidated Sub-soil

Soil which has been consolidated in the past to a pressure greater than the present overburden pressure is termed overconsolidated or pre-consolidated soil. For such soils, the settlement S is computed as follows:

When $P + \Delta P < Pc$

$$S = \frac{H}{1 + e_o} C_r \log_{10} \frac{P + \Delta P}{P}$$

When $P + \Delta P > Pc$

$$S = \frac{H}{1 + e_o} [C_r \log_{10} \frac{P_c}{P} + C_c \log_{10} \frac{P + \Delta P}{P_r}]$$

Where:

C_r	=	recompression index
P_r	=	pre-consolidation pressure

5.5.3 Settlement of Underconsolidated Soils

The situation occasionally arises in which a stratum of cohesive soil is found to have been subjected to a pre-consolidation pressure less than the existing overburden pressure. This circumstance may occur when a deposit has not reached equilibrium under the applied overburden stress e.g. in areas of recent fill or where the ground water table has recently been lowered. In such cases if an additional load is applied to the soil, settlement will occur not only in response to the additional load but to the existing load under which equilibrium has not yet been attained. The total settlement in such case is given by the following equation:

$$S = \frac{H}{1+e_o} C_c \log_{10}\left[\frac{P+\Delta P}{P_c}\right]$$

The following points must be considered when computing the total settlement:

i) The consolidation condition i.e., whether the soil is normally consolidated, overconsolidated or underconsolidated, must be determined from the review of available data on site history, geology and laboratory consolidation test.

ii) The distribution of foundation pressure (increased pressure) along the vertical profiles shall be determined by 2:1 pressure distribution theory for minor structures (Figure 5.1) and by using the influence chart (Figure 5.3) for major structures.

iii) Where there is an infinite depth of compressive layer, the value of H should be taken upto a depth where the pressure is 0.10 times the contact pressure; beyond this, the pressure may be neglected.

iv) For layered soils having different consolidation characteristics, the settlement of each layer is to be computed. The total settlement is then equal to the total of all the computed settlements.

5.6 Bearing Capacity of Granular soil (Sand & Non-plastic silt)

5.6.1 Bearing Capacity of footings & Mats on granular soil (Based on SPT value)

The following empirical equations are more convenient to evaluate bearing capacity.

A. For square footing:

$q_{ult} = 2N^2 B\ Rw + 6(100 + N^2) D\ R'w;$ psf

$q_{ult} = 0.315\ N^2 B\ Rw + 0.944\ (100 + N^2) D\ R'w;$ KN/m²

B. For very long footings:

$q_{ult} = 3N^2 B\ Rw + 5(100 + N^2) D\ R'w;$ psf

$q_{ult} = 0.472N^2 B\ Rw + 0.785(100 + N^2) D\ R'w;$ KN/m²

C. For mat foundations (Terzaghi & Peck)

$$q_{na} = 1/50[N^2 B\ Rw + 3(100 + N^2) D\ R'w] \quad ton/m^2$$

$$q_{na} = 0.22N \quad ton/sft \ ; \qquad (5 \le N \le 50)$$

where,

q_{ult} = net ultimate bearing pressure,

= pressure at bottom of footing in excess of the pressure at the same level due to the weight of the soil immediately surrounding the footing.

q_{na} = net allowable bearing pressure.

N = standard penetration resistance, number of blows per foot, N values should be adjusted if the penetration test is made at shallow depth.

B = width of footing , ft/m.

D = depth of footing, ft/m, measured from ground surface to bottom of footing. If the ground levels on both sides of footing are not equal, D should be measured from the lowest ground level. If D>B, use D=B for computation.

R_w & R_w' = correction factors for position of water level. When the water level is below the bottom of footing, R'_w=1.0; and when water level is above the bottom of footing, R_w = 0.5. Use interpolated values if located in between (Figure.5.2).

5.6.2 Allowable bearing pressure based on tolerable settlement (Peck and Terzaghi)

The following empirical formulae is used for footing and rafts/mats on granular soil.

A. For individual Footings on sand

$$q_a = 720\,(N-3)\,[\frac{B+1}{2B}]^2 R'_w \qquad psf$$

where, q_a = net allowable bearing pressure for maximum settlement of 25 mm (1 inch). It should be taken as the pressure at the bottom the footing in excess of the weight of soil immediately surrounding the footing. If the maximum tolerable settlement is different from 25 mm (1 inch), the equation may be modified on the assumption that settlement is proportional to the bearing pressure.

The value of q_a may be increased linearly with depth of footing upto 100 percent when the depth is equal to the width of the footing. In other words, the above equation may be multiplied by a factor (1+D/B) with a limiting value of 2 when D/B exceeds unity.

B. For Rafts/Mats on sand

The following empirical formula based on N value of SPT give the allowable bearing capacity for a mat foundation for 50 mm allowable settlement (Peck & Tergzaghi);

$$q_{a1} = 360\ (N - 3)\ R'_w \quad \text{psf}$$

$$= 17.27\ (N - 3)\ R'_w \quad KN/m^2$$

$$q_{a2} = 4/3\ N^2BR_w + 4\ (100 + N^2)DR'_w \quad \text{psf}$$

$$= 0.21N^2BR_w + 0.63\ (100 + N^2)DR'_w \quad KN/m^2$$

where,
B = smaller dimension of the mat, ft/m.
D = depth of foundation, ft/m.

R_w & R'_w = **reduction factors for water level.**

Smaller of q_{a1} and q_{a2} should be used.

C. For Mats on sand

Another empirical formula based on N value give the allowable bearing capacity of mats for 25 mm allowable settlement.

$$q_a = N/F_2\ .\ K_d.$$

Where,

q_a = allowable bearing capacity of soil for 25 mm settlement.

K_d = $1 + 0.33\ D/B \le 1.33$

D = depth of foundation.

B = Width of footing.

F_2 = 0.08 in SI unit and 4.0 in FPS unit.

N is the statistical average value of SPT for the footing influence zone of about 0.5B above footing base to at least 2B below.

5.7 Settlement in Cohesionless Soils

Settlement of structures founded on cohesionless soils can be computed form Figure 5.4, when the average N values are known. For certain N values, the allowable bearing pressure for 25 mm settlement or settlement values for any foundation pressure can be computed as below:

i) For an isolated footing with the groundwater table below the least dimension of the footing, the allowable soil pressure can be read from Figure 5.4 corresponding to the average corrected N values under the structure. Where the footing is submerged completely by groundwater, the allowable soil pressure shall be half the previous value.

ii) For raft or combined footing foundation, an allowable bearing pressure of twice that allowed for an individual footing may be taken. Because of its rigidity, the differential settlement of a raft or combined footing is less than that of an individual footing.

iii) Should the value of settlement for any imposed foundation pressure need to be computed, this can be obtained by simple proportioning as the settlement is directly related to the soil pressure for a particular soil type.

5.8 Soil Improvement

When soil at sites at shallow depths are inadequate for support of proposed structures, in most of such cases deep foundations i.e. piles, caissons have been used. But deep foundation is not the only solution in such cases. Under certain condition sub-soil improvement by compaction, soil replacement, pre-compression, vibroflotation, stone column etc. can be carried out.

The primary objective of soil improvement is to enhance the strength and to improve the compressibility characteristics of the soil by increasing the density of coarse grained soils or decreasing the moisture content of fine grained soils.

The following methods of artificial soil improvement may be used:

- *physical:* consolidation by drainage with or without preloading;
- *mechanical:* reducing soil porosity and increasing density by ramming and vibrating of soils, compacting by a pile, and replacement of soft layers.

Of the above methods soil replacement is considered to be the most applicable for field and working conditions where ground water is not a major problem.

Because many soils can be made into useful construction materials, if properly treated soil improvement may be taken up in many of the suitable cases.

Soil improvement methods and types to be taken up includes

i) in-situ deep compaction of clayey and sandy soils and
ii) precompression with and without vertical drains.

In situ densification of loose cohesionless soil can be achieved by

(a) blasting
(b) Vibrocompaction and
(c) heavy tamping.

Vibrocompaction refer collectively, to all methods involving insertion of vibrating probes into the ground with or without addition of

Backfill material. Compaction piles are also considered in the category. Out of all these methods sand compaction piles are suitable for cohesive soils also.

Consolidation by drainage with or without pre-loading may be applicable for both cohesive and non-cohesive soils.

To accomplish the needed improvement the following properties of soil and factors need to be considered :

a) Soil type, especially its gradation and fines content.
b) Degree of saturation and water table location.
c) Initial relative density.
d) Initial in-situ stresses.
e) Initial soil structures.
f) Characteristics of the method used.

5.9 Soil Replacement

When a soft or very soft deposit is encountered near the usable portion of the structure, the soft deposit(soil) may be replaced by a firmer one, and a structure then built directly over it, although if the compressible depth is very high this may not be possible. As the depth of replacement depends on the level of the groundwater table, it is most suitable for shallow depths. The main objective of soil replacement is to :

- reduce settlement of foundations;
- achieve uniform settlement;
- increase stability.

The maximum thickness of a sand cushion under a foundation should be selected in such a way that its settlement, as well as that of soil underlying it, would be less than the allowable settlement for a given structure. If the depth of compressible layer is significant and it is not possible to remove the whole layer the following procedure is to be carried out:

- Calculate the settlement due to whole layer.

- If the computed settlement is more, then select a depth of cushion layer.

- Calculate the settlement with the cushion layer.

- Repeat the process until the allowable settlement is attained.

In constructing a sand cushion, it is essential to ensure that the desired relative density of sand which is related with the desired bearing capacity of soil, is obtained. In selecting soil replacement below ground water, comparative statement should be made between soil replacement, soil improvement and deep foundation.

5.10 Pile Foundations:

5.10.1 Types of piles:

Piles may be classified according to their function, material composition and method of installation.

I. *Classification based on functions:*

a) Point bearing piles: which obtains its major portion of it's support from end bearing on the soil.

b) Friction piles: which obtains its support mainly from its side friction.

c) Compaction piles: such piles are used to increase the relative density of granular soil.

Piles may carry loads both by friction and by end bearing.

II. *Classification based on composition:*

Piles may be classified according to their composition

a) Concrete piles : i) Pre-cast RCC piles
ii) Cast-in-situ RCC piles.

b) Timber piles
c) Composite piles

III. *Classification based on method of installation:*

According to the method of construction, piles can be broadly classified as :

(a) Driven piles
(b) Bored piles

5.11 Pile Capacity

The bearing capacity of a single pile is controlled by the structural strength of the pile and the supporting strength of the soil, and the smaller of the two values should be used for the design purpose. Structural capacity is governed by the permissible stress in the pile materials. Generally building codes stipulate the maximum allowable material stresses.

The general equation for ultimate bearing capacity (Q_{ult}) and allowable bearing capacity (Q_a) of pile by static approach is given by:

$$Q_{ult} = Q_{tp} + Q_{sp} \quad \&$$

$$Q_a = Q_{ult}/F_s$$

where, Q_{ult} = ultimate capacity of pile based on soil support

Q_{sp} = ultimate frictional force developed along the shaft of pile i.e. shaft resistance

Q_{tp} = ultimate point or tip resistance of piles

Q_a = allowable capacity of pile based on soil support

F_s = factor of safety of 2.5

5.11.1 Pile Capacity in Cohesionless Soil

I(a) From Skin Friction

The ultimate capacity from skin friction of pile in cohesionless soil is expressed as

$$Q_{sp} = k_s P_d \tan\delta \, A_s$$

where,

Q_{sp} = Ultimate shaft resistance or skin friction.
K_s = an earth pressure coefficient obtained from Table 5.8.
P_d = effective overburden pressure
δ = angle of wall friction (Table 5.8)
A_s = Embedded Surface area of pile

(b) From Tip resistance

The capacity from base or tip resistance of pile in cohesionless soil is expressed as,

$$Q_{tp} = A_b P_d N_q$$

Where,

Q_{tp} = Net total base resistance
A_b = Area of base of the pile.
N_q = Bearing Capacity factor -related with Φ value.
P_d = effective over burden pressure at pile base level.

Table 5.8

Values of K_s and δ

Pile material	δ	Value of K_s	
		Low relative density	High relative density
Steel	20°	0.5	1.0
Concrete	$3/4\Phi$	1.0	2.0
Wood	$2/3\Phi$	1.5	4.0

Where bored pile in cohesionless soil are installed by 'shelling' or 'bailing', it must be assumed that; the soil will be loosened as a result of the boring operation, even though it may initially be in a dense or medium dense state - the Φ value that will be based to calculate both Q_{sp} and Q_{tp} will be for loose condition. But where piles in cohesionless soil are installed by rotary drilling under a bentonite slurry it may be assumed that the Φ value used to calculate both the Q_{sp} and Q_{tp} will correspond to the undisturbed soil conditions.

The assumption of loose conditions for calculating skin friction and end or base resistance means that the ultimate carrying capacity of bored pile in cohesionless soil will be considerably lower than that of a pile driven into the same soil type. Φ value for bored pile should be taken to be 2° below that obtained by test for normal cases.

II. From standard penetration test

This equation is applicable for cohesionless soils and may be applied for estimation of bearing capacity in pure cohesive soil.

$$Q_{ult} = Q_{all} \times F.S. = 40\,NA_p + \bar{N}A_s/5$$

where, Q_{ult}, Q_{all} are in tones.

N = Average blow count from SPT at pile tip (Average from pile tip to twice the diameter below it.)

$\bar{N}$ = Average blow count from SPT over the length of the shaft.

A_p = Area of pile tip in sq. meter.

A_s = Area of pile shaft, embedded in soil, in sq. meter.

In SI units with Q_{ult}, Q_{all} in KN and rest of quantities as above meaning. The above equation may be written as,

$$Q_{ult} = Q_{all} \times F.S. = 392NA_p + 1.96\bar{N}A_s.$$

Factor of safety in case of above two equations is not less than 4.

5.11.2 Pile Capacity in Cohesive Soils

(a) The ultimate skin friction on pile shaft is given by the equation,

$$Q_s = \alpha c_d A_s$$

where,

α = adhesion factor. Where there is no previous experience of adhesion factors in any particular type of clay, M.J. Tomblinson recommends adopting value for α = 0.45 (but not more than 100 KN/m²) for skin friction in firm to stiff clays, with adequate load testing to confirm the design value. However there is no reason to suppose that the full shear strength of very soft to soft clay should not be mobilized in skin friction provided that time is given for regain of shear strength lost by remolding during boring operations.

C_d = Average undisturbed shear strength of clay adjacent to shaft.

A_s = Surface area of the shaft.

A general safety factor on the ultimate load is 2.5. The allowable bearing capacity i.e. the sum of the base resistance and skin friction divided by safety factor should ensure that the settlement at the working load will not exceed a tolerable value.

In case of uniform clays or clays increasing progressively in shear strength with depth the average value of shear strength over the whole shaft length is taken for C_d. Where the clay exist in layers of appreciably differing consistency e.g. soft clay over stiff clay, the skin friction is separately calculated for each layer using the adhesion factor appropriate to the shear strength and over burden conditions.

Unless proven by pile load test, the values of adhesion given below may be used for design purpose:

Materials of pile	Soil consistency	Cohesive strength (psf)	Adhesion to pile (psf)
Concrete and timber	soft medium stiff	0 - 750 750 - 1500 1500 - 3000	0 - 700 700 - 900 900 - 1300

A friction pile in clay is supported by adhesion between the pile and the soil. The bearing capacity of such a pile is approximately equal to the unit adhesion multiplied by the embedded area of the pile.

pile driving in soft cays tend to disturb the clay around the pile. consequently the clay looses a large portion of its strength. The disturbed clay begins to consolidate and to gain strength rapidly and immediately after driving piles. In most cases, the full strength of the virgin clay is likely to be regained one month or less after driving. However, the skin friction (adhesion) is not necessarily equal to the cohesion of the soil, since driving a pile into a cohesive soil can alter the physical characteristics of the soil to a marked extent.

(b) The ultimate base or tip resistance of pile in clay or clayey silt is expressed as

$$Q_{tp} = N_c C_b A_b$$

where,

N_c = a bearing Capacity factor, the maximum value of N_c is 9, unless depth of embedment of pile is less than five times the pile diameter. N_c may be selected from the chart given below:

C_b = Undisturbed Shear Strength at the base of the pile. C_b is representative of the fissured shear strength i.e. the lower range of values.

A_b = Area of base of the pile.

pile size	N_c
less than 450 mm dia	9
450 mm to 900 mm dia	7
> 900 mm dia	6

5.11.3 Pile capacity based on Dynamic formulae

All the pile formulae are based on performance of pile during driving, a dynamic condition, whereas most of the piles in foundation are subjected to practically static loading condition. Still these formulae have been used satisfactorily.

There is not a single dynamic formula which could predict satisfactorily the pile capacity. There are so many dynamic formulae to determine the pile capacity. Out of these the S., Janbu and Gates formulae appear to be more accurate and as such are recommended for a check of bearing capacity computation and providing a guide line for monitoring the installation process of pre-cast RCC piles.

(a) Gates Formula

$Q_{ult} = 3.Q_{all} = 105[e_hE_h]^{1/2} \log(250/s_e)$

Here F.S. = 3.

In SI unit $Q_{ult} = 3.Q_{all} = 4.45[e_hE_h]^{1/2} \log(250/s_e)$

Where, Q_{ult} and Q_{all} have usual meanings.

E_h = $W_h.H$ for drop hammer, or single acting steam hammer of wt. W_h and falling through a height H (free fall)

s_e = Final set (penetration) per blow; usually taken as average penetration for the last 5 blows of a drop hammer; or 20 blows of steam hammer; in mm.

e_h = pile hammer efficiency factor as given in Table. 5.9.

Table 5.9

Recommended values for pile hammer efficiency

Hammer Type	e_h **
Drop hammer	
Trigger release	1.00
Rope and friction winch	0.75
Single-acting steam hammer	
McKiernan-Terry	0.85
Warrington-Vulcan	0.75
Double-acting steam hammer	
McKiernan-Terry	0.85
National	0.85
Union	0.85
Differential-acting steam hammer	0.75
Diesel hammer	1.00

** Increase by 10 percent when computing fibre stresses.

(b) Danish or S_o formula

$$Q_{ult} = 3.Q_{all} = \frac{10\, e_h E_h}{S_e + \frac{1}{2}\sqrt{\frac{2 e_h E_h L}{A E_{pile}}}}$$

where the various terms are defined as previously but the term

$\sqrt{\frac{2 e_h E_h L}{A E_{pile}}}$ should be in mm.

In SI units it is written as:

$$Q_{ult} = 3.Q_{all} = \frac{1.835\, e_h E_h}{S_e + \frac{1}{2}\left[\frac{2 e_h E_h L}{A E_{pile}}\right]^{1/2}}$$

L = length of pile.

E_{pile} = modulus of elasticity of pile material. Unit should be such that AE_{pile} should be in Kg where A = Area of pile. In SI unit L in meter, AE_{pile} in Kn.

(c) *Janbu Formula*

$$Q_{ult} = 3 . Q_{all} = \frac{1}{k_u} \frac{10\, e_h E_h}{S_e}$$

where, K_u is a dimensionless co-efficient defined as

$$k_u = c_d [1 + \sqrt{1 + \frac{1}{2\, c_d} \cdot \frac{S_o^2}{S_e^2}}]$$

where, c_d = empirical dimensionless co-efficient.

= 0.75 + 0.15 W_p/W_h.

Factor of Safety = 3 to 6.

W_p = Wt. of pile.

W_h = Wt. of hammer.

$$S_o = \sqrt{\frac{2\, e_h E_h L}{A E_{pile}}}$$

5.12 Structural Design of Pre-cast RCC Pile

Design of R.C. pile is usually done from the consideration of total load coming on it from the structure, the handling stresses during lifting and stresses during driving.

5.12.1 Steel

(a) *Main reinforcement:*

The pre-cast piles are designed for bending stresses during pickup and transport at the site, for bending moments from lateral loads and to provide sufficient resistance to vertical loads and any tension forces developed during driving. The minimum pile reinforcement should be 1 percent during driving.

(b) *Lateral reinforcement:*

Similar to columns, the lateral reinforcement in the piles may be provided in the form of links made out of m.s. bars of diameter not less than 6 mm. The volume of the lateral reinforcement in the body of the pile should not be less than 0.2% of the gross volume of the pile. The volume of lateral reinforcement at either end of the pile for a distance equal to 3 times the least width of the pile should not be less than 0.6% of the gross volume.

The transition between the close spacing of the laterals at the end of the pile and the wider spacing in the body of the pile should be made gradually. If the pile is required to penetrate through hard strata additional reinforcement in the form of helix, should be provided in the top of the pile for a distance equal to three times the least width.

5.12.2 Concrete mix

The concrete mix shall have to be selected in such a way that it attains a minimum 28 day cylinder strength of 20 N/mm^2 after 27 days curing.

5.12.3 Clear Cover

The thickness of concrete covering the main bars should not be less than 40 mm. In places where the steel is liable to corrode, say in sea water, the clear cover should be increased to 55 mm.

5.12.4 Forces acting on piles

Pile must be capable of resisting without damage (a) crushing under the permanent design vertical load (b) crushing caused by impact force during driving (c) bending stress occurring during handling (d) tension from uplift force (e) bending stress due to horizontal force (f) bending stress due to eccentric location (g) column action for portion not receiving lateral support from the ground but free-standing in air, water, or very liquid mud.

R.C.C. pile is usually designed as column considering it fixed at one end and hinged at the other, the effective length of the pile being taken 2/3rd the length embedded in the soil. In this regard the formulae for column design shall have to be used.

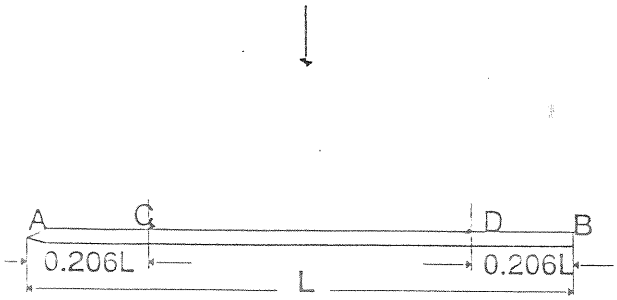
5.12.5 Stresses produced during handling

Pre-cast pile is normally subjected to more bending and shear stresses during, handling, hoisting and driving than under its permanent bearing load. The pile has to be lifted form the places where it was cast and transported to the site of work. For this purpose 25 mm to 50 mm diameter toggle holes or hooks are provided in the pile, during casting. The number of toggle holes/hooks and their location in the pile length require a careful consideration. When the pile is lifted, it is subjected to bending moments and shearing forces on account of the self weight of the pile at the point of suspension.

The proof in respect of the location of the point of suspension mentioned above is given below.

4. *Pile suspended at two points*

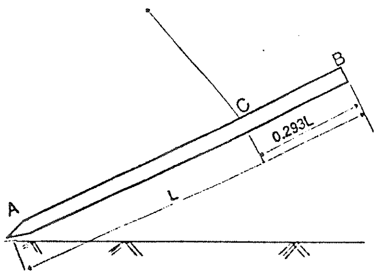
Let, w, kg/m be the self weight of the pile and 'L' meter be the length of the pile. The location of suspension point is shown in figure below.



The value of maximum B.M. (cantilever or mid span B.M) = $\pm\ wl^2/47$.

3. *Location of the point of Support for Erecting a pile*

Before driving, the pile is held vertical at the appropriate place, For this, the pile lying on the ground has to be erected by being suspended by the crane at one point while the shoe of the pile rests on the ground at the other end.



In order that the maximum bending in the cantilever portion CB and in the length AC of the pile may be same, the point C should be located at a distance of 0.2931 from the end B. In this case the maximum bending moment will work out to $wl^2/23$.

5.13 Basic/Structural guide lines for Bored Piles

5.13.1 Steel

The minimum pile reinforcement should be 1 percent

5.13.2 Concrete

Concrete strength for bored piles shall be minimum 22.0 N/mm^2 and shingle or crushed stone shall be used as coarse aggregate.

5.13.3 Clear Cover

Minimum clear cover shall not be less than 75mm.

5.13.4 Size of Bored pile

The extensive researches carried out on large diameter bored piles have provided a sound basis for the practical design purpose. It is general practice to use minimum 400 mm dia bored R.C.C. pile.

5.14 Timber Piles

Timber piles are usually for minor and lightly loaded structure. If timber piles are kept permanently wet or permanently dry i.e. driven wholly above water level, they can have a very long life. However, they are liable to decay in the zone of a fluctuating water table. Care in selection and treatment of timber can prevent or minimize attack by wood destroying insects.

Those parts of timber piles which are permanently buried in the ground and below the lowest ground water level can be untreated. Whenever possible, the concrete pile caps should be taken down so that their underside are below ground water level and the portion of the pile embedded in the caps should be cutoff square to sound wood and liberally coated with creosote or other preservative. Timber piles in general cases should not be used.

5.15 Recommended Pile spacing

The recommended pile spacings should be used only as a guide, not as a hard and fast rule. In dense sand and saturated silt close spacing should be avoided, while for loose sand closer spacing is recommended. Maximum pile spacings are not given in building codes but spacings as high as 8 or 10 times pile diameter have been used on occasion.

Function of piles	Minimum pile spacing
Point bearing piles in hard stratum	2 to 2.5 times butt diameter $\geq$ 750 mm c/c (2 ft 6 in.)
Friction piles	3 to 5 times butt diameter or 1.00 m (c/c of pile)

5.16 Efficiency of a Pile Group

Where the foundation consists of a group of friction piles, soil pressures developed by the individual piles will overlap each other and reduce the bearing capacity of each individual pile. A pile group efficiency factor Eg is then introduced, with which the sum of permissible loads of the piles must be multiplied to arrive at the pile group capacity.

The efficiency or capacity of a pile group, Eg defined as the ratio of actual group capacity to the sum of individual pile capacities can be computed as below:

$$E_s = 1 - \frac{\Phi}{90}\left[\frac{(n-1)m+(m-1)n}{mn}\right]$$

Where,

m = no. of columns in pile group

n = no. of rows.

$\Phi = \tan^{-1}\frac{d}{s}$ (degree)

d = pile diameter

s = centre to centre spacing.

Where piles transmit the structural load directly to the underlying soil, partly by end bearing and partly through skin friction, the ultimate capacity is given by,

$$Q_{ult} = sLP + q_{ult} A$$

Where,

s = shearing resistance (1/2 X the unconfined compression strength of cohesive soil)
L = total embedded length of friction pile or bearing pile.
P = perimeter of the pile group.
A = area enclosing the pile group.
q_{ult} = ultimate bearing capacity of soil at the level of pile tips.

For friction piles in clay, the group capacity is peripheral shear only, viz.,

$$Q_{ult} = sLP.$$

5.17 Settlement of pile groups in cohesive soil

The total settlements of pile groups may be calculated by making use of consolidation settlement equations. The problem involved here is to evaluate the increase in stress ΔP beneath the pile group when the group is subjected to a vertical load P. The computation of stresses depends on the type of soil through which the pile passes.

i) If the soil is of homogeneous clay, the load is assumed to act on a fictitious footing at a depth of 2/3rd from the surface and distributed over the sectional area of the group. The load on the pile group acting of this level is assumed to spread out at 2:1 slope.

ii) Where the pile passes through a very weak layer of depth D and the lower portion of length D_2 is embedded in strong layer, the load is assumed to act at a depth equal to 2/3rd D_2 and spread at 2:1 slope.

iii) In case of point bearing pile supported on a firm strata, the load is assumed to act at the level of the firm strata and spread out at 2:1 slope.

Allowable settlement for piles should be same as shallow foundation.

For pile foundation on sands not underlain by more compressible soil a maximum settlement of 25 mm for isolated footings and 50 mm for raft foundations are frequently allowed. Using the concept of equivalent pier foundation preliminary estimates of the settlement of a pile group in a homogeneous sand deposits can be made directly from the results of penetration tests as for spread.

Meyerhof, Skempton and Mcdonald established the following relationship for estimating the settlement of pile foundation;

$$S = 2PB^{1/2}/N$$

where;

B	=	width of pile groups in ft.
P	=	Net foundation pressure in ton/sft.
N	=	Average corrected SPT.

References:

1) Terzaghi and Peck, Soil Mechanics in Engineering Practice
2) Peck and Hansen, Foundation Engineering
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4) Bowles Joseph. E., Foundation Analysis and Design (4th edition)
5) Punmia B. C., Soil Mechanics and Foundations
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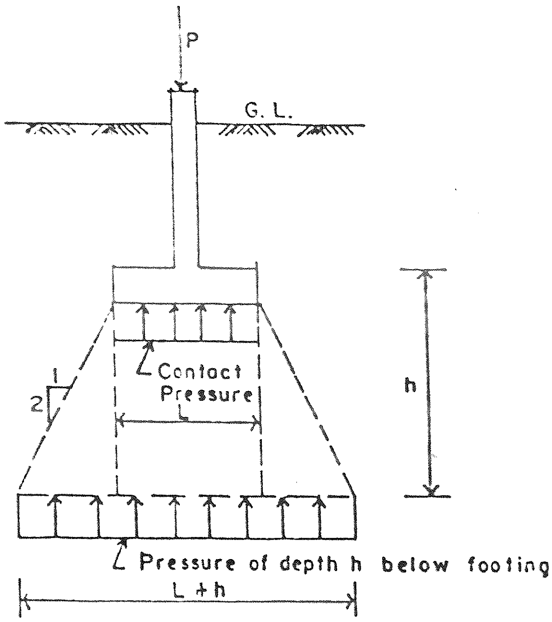
10) SRP, Design Criteria, BWDB System Rehabilitation Project, October, 1993
11) CIDA-1993, Foundation Design for Small Scale Hydraulic Structures



Approximate distribution of pressure within soil mass under footing

Fig. 5.1

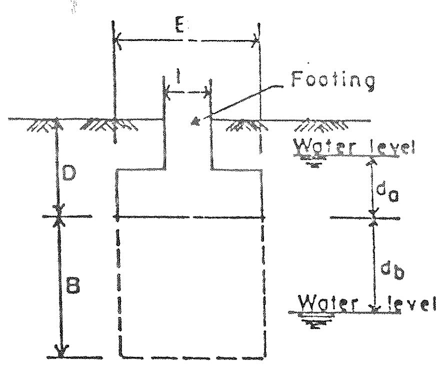


Fig. 5.2 (a)

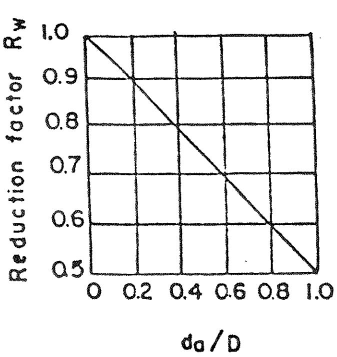


Fig. 5.2 (b)

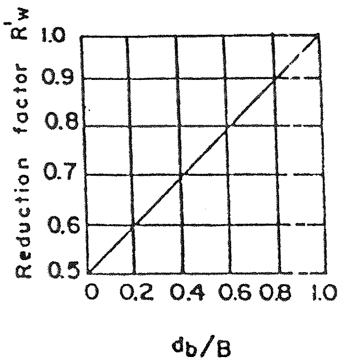


Fig. 5.2 (c)

Fig. 5.2 Correction factor for position of water level :

(a) depth of water level with respect to dimension of footing;

(b) water level above base of footing ;

(c) water level below base of footing. After AREA.

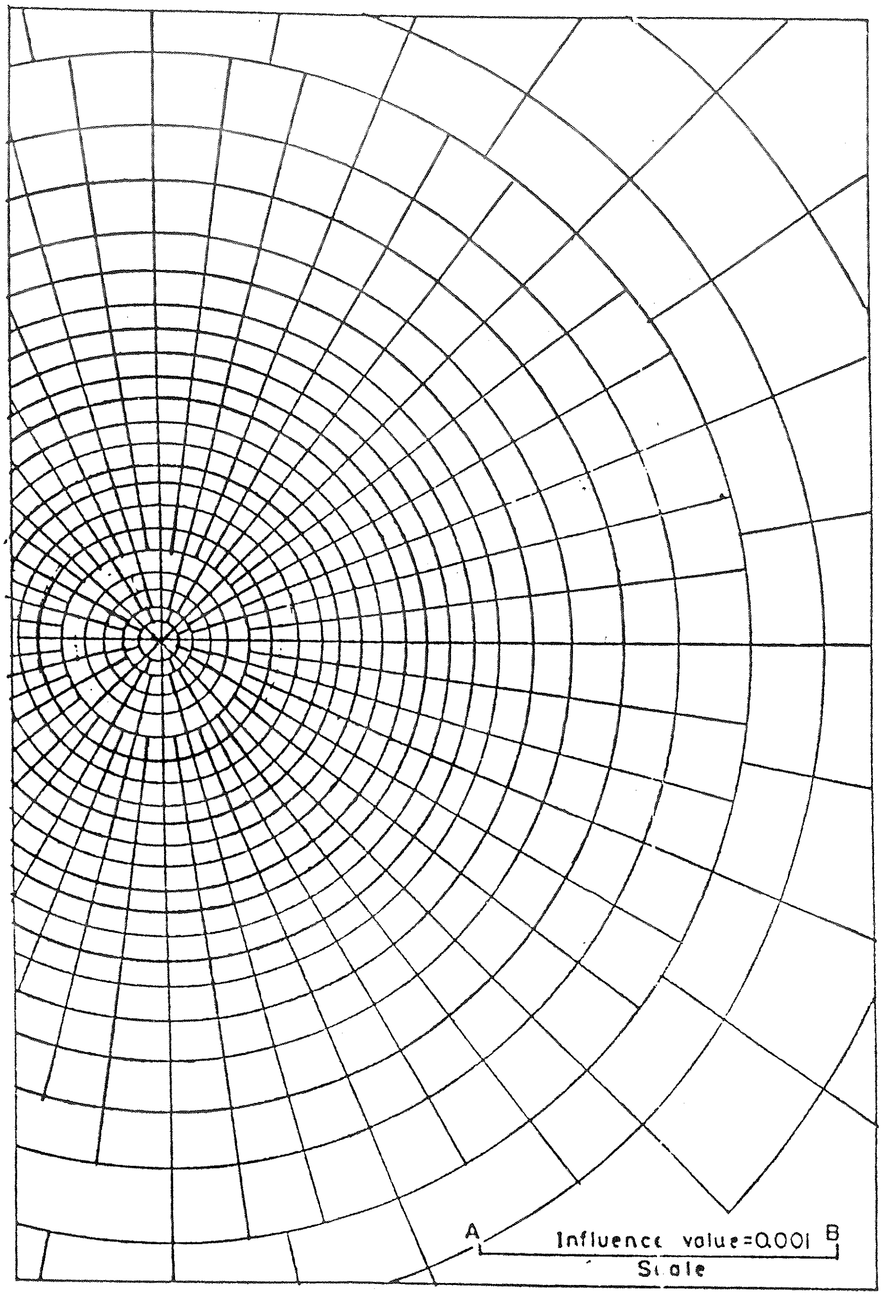


Fig. 5.3 Newmark influence chart for computing vertical pressure. After Corps of Engineers.

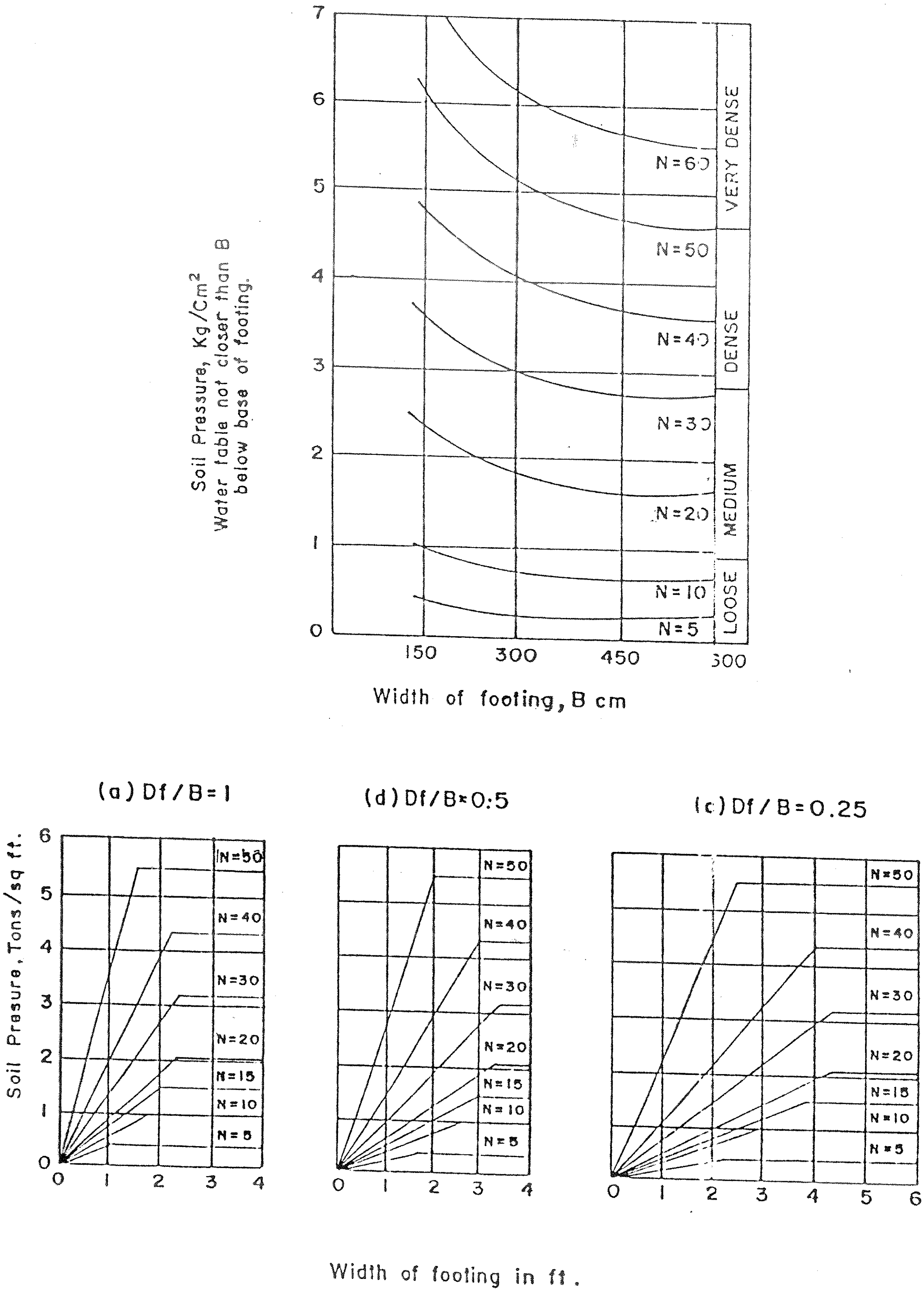


Fig.-5·4 Allowable Bearing Pressure of Footing on Sand from Settlement Consideration.

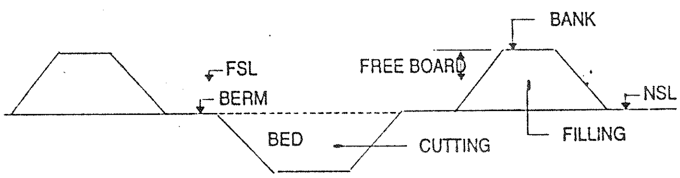
6.0 DESIGN CRITERIA FOR IRRIGATION STRUCTURES

6.1 Irrigation Canal

6.1.1 Layout of Irrigation Canal

The irrigation distribution system consists of various components like regulating structures, cross-drainage structures (syphons, aqueducts), check structures, off-take structures, bridges, lined or unlined canals etc.

Based on the contour maps (irrigation planning map) layout plan for the irrigation system would be developed with due consideration of the drainage system. Irrigation canal would follow the high ridge as far as possible. Layout plan will include demarcation of area to be served by each field outlet.



TYPICAL SEC.OF AN IRRIGATION CANAL

6.1.2 Discharge

6.1.2.1 Duty (Flow Duty)

For small projects duty can be assumed from the following table

Crop	Duty in hectares/cumec
Rice	775
Other Kharif	1500
Rabi	1800
Sugar cane	730
Perennial	1100
Hot fodder	2000

For large projects the duty should be calculated by analysing hydro meteorological data. In that process the steps described in article 1.5.2 may be followed.

6.1.2.2 Capacity of Irrigation Canal

The discharge at field outlet is calculated on the basis of command area, cropping pattern and duty. Taking into consideration the seepage and percolation losses, the discharge of Irrigation Canal at each level is calculated by adding discharge at field outlets.

6.1.2.3 Canal Losses

During the passage of water through the main canal to the outlet at the head of the water course, water is lost by evaporation from the surface and by seepage and percolation through the peripheries and bed of the canals. These loesses are sometimes very high, and of the order of 25 to 50% of the water diverted.

A combined figure for seepage as well as for evaporation losses are given below in tabulated form:

Soil Type	Total loss in cumec/million sq.m of wetted area
1. Impervious clay loam	0.9 to 1.2
2. Silty soil	1.2 to 2.7
3. Sandy clay loam	2.7 to 3.6
4. Sandy loam	3.6 to 5.2
5. Loose sandy soil	5.2 to 6.1

6.1.2.4 Canal Efficiency

As conveyance and operational losses are often difficult to quantify, efficiencies for main, secondary and field canals based upon experience in the country are taken for determining the canal discharge required at the main pumping station or at the canal head works. The following conveyance/operational efficiencies, which assume that 25% of the losses from the fields and secondary system would be re-used will be taken for canals:

- main - 85%
- secondary - 86.5%
- tertiary - 66.2%

For design purpose, an overall efficiency of 50% will be adopted.

6.1.2.5 Section Design

With the known discharge and assumed bed slope the canal section is designed by Lacey's parameters and Manning's equation. The bed width and depth ratios are kept within specified limits. When the canal carries fairly high sediment the Lacey's theory will be adopted.

Lacey's Theory

$$V = \left(\frac{Qf^2}{140}\right)^{1/6}$$

Where,

V = Average velocity of flow (m/sec)
f = Silt factor
= $1.76\sqrt{d_m}$
d_m = Average particle size in mm
Q = Dominant discharge in cumec

$$R = \frac{5V^2}{2f}$$

A = Q/V

P = $4.75Q^{½}$

$$S = \frac{f^{5/3}}{3340 \times Q^{1/6}}$$

where,

R = Hydraulic mean radius
A = Area of canal section
P = Wetted Perimeter of the canal section.
S = Bed slope

The canals which do not carry much silt (secondary and tertiary level canals) the Manning's equation will be adopted for determining the canal section. The Manning's equation is,

$$V = \frac{R^{2/3}S^{1/2}}{n}$$

where,

V = mean velocity of flow (m/sec)
R = hydraulic mean radius
S = canal slope
n = Coefficient of roughness

6.1.3 Bed width vs depth ratio

(a) For canals upto 15 cumecs

Y = depth of water in canal
= 0.50 B
B = Base width of canal

(b) Canal section design will be governed by the Full Supply Level (F.S.L), sediment in the discharge, land saving etc. Thus the ratio of bed and depth of canal may be worked out as suitable for individual case considering the above factors. A general guideline indicating the depth against each flow for canals of capacity 15 cumec and above is given in the following table.

Discharge Q (cumecs)	Depth Y in metres
15	1.7
30	1.8
75	3.3
150	2.6
300	3.0

6.1.4 Supply Level/Water surface Elevation

(a) In branch canal etc. where some distributaries have to take off, the FSL of the canal should be kept at least 15cm higher than the FSL at the off-taking canal, so as to allow for losses in head regulator.

(b) The FSL of the canal should be about 10 to 20 cm above the ground level for most cases along its length.

6.1.5 Curves of the Canal

Curves in unlined canals shall be as gentle as possible. The curves are generally, simple circular curves.

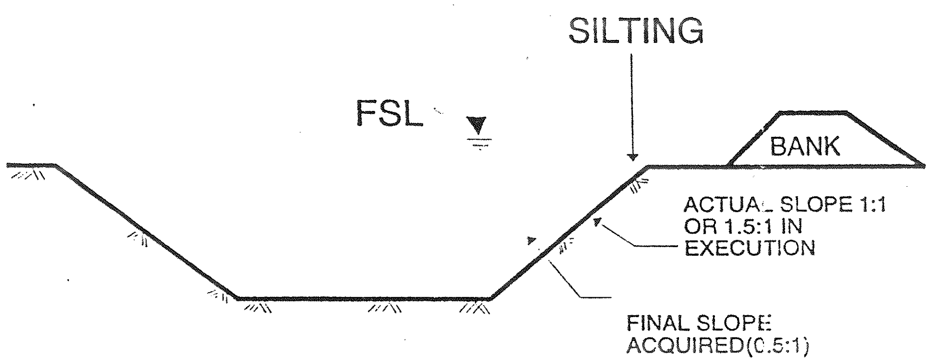
The minimum radii of curves shall be as given in the following table:

Unlined Canal		Lined Canal	
Discharge (m^3/sec)	Radii min. (m)	Discharge (m^3/sec)	Radii min. (m)
80 & above	1500	280 & above	900
80 to 30	1000	280 to 200	760
30 to 15	600	200 to 140	600
15 to 3	300	140 to 70	450
3 to 0.3	150	70 to 40	300
Less than 0.3	90		

6.1.6 Section Parameters

6.1.6.1 Side Slope

The side slopes should be such that they are stable depending upon the type of soil. Generally 1.5H : 1V to 2H : 1V slope in cutting and 2H : 1V to 2.5H : 1V in filling are generally adopted in case of canals with silt laden water. The actual capacity of the canals worked out with 1/2H : 1.0V side slopes even though flatter slopes such 1.0H : 1.0V or 1.5H : 1.0V shall be constructed at the time of excavation. This is because of the fact that the sides of such a canal get silted up to a slope 1/2H : 1.0V in the passage of time as shown in the following fig.



SEC. OF CANAL CARRING SILT LAIDEN WATER

More over, recommended side slopes for unlined canals are given below;

a. Very light loose sand to average sandy soil	1.5:1 to 2:1 2:1 to 3:1	(in cutting) (in filling)
b. Sandy loam and similar soils	1:1 to 1.5:1 2:1	(in cutting) (in filling)
c. Sandy soil or gravel	1:1 to 2:1	
d. Hard soil	0.75 to 1.5:1	

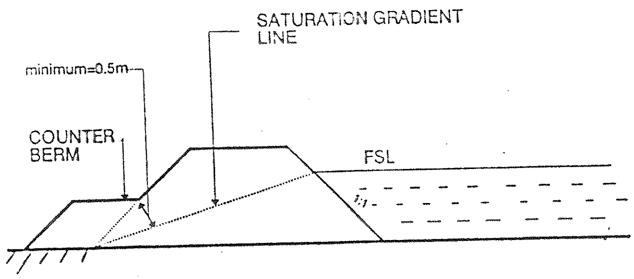
6.1.6.2 Free Board

The height of free board depends upon the size of channel. Generally provided values of free board are given below :

Discharge in cumec	Free Board (metre)
0.5 <	0.25
0.5 - 1.0	0.30
1.0 - 5.0	0.50
5.0 - 10.0	0.60
10.0 - 30.0	0.75
30.0 - 150.0	0.90

6.1.6.3 Back Berm or Counter Berm

Even after providing sufficient section for bank embankment, the saturation gradient line may cut the downstream end of the bank. In such a case, the saturation line can be kept covered at least by 0.5 metre with the help of counter berm.



SEC. OF BANK EMBANKMENT IN WHICH SATURATION GRADIENT LINE HAS CUT THE DOWNSTREAM END OF THE BANK

The straight saturation gradient line may be drawn with the following slopes :

Type of soil	Slope, V : H
Clay	1:4
Silty sand	1:5
Clayey loam	1:6
Loam	1:8
Loamy sand	1:10
Sand	1:15

6.2 Head and cross regulator/check structure

6.2.1 Water way

The effective water way of canal head regulator should not be less than 60% of the bed width of off taking canal and the velocity of flow for fully open regulator should not ordinarily exceed 2.5 m/sec.

The water way of cross regulator/check structure should be fixed with the consideration of the maximum allowable afflux of about 150 mm.

6.2.2 Crest level

The crest level of the distributory head regulator is generally kept at 0.30 to 0.60 m higher than the crest level of the cross regulator. The level of the crest should be provided so as to pass the authorized discharge into the offtaking canal even under normal anticipated low supplies in the parent canal (say 2/3rd of full discharge).

The crest level of cross regulator/check structure is provided normally at the u/s bed level of parent canal. The crest is about 0.10D in height can be provided if parameters so permit (D is the water depth in parent canal).

6.2.3 Shape of the Crest

Upstream face of the crest should be given a batter of 1:1. The downstream sloping glacies should not be steeper than 2:1.

6.2.4 Vent Size

Vent size should be determined with design head difference between upstream and downstream.

6.2.5 Apron

Apron should be designed considering free flow condition (Critical Condition) Design of apron should follow Chapter-2.

6.2.6 Uplift Pressure Calculation

Static Condition

Adequate thickness of the floor should be provided to counteract seepage head from point to point under no surface flow condition.

Dynamic Condition

The floor must be safe under different flow conditions that may prevail. The floor in the jump trough shall be made safe against the seepage head under the specific condition plus 50% of the head difference between the hydraulic jump gradient line and pre jump depth at the point of formation of jump.

6.3 Cross Drainage Works

6.3.1 Design Discharge

Design Discharge for drainage crossings may be taken as 1 in 25 year flood.

6.3.2 Free Board

To guard against unprecedented flood damage specified free board against design discharge are shown below.

Sl. No.	Catchment Area	Increase in design discharge
1.	Upto 500 km^2	30% to 25% decreasing with increase in area
2.	500 to 5000 Km^2	25% to 20%
3.	5000 to 25000 km^2	20% to 10%
4.	Above 25000 km^2	Upto 10%

6.3.3 Head Loss Through the Cross Drainage Works

In case of syphon or syphon aqueducts, Unwin's formula would be used to calculate the head loss;

$$h = [1 + f_1 + f_2 * L/R] * V^2/(2g)$$

Where,

- h = head loss in metres
- L = Length of barrel in metres
- R = hydraulic mean depth in the barrel in metres
- V = velocity of flow in m/sec
- f_1 = coefficient for loss of head at entry taken as 0.505 for an unshaped mouth and 0.80 for bell mouth
- f_2 = $a(1 + b/R)$, where the values of a & b for different materials may be taken as follows :

Nature of surface of barrel	a	b
Smooth iron pipe	0.00497	0.025
Inscructed iron pipe	0.00996	0.025
Smooth cement plaster	0.00316	0.030
Ashlar or brickwork or planks	0.00401	0.070
Rubble masonry or stone pitching	0.00507	0.250

As far as possible the entry should be bellmouthed to reduce the entry losses.

Velocity in the barrel should be between 2.0 to 3.0 m/sec during design flood and 1.0 m during dominant discharge. The height of barrel shall not be kept less than 1.0 m from the considerations of manual clearing of deposits therein.

6.3.4 Fluming of the Canal Waterway

The flumed portion of the trough is joined with the normal section by providing minimum transition of 2:1 on the upstream side and 3:1 on the downstream side.

6.3.5 Uplift Pressure

(i) On the roof or under side of trough

When the drainage water level is higher than the underside of the trough, uplift pressure would be exerted on the trough carrying canal water.

The trough should be designed for two conditions :

(a) Full supply level in trough but no uplift pressure
(b) Minimum uplift pressure and no water in the trough.

(ii) On the floor of the syphon aqueduct

The floor of a syphon is subjected to uplift due to the following two causes:

(a) Sub-soil water table in the drainage bed. The maximum net uplift would occur when there is no drainage flow. The level difference between the sub-soil water level and the bottom of the floor would then represent the maximum static head.

(b) Seepage of water from the canal to the drainage. It is maximum when the canal is full and no water is flowing in the drain.

For works of small magnitude the Bligh's creeps theory may be used for assessment of uplift pressure under the floor. For works of large magnitude the pressure should be worked out by electric analogy method.

(iii) Uplift in syphon barrel

In case of barrel type drainage syphon, the barrel section should be checked against uplift for the following conditions :

(a) The canal is running at FSL barrel dewatered for repairs and drainage is dry. In this condition the total uplift is the difference between the FSL and the bottom of the barrel.

(b) The drainage running full at design flood level and canal head is dry.

(c) Canal is suddenly closed and the barrel is dry. In this case residual head varying from 50% to 80% of the head corresponding to canal FSL may be taken depending on the type of embankment soil.

(d) Experienced has shown that in situations where spring level is higher than the top of barrel, empty barrel with no earth load from top and sides shall be checked against flotation.

References:

1) Garg S.K., Irrigation Engineering and Hydraulic Structures

2) Varsaney, Theory and Design of Irrigation Structures

3) SRP, Design Criteria, BWDB System Rehabilitation Project, October, 1993

7.0 DESIGN CRITERIA FOR EMBANKMENT

7.1 General

For embankment design the major design parameter required is the selection of the magnitude of flood flows. Corresponding flood levels and their frequency of occurrence. Selection of the frequency of occurrence again depends on the degree of protection desired. Whatever may be the degree of protection, an embankment must be safe and stable during all phases of construction and operation.

To accomplish this, the design of embankment must fulfil the following two major criteria:

i) The embankment must not be overtopped during the passage of the design flood i.e., it should have sufficient freeboard.

ii) The body of the embankment must remain stable against external forces & foundation failure during normal and critical conditions of loading.

To meet these criteria the design of embankment cross-section may be divided into:

- selection of preliminary cross-section
- stability analysis of the cross-section

7.2 Types of Embankments

Embankments may be classified under three major heads depending on the nature of protection they provide and their locations:

- full flood protection embankments;
- submersible embankments;
- Sea dykes.

The full flood protection embankment is aimed at checking the highest flood flows across the embankment into the polder. The peak flood level is determined at selected return period depending upon the importance of the embankment.

Submersible embankments are usually built to afford protection during the pre-monsoon flood; thereafter they are overtopped and remain submerged during the monsoon. Sea dykes are usually built along the coastal belt to protect lands against flooding and intrusion of saline water. in addition, these embankments must give protection against tidal surges during cyclones.

7.3 Alignment of Embankment

The alignment of an embankment is governed mainly by technical, economical and morphological considerations. Economically the best alignment is that, which can be built as efficient and cheap as possible, requires least land acquisition, uses locally available suitable material and encloses as much land as possible. The following points have to be considered in fixing the alignment of an embankment:

i) set back should carefully be fixed considering scouring of the river bank.

ii) peat soil should be avoided from the alignment. If not possible peat soil should be removed or appropriate allowance for settlement in peat soil should be added to the height of the embankment.

iii) preferably alignment should pass over the land consist of fair portion of clay.

iv) alignment should pass the area where required earth are available for construction of embankment.

v) sharp corners in the alignment should be avoided as much as possible.

vi) due attention should be given for avoiding blockage of existing transportation system.

vii) alignment should run as far as possible along the highest ground & not across depressions.

7.4 Set Back

Set back is the space between actual river bank & river side toe of the embankment. Set back of an embankment is to be used on:

i) The foreland should be sufficient to allow for borrow pits on the riverside and berm on either side for embankment construction.

ii) At places where erosion has taken place regularly over past years, a set back may be based on this erosion rate. An extra margin, equivalent to 10 years of the present erosion rate, should be added to the minimum set back to be applied.

iii) Where embankments are to be provided on both sides of a river, the minimum set back shall be determined from the floodway requirement to pass the design flood under confined conditions.

iv) If the above ~~criteria~~ can not be attained, a minimum set back equal to 6.0 m (20 ft.) from the eroded bank may be kept but bank protection works are to be provided.

v) For sea dykes the foreland should be wide enough to be afforested with adequate plants to reduce wave actions during flood conditions.

7.5 Design Crest level

Design crest level is obtained by providing free board over the design flood level. A rational determination of freeboard requires a determination of the height and action of waves. The height of waves generated by winds of the surface of a large body of water depends on the wind velocity, the duration of wind, the fetch (the distance over which the wind can act on a body of water, being generally defined as the normal distance from the windward shore to the embankment being designed), depth of water and the width of the water surface. Upon contact with the face of the embankment, the waves move up the inclined plane and expend part of their energy in raising the water level.

The design crest level is composed of design flood level (H_{DES}), height of wind set up (H_W), height of wave runup (H_Z) and safety margin.

7.5.1 Height of Embankment

Height of embankment is the difference between the design crest level of the embankment and the average ground level on which the embankment is constructed.

7.5.2 Selection of Design Flood Frequency

The frequency of occurrence of floods that needs to be selected for the design of a particular embankment depends on the acceptable extent of damage by inundation in the locality. Considering likely agricultural damage, damage to important installations and loss of human lives, the following flood frequencies may be adopted:

- 1:20 years flood where agricultural damage is predominant;
- 1:100 years flood where loss of human lives, properties and installations are predominant. In general, embankment along Jamuna, Padma and Meghna rivers shall be designed with this return period.

7.5.3 Design Flood Levels

Having selected the flood frequency the design flood levels need to be assessed separately in two cases, as follows, depending upon whether embankments are to be provided on one bank or on both banks.

i) Embankment on one bank

Where an embankment needs to be constructed on one bank only, the design flood levels may be computed by frequency analysis of available annual maximum river level data for full flood protection embankments, and by frequency analysis of maximum river level data before some specified time during the year, e.g. 31 May, for submersible embankments.

ii) Embankments on both sides

Where embankments are to be constructed on both banks, the computation of design flood levels by frequency analysis of historical river level data at a particular location is not a logical approach as the recorded river levels are measured with overbank flow conditions. Accordingly, the design flood levels need to be computed from the design flood discharge under confined conditions. A model study may required to be taken up to determine the confinement effect.

7.5.4 Free Board

Free board (F_B) = height of wind setup (H_w) + height of wave runup (H_z) + safety margin

i) Wind set up (H_w)

The general formula for calculating the wind set up H_w(m) is

$$H_w = I_w \times l \times \cos \Phi$$

The wind set up gradient is given by

$$I_w = 4 \times 10^{-6} \frac{W^2}{gh}$$

Where,

W	=	wind velocity (m/sec)
g	=	acceleration due to gravity (9.81 m/sec²)
h	=	average water depth (m) along stretch l
I_w	=	gradient of wind set up
l	=	length of water area over which the wind is blowing (m)
Φ	=	angle at which wind approaches the coast

Wind set up develops with wind condition that last for at least 24 hours. A short storm of less duration will hardly generate any significant wind set up. The effect of wind setup may be included in the design calculations in case the water level data are from a temporary gauge with records of less than 10 years. Water level data from permanent gauge stations already includes effects of wind setup.

ii) Wave runup (H_z)

The main points to be considered are:

- o the force of breaking waves against the slope of an embankment causes erosion if no protection is provided.
- o the runup of waves against a slope might cause overtopping of the embankment if the crest level is not high enough.
- o wave height increases with the increase in fetch.

Wave runup Hz (m) is given by

$$H_z = 8fH \tan\alpha \cos\beta \, (1-B/L)$$

where,

f = constant
H = wave height (m)
α = angle of embankment side slope to horizontal
β = angle between direction of incident wave and normal to embankment
B = berm width (m)
L = wave length (m)

Wave height H is computed from

$$H = 0.0555\ (W^2F)^{1/2}$$

Where,

F = length of fetch (nautical miles)
w = wind speed (knots)

Wave length L is given by

$$L = T^2q/2\pi$$

Where,

q = constant
T = wave period (seconds)
$= 0.5\ (W^2F)^{1/2}$

It may be noted that:

(i) (1-B/L) may be omitted if no berm is provided.

(ii) f varies from 0.75 for very rough rip-rap to 1.25 for smooth asphalt concrete slopes. For turfed slopes, f = 1.1.

(iii) Berm are required for sea dykes and embankments exposed to very high waves. Berm are not economical for dykes subjected to small waves.

(iv) Berm should be provided on both sides of a closure to increase stability and to provide an increased factor of safety against erosion.

(v) The formula is valid for slopes with $1/8 < \tan\alpha < 1/3$ Under no circumstances should the freeboard be less than 0.90 m for a full flood protection embankment.

7.5.5 Crest width.

The crest width of the embankment should be selected on the basis of the following criteria:

i) crest width should not be less than 2.50 m.

ii) if the embankment is used as inspection road minimum crest width should be 4.30 m.

iii) if the embankment is used as road, width shall be selected based on the type of road structure + 1.00 m shoulder on both sides. (Reference Art 9.1.5.1).

7.5.6 Side Slope

The criteria for selection for side slopes shall be based on:

- embankment slopes should be stable against adverse seepage flow.
- embankment should be stable against shear failure through its base.

Generally side slopes for an embankment (except sea dyke) is 1V:2H on the country side and 1V:2H to 1V:3H on the river side unless stability considerations indicate that flatter slopes are required.

7.5.7 Berm and Borrowpits

To ensure least usage of valuable land, borrowpits are to be located on the river side. The berm i.e. distance between the river side toe of the embankment and the edge of borrowpit, is to be from 3.0 m to 10.0 m, depending upon the depth of borrowpit and height of embankment.

The depth of borrowpit should not exceed 2.0 m. a berm of not less than 6.0 m should be left between the edge of borrowpit and the river bank. To prevent development of flow concentrations during high river stages, cross-bunds perpendicular to embankment center line should be left in the borrowpit every 30 m measured along the embankment. The width of such bunds should be at least 6.0 m.

7.6 Design for section

The embankment section must be selected so that:

- countryside slope remains stable during steady seepage at design high flood level.
- riverside slope must be stable during rapid drawdown conditions where these prevail.
- phreatic line should be well within the downstream face so that no sloughing of the slope takes place and the factor of safety against boiling is not less than 1.5.
- foundation base is flat enough to ensure a suitable factor of safety with respect to induced shear stress and shear strength of the embankment fill.
- river side slope must be protected against erosion by wave action, and the crest and countryside slope must be protected against erosion by wind and rain.

For the proper design of an embankment section the following soil data will be needed.

- angle of internal friction (Φ)
- cohesion (C)
- compression index (C_c)
- permeability (k)

Soil sample should represent the foundation soil as well as borrow material.

7.7 Phreatic line or line of seepage

Line of seepage or phreatic line or saturation line is defined as the line within the embankment section below which there are positive hydrostatic pressure in the embankment. The hydrostatic pressure on the phreatic line is equal to the atmospheric pressure and hence, equal to zero.

It is essential to determine the position of phreatic line, as its position will enable the designer to determine the following things:

i) It gives a divide line between the dry and submerged soil. The soil above the seepage line will be taken as dry and the soil below seepage line shall be taken as submerged for computations of shear strength of soil.

ii) It represents the top streamline and hence helps in drawing the flow net.

For stability analysis of embankment, determination of phreatic line is the prime requirement. The phreatic line is assumed to be a base parabola.

The procedure for locating the phreatic line depends on the downstream slope angle (For details, see Bowles Chapter 9-5 and 9-6).

With reference to Fig. 7.1 associated parameters are described below;

Correction factor Δa is given by:

$$\Delta a = (a + \Delta a)\left(\frac{180 - \alpha}{400}\right)$$

where, $(a + \Delta a)$ & α are measured from the figure, knowing Δa the point K is plotted & the phreatic line BIK is completed.

In another way the point K can be located by finding the value of 'a' from the following equation: (α<30°)

$$a = \frac{b'}{\cos\alpha} - \sqrt{\left(\frac{b'}{\cos\alpha}\right)^2 - \left(\frac{H}{\sin\alpha}\right)^2}$$

where,

b', α & H are shown in the figure 7.2. α between 30° & 60°.

$$a = \sqrt{b^2 + H^2} - \sqrt{b^2 - H^2 \cot^2\alpha}$$

7.8 Uplift and seepage quantity

The method involves drawing a flow net for the embankment under the design flood condition and taking values of nf and nd. The number of flow channels and number of potential drops respectively, from the flow net.

i) Uplift

Maximum upward hydraulic gradient $i_{max} = \tan\alpha$

Maximum uplift force $F_u = \gamma_w \times i_{max} \times V$ (where $V = 1.0\ m^3$)

Buoyant weight of soil $W_1 = \gamma_{sat} - \gamma_w$

Factor of safety, $F.S = W_1 / F_u \nless 1.5$

ii) Seepage Quantity

It is assumed that the embankment soil is homogeneous and isotropic in nature i.e. horizontal permeability K_H = Vertical permeability K_v. First the flow net diagram of the embankment section is drawn by freehand sketching and making suitable adjustment and corrections until the flow lines and equipotential lines intersect at right angles. The seepage rate 'q' can be computed from the flow net, using Darcy's law. The required expression representing discharge passing through a flow net is given by,

$$q = \frac{KH}{N_d} N_f$$

where,

$$K = \sqrt{K_H \cdot K_V}$$

H = Total head loss
N_d = Total number of drops in the completed flow net.
N_f = Number of significant flow lines.

Limiting value of q should not exceed 1.0 m^3/day/m of embankment. Filter should be provided in discharge face if limiting value of q exceeds in any reach.

7.9 Slope Stability Analysis

An earth embankment usually fails, because of the sliding of a large soil mass along a curved surface. It has been established by actual investigation of slides of railway embankments in Sweden, that the surface of slip is usually close to cylindrical i.e. an arc of a circle in cross-section.

Generally the embankment slope stability is determined by the following methods:

i) Swedish Slip Circle Method
ii) Bishop's Method

For small types of project the Swedish Slip Circle and for bigger types and important projects Bishop's Method of computation should be used.

To locate the centre of the possible failure arc angle α & β are taken from Figure 7.3 and the point of intersection of the angles α & β is the possible failure angle. The earth mass within the arc is divided into six to twelve vertical segments called slices. These vertical are usually equally spaced. The forces between these slices are neglected and each slice is assumed to act independently as a vertical column of soil of unit thickness and width. The weight W of each slice is assumed to act at its centre. The weight W of each slice can be resolved into two components; a normal component (N) and a tangential component (T) such that

$$N = W \cos\alpha$$
$$T = W \sin\alpha$$

where α is the angle which the slope makes with the horizontal.

The factor of safety (F.S.) for the entire slip circle is then given by the equation.

$$F.S. = \frac{C.L. + \tan\Phi(\sum N - \sum U)}{\sum T}$$

where,

$\sum T$= summation of tangential forces of all the slices tending to produce movement of the soil along the circumference of the arc.
$\sum N$= Summation of normal forces for all the slices.
Φ = Angle of internal friction of the material used for the construction of the embankment.
L = Length of arc intersecting the embankment.
C = Cohesion per sft.
$\sum U$= Total pore water pressure on all the slices.

The usual factor of safety (F.S.) may be taken as 1.5.

To determine the most dangerous circle more slip circles are drawn taking centres around the previous centre and F.S. for each slip circle is determined as mentioned before. The circle which produces the minimum F.S. is the most dangerous circle. The minimum F.S. will dictate whether the assumed slope of the embankment needs further modification.

7.10 Settlement of Embankment

Settlement of the subsoil strata under the body of the embankment is given by

$$S = \frac{C_c H}{1 + e_0} \log_{10} \frac{P_0 + \Delta p}{p_0}$$

Where,

S = settlement
C_c = compression index
H = height from the G.L. to the centre of the compressible layer.
P_o = present overburden pressure
Δp = increase in pressure
e_o = initial void ratio

Δp at any depth can be obtained by using the chart in figure 7.4.

7.11 Slope Protection

To protect the side slope from rain & wave erosion turfing of the slope and the berm may be used by locally available grass species. Turfed slope surface will also increase the stability of the side slopes. Afforestation of the force land may also protect the embankment from wave erosion. Planting of tree should be done on the berm and not on the slope or crest of the embankment.

Where severe wave action or erosion due to current is anticipated, revetments may be provided to protect the side slopes.

7.12 Submersible Embankments

7.12.1 General

The Haor submersible embankments are designed to restrict flood waters from entering the sub-project until May 31st. The delay in flooding would enable farmers to harvest boro rice without the risk of crop damage.

7.12.2 Crest Level

Submersible embankments in haor areas are designed for a 1 in 10 year pre-monsoon flood expected to occur before May 31st. In addition, a nominal height of freeboard of about 0.30 m is provided. The freeboard is provided to account for possible increases in the pre-monsoon flood levels due to embankment confinement effect. To ensure access at any time, the operating decks of the regulators are designed above 1:20 year annual flood. The embankment crest should be gradually raised to the regulator deck level over a 50 m length on each side of the structure. This will prevent washout of the embankments adjacent to the structures.

7.12.3 Cross Section

The embankment will have a crest width (as defined at Section 7.5.5) and preferably 1V:3H side slopes. (It has to be sufficient for use as a local road during the dry season and strong enough to withstand submergence and overflowing which will take place in both directions).

7.13 Closure Dams

7.13.1 General

The criteria to be adopted for the design of closure dams depend on the type of river on which the same will be constructed. The river may be tidal or non-tidal. It may also be flowing or non-flowing or dry during construction period. The design methods for deciding the design crest level and section of closure are however, nearly the same as that of embankment as regards to its purpose and stability requirement. However, special requirements such as flatter slopes and/or berm with respect to its foundation stability needed when it is constructed on tidal or non-tidal flowing rivers having soft or weak foundation beds. The construction technique to be adopted to arrive the design section and height of the dams is also most important specially in case of tidal rivers.

7.13.2 Crest level

As the damage due to overtopping of embankments has serious consequences and is difficult to repair than repairing a breach in an embankment, the crest level of closure dam is to be raised 0.5 m above the adjacent embankments.

7.13.3 Design Cross-section

The crest width of major closure dam shall be 6.00 m except for minor one where the same may be kept to the design crest width of adjacent embankment. The side slopes of embankment depend on the slope stability and foundation stability which can be analyzed by the methods as stated under "Embankment" section. However, in tidal closure dam the R/S under water slope of 1:5 to 1:7 and C/S underwater slopes of 1:4 to 1:5 are usually adopted. Further, a berm of 5 m to 10 m is to be provided at the existing ground level at both sides of the crest to provide extra stability of dam and to serve as extra safety with respect to subsidence of the underwater slope which might occur during the settlement and shrinkage of the dam. Above the ground, the slopes may be kept to that of adjacent embankments.

7.13.4 Closure Method

The closure method may be horizontal, vertical or combination of horizontal and vertical. In tidal river, this choice of method depends on physical conditions at the specific site i.e.

- tidal condition
- tidal area/volume
- profile area/volume
- type of bottom material

7.13.5 Elevation and Sill length of Vertical Closure Method

For the calculation of the elevation and length of the sill of a vertical closure method, the following criteria shall be followed:

- the velocities of the water passing over the sill during extreme conditions (i.e. spring tide) should not exceed the maximum allowable velocity for the materials used in the sill.

- the crest elevation of the sill should be such that during low water stage, the sill is still submerged for 0.5-1.00 m. This is to prevent highly turbulent flow. The detail design procedures stated in Design manual of Delta Development Project shall be followed to design the elevation and sill of vertical closure design.

References:

1) Garg S.K., Irrigation Engineering and Hydraulic Structures

2) Varsaney, Theory and Design of Irrigation Structures

3) United States Department of the Interior, Design of Small Dams

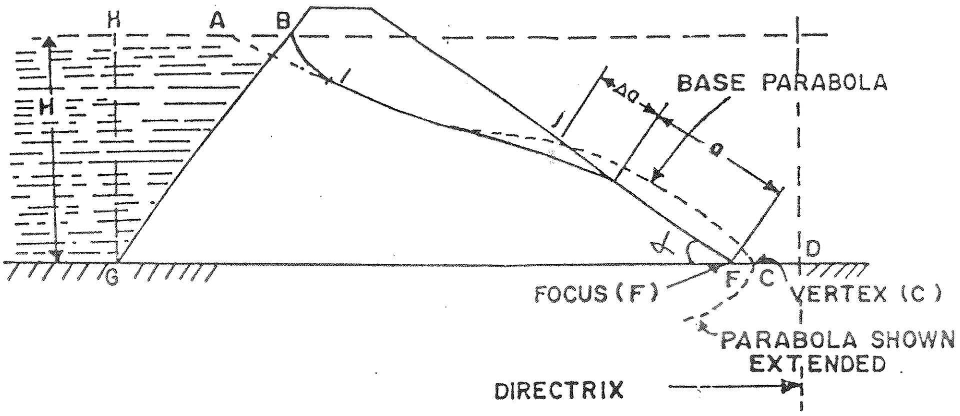


Fig. 7 · 1

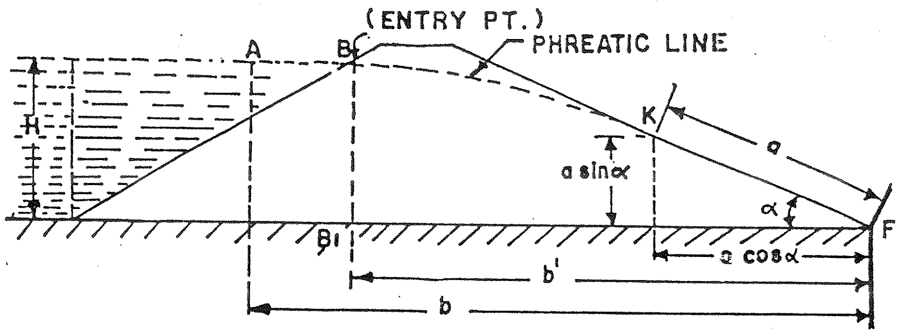


Fig. 7 · 2

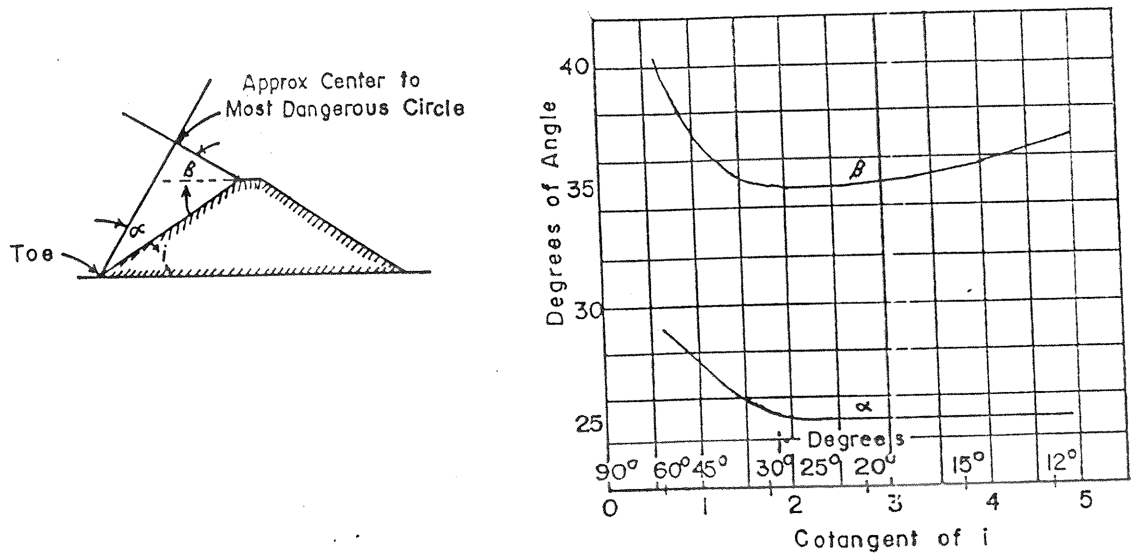
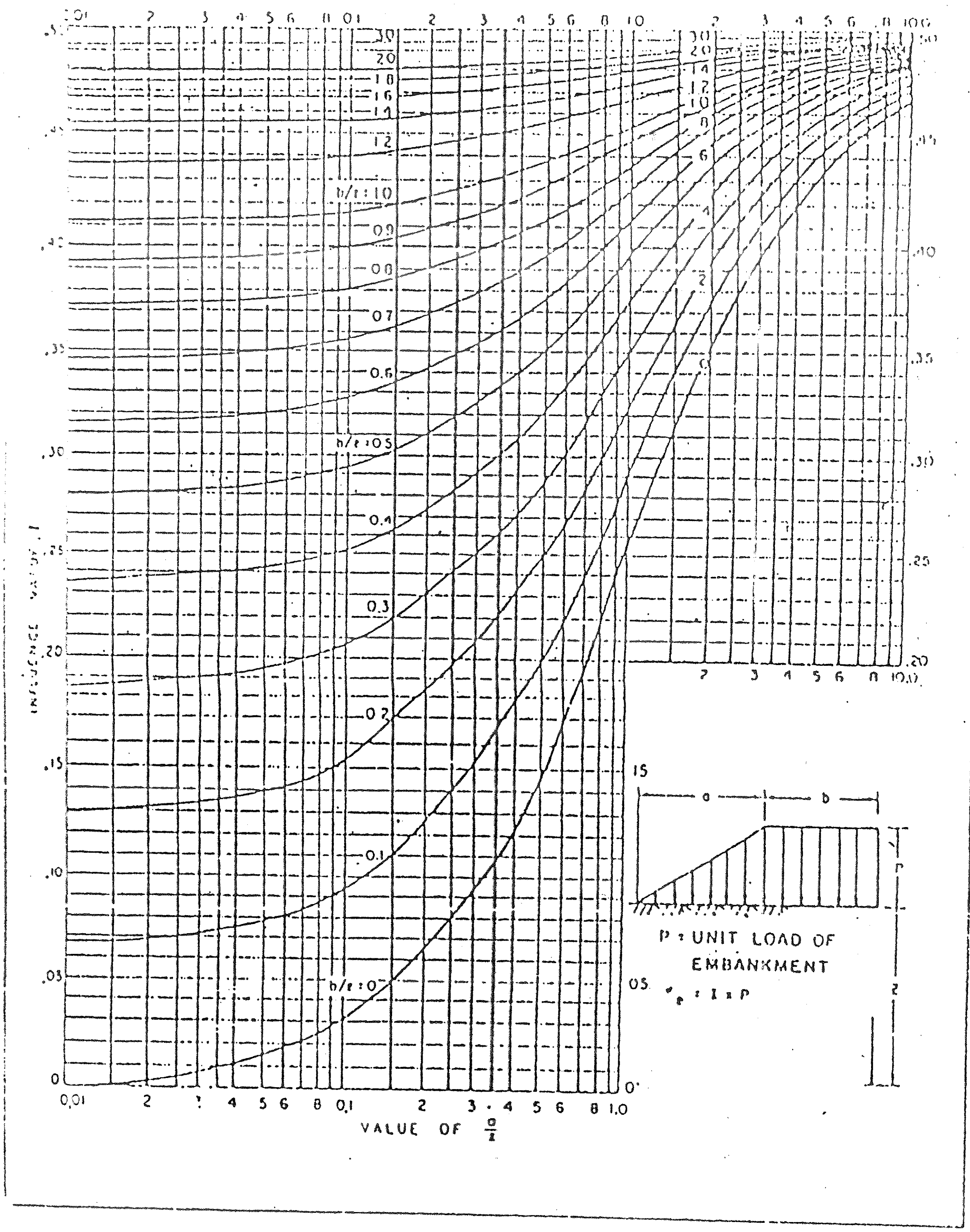


Fig.7·3 Angles and for different slopes to determind center of most dengerous circle which passes through toe .

Fig. 7·4



Influence Value for Vertical Stress under Embankment Load of Infinite Length

8.0 PIPE STRUCTURES

8.1 Structural Design of Pipe

The structural design procedure for the selection of a pipe culvert consists of the following steps:

- o Determination of Earth load & Live Load.
- o Selection of Bedding.
- o Determination of Load Reduction Factor.
- o Application of Factor of Safety.
- o Selection of Pipe

8.1.1 Determination of Earth Load

The earth or backfill load transmitted to a pipe culvert is largely dependent on the type of installation. The three common types of installation are Positive Projecting Embankment, Negative Projecting Embankment and Induced trench. These are shown in Figure 8.1.

Positive Projecting Embankment Installation

This type of installation is suitable for a relatively flat stream bed or drainage path. The pipe is installed on the natural ground or compacted fill and then covered by an earth fill or embankment.

The fill load, W_E, on a pipe installed in a positive projecting embankment is influenced by the settlement ratio rsd and the projection ratio p.

The settlement ratio r_{sd} expresses the relationship between the pipe deflection and the relative settlement between the prism of fill directly above the pipe and the adjacent soil. Recommended design values of settlement ratio r_{sd} are listed in Table 8.1. The projection ratio p is the height of the pipe projecting above the natural ground divided by the outside height (B_c) of the pipe as shown in Figure 8.1.

Table 8.1
Design Values of Settlement Ratio, r_{sd}

Installation and Foundation Condition	Settlement ratio, r_{sd}	
	Usual range	Design value
Positive Projecting Very firm soil *Ordinary soil Soft soil	 1.0 0.5 to 0.8 0.0 to 0.5	 1.0 0.7 0.3
Zero projection		0.0

* With good construction method resulting in proper compaction of bedding and sidefill materials the lower value of 0.5 may be used for r_{sd}.

The fill loads in KN per linear metre for various fill heights and pipe sizes are shown in Table 8.2. for different values of the production pr_{sd}.

Table 8.2

Fill load W_E in KN per Linear Metre for Circular pipe Culvert in Positive Projecting Embankment Installation

Inside dia of pipe mm	r_{sd}	Height of fill above crest, mm										
		600	900	1200	1500	1800	2100	2400	2700	3000	4500	6000
300	0.0	3.6	5.8	7.4	9..9	11.7	13.4	15.3	17.5	19.7	29.2	36.7
	0.1	4.5	6.0	9.9	12.0	14.3	16.8	19.0	21.9	24.1	36.6	48.1
	0.3	5.3	8.2	10.9	13.9	16.8	19.7	21..9	24.8	27.0	40.9	54.0
	0.5	5.6	8.5	11.7	14.6	17.5	20.4	23.3	26.3	29.2	43.8	51.4
	1.0	6.6	9.9	13.1	16.8	19.7	22.6	26.3	19.9	32.8	50.3	66.4
600	0.0	7.3	11.7	14.6	18.2	21.9	25.5	29.2.	32.8	36.5	54.0	73.0
	0.1	8.8	13.3	18.2	22.6	26.3	31.4	36.5	40.9	44.5	67.1	90.5
	0.3	9.0	13.9	20.4	26.3	31.4	36.5	40.9	46.7	51.1	77.3	102.1
	0.5	8.5	14.3	20.4	26.3	32.1	37.9	43.0	48.1	52.5	80.2	109.4
	1.0	8.0	13.9	20.4	26.3	36.5	42.3	48.1	54.0	60.5	93.4	124.0
900	0.0	10.9	16.0	21.9	27.0	32.1	37.2	43.0	48.1	53.3	80.2	108.0
	0.1	11.7	19.0	26.3	32.1	39.4	46.0	52.5	59.1	65.7	100.7	134.2
	0.3	12.0	19.0	27.0	36.5	43.8	52.5	51.4	65.7	73.7	110.2	153.2
	0.5	11.8	19.0	26.3	35.7	45.2	54.0	62.7	70.0	78.8	119.6	167.8
	1.0	12.4	17.5	27.0	35.7	45.2	55.4	67.1	80.2	89.0	135.7	189.7
1200	0.0	13.9	20.4	27.7	35.7	42.3	49.6	56.2	62.7	68.6	104.3	140.1
	0.1	15.3	24.1	34.3	43.8	51.1	60.6	68.6	78.8	86.1	131.3	151.0
	0.3	15.3	24.1	32.8	43.	53.3	67.1	75.9	87.5	94.8	145.9	197.0
	0.5	15.3	24.1	32.8	43.8	53.3	65.7	80.2	94.8	102.1	160.5	211.6
	1.0	15.3	24.1	33.6	43.8	54.0	65.7	80.2	93.4	109.4	175.1	240.7
1500	0.0	17.5	26.3	35.0	43.8	52.5	61.3	70.0	78.8	87.5	131.3	175.1
	0.1	18.2	29.2	39.4	51.8	64.2	75.1	84.6	95.6	105.1	167.8	218.9
	0.3	19.0	29.2	40.1	51.8	65.7	78.8	94.8	109.4	119.6	182.4	240.7
	0.5	19.0	29.2	39.4	51.1	64.2	77.3	90.5	105.1	124.0	189.7	248.0
	1.0	19.7	29.9	40.9	52.5	66.7	78.8	91.9	106.5	124.0	218.9	299.1

Note: Values of W_E are based on a unit weight of fill material of 1600 kg/m^3. For other unit weights adjust W_E proportionally, e.g. for fill weighing 1440 kg/m^3 reduce W_E by 10%, for 1760 kg/m^3 increase W_E by 10% etc.

8.1.2 Determination of Live Load

Live load considerations are necessary in the design of pipe culverts which are installed with shallow cover under unsurfaced highways or underways with flexible pavements. The effect of live load increases as the height of fill above the top of the pipe increases. It is recommended that at least 600 mm of fill must cover the pipe culvert at all times. Highway live loads, Wl in KN per linear metre, transmitted to a circular pipe culvert are presented in Table 8.3. for the AASHTO MS18 loading.

Critical loading configuration used in the Table are as follows:

For 600 m ≤ H ≤ 1200 mm two 72 KN dual-tired wheels in passing mode, spaced at 1200 mm centres.

For H > 1200 mm, alternate MS18 loading.

The passing mode is, however, possible only for the 7.32 m width of roadway and so the loads in Table 8.3. are conservative for the 3.66 m width of roadway.

Table 8.3

Highway Loads on Circular Pipes, W_L in KN per linear metre for AASHTO MS18 loading.

Inside diameter of pipe, mm	Height of fill H above top of pipe, mm							
	600	900	1200	1500	1800	2100	2400	2700
300	15.8	8.0	5.5	4.2	3.4	2.8	2.3	1.9
600	26.0	13.6	9.3	7.3	6.8	4.8	4.1	3.5
900	34.0	17.9	12.5	9.8	8.0	6.6	5.5	4.8
1200	36.2	21.4	15.2	12.0	9.8	8.2	6.9	6.0
1500	32.8	24.6	17.4	13.9	11.4	9.5	8.2	7.0

Note: 1. Minimum recommended earth fill on top of pipe is 600 mm.
2. Truck live loads for fill heights 3 m or more are insignificant.

8.1.3 Selection of Bedding

A bedding is provided to distribute the vertical reaction around the lower exterior surface of the pipe and reduce stress concentrations within the pipe wall. In view of the uncertain compaction attainable in the field with other types of bedding, it is recommended that Class A bedding with a concrete cradle be used as standard for the pipe culverts to be constructed. This standard bedding is shown in Figure 8.2.

8.1.4 Determination of Load Reduction Factor (L_f)

The laboratory strength of a pipe is usually determined by the so called three edge bearing test. This is a more severe loading condition than the in-place load on the buried culvert. The design of the pipe culverts presented in this Manual is based on a loading that simulates the three edge bearing test. The actual in-situ structural capacity of the pipe culvert is likely to be more than the designed ultimate three edge bearing loads presented in this Manual.

The anticipated fill and live loads transmitted to the culvert should therefore be reduced by suitable factors in order to arrive at an equivalent three edge bearing load. The load reduction factor, L_f, is the ratio of the strength of the pipe culvert under installed condition of loading and bedding to the designed strength of the pipe in the three edge bearing condition.

The recommended load reduction factors for fill load for a positive projecting embankment installation of circular pipe culverts are listed in Table 8.4 for the standard concrete cradle bedding. In the special case of a zero projecting embankment installation, i.e. when the project ratio p is equal to zero, L, may be taken as 2.83 for all values of H/B_c and pr_{sd}.

The live load reduction factor appropriate for an unsurfaced roadway or flexible pavement, has been fixed at 1.5 for the pipe culverts designed in this Manual.

Table 8.4

Load Reduction Factor for Earth Fill Load on Circular Pipes in Positive Projecting Embankment Installation (Concrete Cradle Bedding)

H/B_c	p = 0.9					p = 0.7				
	r_{sd}					r_{sd}				
	0	0.3	0.5	0.7	1.0	0	0.3	0.5	0.7	1.0
0.5	11.26	8.87	8.87	8.87	8.87	7.51	6.54	6.54	6.54	6.54
1.0	6.61	5.37	5.37	5.37	5.37	5.61	4.79	4.79	4.79	4.79
1.5	5.81	4.62	4.47	4.47	4.47	5.17	4.31	4.19	4.19	4.19
2.0	5.48	4.37	1.23	4.19	4.19	4.98	4.22	4.08	4.00	3.98
3.0	5.18	4.25	4.10	4.81	3.92	4.80	4.12	3.99	3.91	3.84
5.0	4.97	4.15	4.01	3.93	3.84	4.66	4.05	3.92	3.85	3.78
10.0	4.82	4.08	3.94	3.86	3.79	4.57	4.00	3.88	3.80	3.74
15.0	4.77	4.06	3.91	3.84	3.77	4.53	3.98	3.86	3.78	3.72
H/B_c	p = 0.5					p = 0.3				
	r_{sd}					r_{sd}				
	0	0.3	0.5	0.7	1.0	0	0.3	0.5	0.7	1.0
0.5	4.84	4.55	4.55	4.55	4.55	3.49	3.42	3.41	3.41	3.41
1.0	4.33	3.97	3.79	3.97	3.97	3.40	3.29	3.28	3.28	3.28
1.5	4.18	3.79	3.72	3.68	3.68	3.37	3.26	3.24	3.22	3.20
2.0	4.11	3.75	3.68	3.63	3.58	3.35	3.25	3.23	3.22	3.20
3.0	4.04	3.72	3.66	3.60	3.54	3.34	3.24	3.22	3.20	3.18
5.0	3.99	3.68	3.62	3.56	3.51	3.33	3.23	3.21	3.19	3.17
10.0	3.95	3.66	3.59	3.54	3.49	3.32	3.23	3.21	3.19	3.17
15.0	3.94	3.62	3.58	3.54	3.48	3.32	3.23	3.21	3.19	3.17

8.1.5 Application of Safety Factor

A factor of safety of 2.0 is recommended for the structural design of pipe culverts presented in this Manual.

8.1.6 Selection of Pipe Section

Standard designs of reinforced concrete pipe culvert presented in this Manual cover a range of inside diameters from 300 to 1500 mm. Culvert of each diameter has been designed with a number of wall thicknesses and within each wall thickness a variable amount of reinforcement has been used. This has resulted in a wide variety of designs each having a different strength. The ring and longitudinal reinforcements in a pipe section are shown in Figure 8.3.

The ultimate three edge bearing strengths of various pipe culvert sizes have been expressed in terms of so called D-loads in Table 8.5 through 8.9. The D-load concept provides strength classification of pipes independent of pipe diameter. For circular pipe culverts, the three edge bearing capacity in KN per linear metre equals D-load multiplied by inside diameter of the pipe in metre.

With a fill load W_E and a live W_L transmitted to the culvert, the required D-load capacity of a circular R.C. pipe culvert is expressed as:

$$\text{D-load} = [(W_L/1.5) + (W_E/L_f)(F.S/D)]$$

where F.S. is the factor of safety and D is the inside diameter of the pipe in metres.

8.1.7 Summary of Design Procedure

The structural design process for a circular R.C. pipe culvert consists of the following steps:

1) Determine fill load W_E transmitted to the culvert from Table 8.2, as described in Art. 8.1.1.

2) Determine live load W_E transmitted to the culvert from Table 8.3 as described in Art. 8.1.2.

3) Determine live load reduction factor. If from Table 8.4, as described in Art. 8.1.4.

4) Calculate required D-load from Art. 8.1.6, with a factor of safety F.S. = 2.

5) Select appropriate Table for the relevant pipe diameter from among Table 8.5 through 8.9. Read down the columns corresponding to various thicknesses in the table to obtain the D-load calculated in step 4 above. Select the corresponding thickness and reinforcement for the pipe culvert.

8.2 Pipe Joints

For ease of construction, transportation and placement, pipe sections are cast in short lengths. When laid end to end on the prepared bedding at the site they need to be jointed. Various methods of jointing ranging from simple collar joints to a more elaborate spigot groove and O-ring gasket are available. Some of the simpler joints suitable for smaller dia pipe and minor earthfilling over them are illustrated in Figure 8.4.

A precast concrete collar slipped over and lapping the pipe ends is illustrated in Figure 8.4(a). The collar is placed in position alongwith the pipes before the bed is cast, and is sealed with the pipe by means of mortar backing. When a better sealing of the joint is desired the spigot and socket joint shown in Figure 8.4(b) may be used. In this method one end of the pipe is formed into a socket using a special mould for casting and the joint is sealed by mortar packing. When the thickness of pipe is suitable, the tongue and groove joint shown in Figure 8.4(c) may be used. Each segment of pipe is cast with a 'tongue' formed at one end and a 'groove' at the other. The joint is sealed by a mortar or mastic packing.

For pipe diameter 300mm and above and earth filling of great depth, cast-in-situ RCC pipe collar, shown in Figure 8.5 shall be used.

8.3 Reinforcement

8.3.1 Circumferential Reinforcement

A line of Circumferential reinforcement for any given total area may be composed of two layers for pipe with wall thickness of less than 180 mm or three layers for pipe with wall thicknesses of 180 mm or greater. The layers shall not be separated by more than the thickness of one longitudinal plus 6 mm. The multiple layers shall be fastened together to form a single rigid cage. All other specification requirements such as laps, welds, and tolerances of placement in the wall of the pipe, etc. shall apply to this method of fabricating a line of reinforcement.

- Where one line of circular reinforcement is used, it shall be placed from 35 to 50% of the wall thickness from the inner surface of the pipe, except that for wall thickness less than 60 mm, the protective cover over the circumferential reinforcement shall be 20 mm.

- In pipe having two layers of circular reinforcement, each line shall be so placed that the protective cover over the circumferential reinforcement shall be 25 mm.

- The spacing of circumferential reinforcement in a cage shall not exceed 100 mm for pipe having wall thickness of 100 mm or less. For pipe having wall thickness larger than 100 mm, the spacing of circumferential reinforcement shall not exceed the wall thickness and in no case exceed 150 mm.

- If splices are not welded, the reinforcement lap not be less than 20 diameters for deformed bars and deformed cold-worked wire, and 40 diameters for plain bars and cold-drawn wire. In addition, where lapped cages of welded-wire fabric are used without welding, the lap shall contain a longitudinal wire.

- When splices are welded and are not lapped to the minimum requirements above, pull tests of representative specimens shall develop at least 50% of the minimum specified strength of the steel, and there shall be a minimum lap of 50 mm. For butt-welded splices in bars or wire, permitted only with helical wound cages, pull tests of representative specimens shall develop at least 75% of the minimum specified strength of the steel.

8.3.2 Longitudinal Reinforcement

Each line of circumferential reinforcement shall be assembled into a cage that shall contain sufficient longitudinal bars or members, extending through the wall of the pipe, to maintain the reinforcement rigidly in shape and in correct position within the form. The exposure of the ends of longitudinal, stirrups, or spacers that have been used to position the cages during the placement of the concrete shall not be a cause for rejection.

The longitudinal reinforcement in pipe shall not be less than 0.0025bt, b = width considered, t = thickness of pipe.

8.3.3 Joints

The joints shall be of such design and the ends of the concrete pipe sections so formed that when the sections are laid together they will make a continuous line of pipe with a smooth interior surface free from appreciable irregularities in the flow line.

Joints may be of two major types such as Non-Rubber Gasket Joints and Rubber Gasket Joints. The most type in current use is Non-Rubber Gasket Joint between the concrete pipe without bell and spigot. The following sections shall deal with face to face non-rubber gasket joints of concrete pipes.

8.3.4 Joint Reinforcement

Broadly there shall be two types of joints based on pipe diameter. For pipes having diameter of upto 225mm, the joints shall be made of standard pre-cast reinforced concrete sockets. The length of the socket shall be not less than 125mm with 50mm wall thickness. The socket shall be provided with 3 nos circular reinforcement and 5 nos longitudinal reinforcement of 6mm dia M.S.bar equally spaced. The gap between the two pipe ends at joints shall be not more than 10mm. The entire joint assembly shall be covered by adequate mass concrete.

For pipes having diameter larger than 225mm but upto 1350mm, the joints shall be made of cast in situ reinforced collar. The length of the collar shall be not less than 200 mm and of rectangular or square shape according to requirement. The thickness of the collar along the pipe diameter in any direction shall not be less than 100 mm. The gap between the two pipe ends shall not be less than 10 mm. The typical reinforcement details of collar has been shown in volume-II of this manual.

8.4 Foundation Design

The invert of the box culvert shall be set 0.3m below the existing or design bed level of the channel. The foundation of the box culvert shall be analyzed as a raft as far as bearing capacity and settlement are concerned.

Table 8.5

Values of Ultimate D-load Capacity for Circular R.C. Pipe Culverts in KN/m/m inside diameter.

Inside diameter = 300 mm

Ring Reinforcement	Wall thickness, mm		
	50	75	100
D6 @ 75 c/c	145.2	235.2	271.0
D6 @ 100 c/c	115.5	182.2	252.6
D6 @ 125 c/c	95.6	150.4	203.9
D6 @ 150 c/c	81.2	127.5	172.9
D6 @ 175 c/c	70.3	110.4	149.2
D6 @ 200 c/c	63.2	97.1	129.3
D10 @ 75 c/c	-	-	621.2
D10 @ 100 c/c	-	349.4	500.4
D10 @ 125 c/c	-	295.9	414.3
D10 @ 150 c/c	155.5	254.4	355.1
D10 @ 175 c/c	138.3	224.0	310.3
D10 @ 200 c/c	125.1	199.7	274.6
Longitudinal Reinforcement	D6 @ 250 c/c	D6 @ 150 c/c	D6 @ 125 c/c

Table 8.6

Values of Ultimate D-load Capacity for Circular R.C. Pipe Culverts in KN/m/m inside Diameter.

Inside Diameter = 600 mm

Ring Reinforcement	Wall thickness, mm				
	50	75	100	125	150
D6 @ 75 c/c	36.3	58.8	81.0	104.3	127.4
D6 @ 100 c/c	28.9	45.6	63.2	79.6	97.2
D6 @ 125 c/c.	23.9	37.6	51.8	64.5	77.3
D6 @ 150 c/c	20.3	31.9	43.2	53.7	65.9
D6 @ 175 c/c	17.6	27.6	37.3	47.4	-
D6 @ 200 c/c	15.8	24.3	32.3	41.1	-
D10 @ 75 c/c	-	-	155.3	205.5	254.5
D10 @ 100 c/c	-	87.3	125.1	161.9	199.8
D10 @ 125 c/c	-	74.0	103.6	133.4	162.5
D10 @ 150 c/c	38.9	63.6	88.8	112.8	137.9
D10 @ 175 c/c	34.6	56.0	77.6	98.6	118.9
D10 @ 200 c/c	31.3	49.9	68.6	87.0	106.0
Longitudinal Reinforcement	D6 @ 250 c/c	D6 @ 150 c/c	D6 @ 125 c/c	D6 @ 100 c/c	D6 @ 75 c/c

Table 8.7

Values of Ultimate D-load Capacity for Circular R.C. Pipe Culverts in KN/m/m inside Diameter.

Inside Diameter = 900 mm

Ring Reinforcement	Wall thickness, mm				
	50	75	100	125	150
D6 @ 75 c/c	16.1	26.1	36.0	46.4	56.6
D6 @ 100 c/c	12.8	20.2	28.0	35.4	43.2
D6 @ 125 c/c	10.6	16.7	22.6	28.7	34.4
D6 @ 150 c/c	9.0	14.2	19.2	23.8	29.3
D6 @ 175 c/c	7.8	12.3	16.6	21.1	-
D6 @ 200 c/c	7.0	10.8	14.4	18.2	-
D10 @ 75 c/c	-	-	69.0	91.4	113.1
D10 @ 100 c/c	-	38.8	55.6	72.0	88.8
D10 @ 125 c/c	-	32.9	46.0	59.1	72.2
D10 @ 150 c/c	17.3	28.3	39.5	50.2	61.3
D10 @ 175 c/c	15.4	24.9	34.5	43.8	52.8
D10 @ 200 c/c	13.9	22.2	30.5	38.7	47.1
Longitudinal Reinforcement	D6 @ 250 c/c	D6 @ 150 c/c	D6 @ 125 c/c	D6 @ 100 c/c	D6 @ 75 c/c

Table 8.8

Values of Ultimate D-load Capacity for Circular R.C. Pipe Culverts in KN/m/m Inside Diameter.

Inside Diameter = 1200 mm

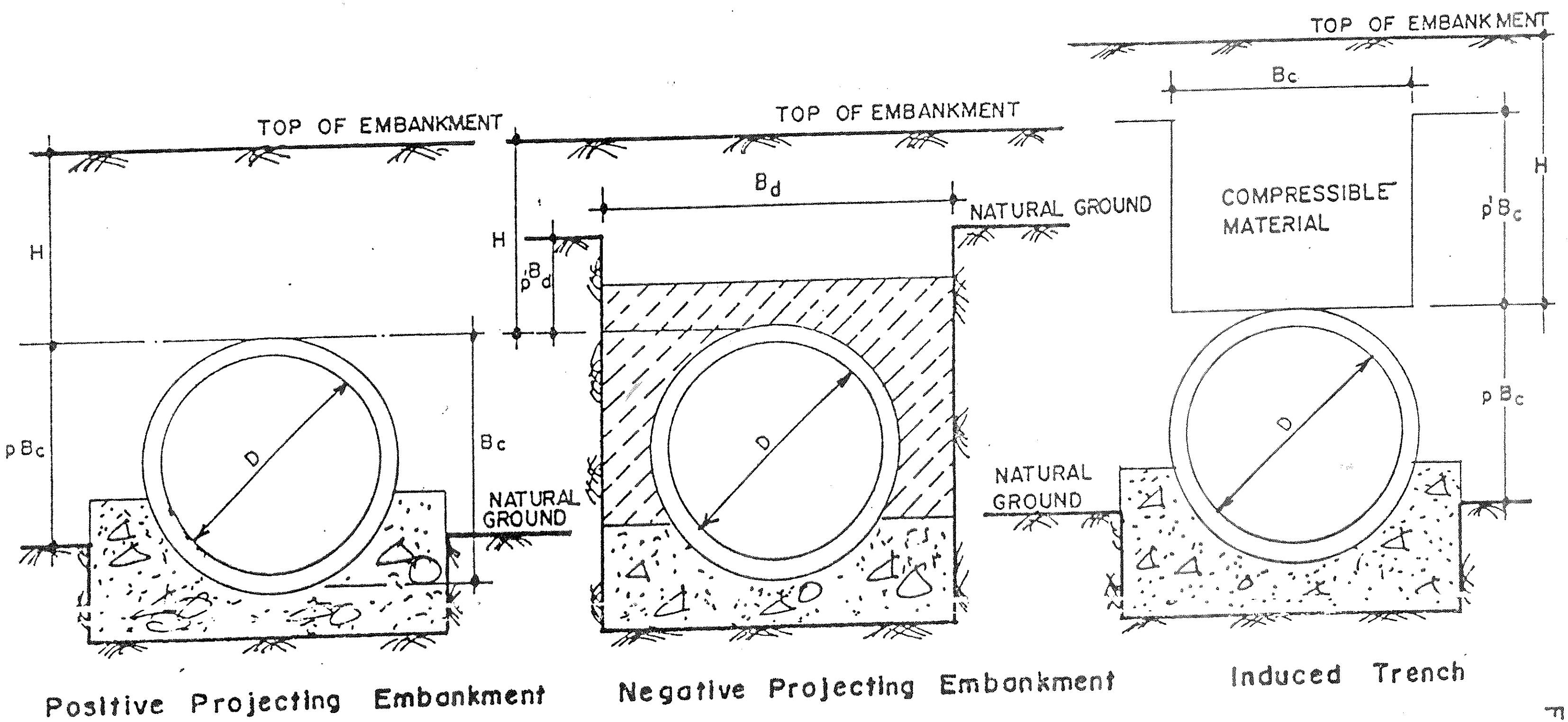
Ring Reinforcement	Wall thickness, mm				
	50	75	100	125	150
D6 @ 75 c/c	9.1	14.7	20.2	26.1	31.9
D6 @ 100 c/c	7.2	11.4	15.8	19.9	24.3
D6 @ 125 c/c	6.0	9.4	12.7	16.1	19.3
D6 @ 150 c/c	5.1	8.0	10.8	13.4	16.5
D6 @ 175 c/c	4.4	6.9	9.3	11.8	-
D6 @ 200 c/c	3.9	6.1	8.1	10.2	-
D10 @ 75 c/c	-	-	38.8	51.4	63.6
D10 @ 100 c/c	-	21.8	31.3	40.5	49.9
D10 @ 125 c/c	-	18.5	25.9	33.4	40.6
D10 @ 150 c/c	9.7	15.9	22.2	28.2	34.5
D10 @ 175 c/c	8.7	14.0	19.4	24.6	29.7
D10 @ 200 c/c	7.8	12.5	17.2	21.7	26.5
D12 @ 100 c/c	-	-	-	-	81.6
D12 @ 125 c/c	-	-	-	56.2	68.3
D12 @ 150 c/c	-	-	-	47.4	58.8
D12 @ 175 c/c	-	-	-	41.7	51.4
D12 @ 250 c/c	-	-	-	37.3	45.6
Longitudinal Reinforcement	D6 @ 250 c/c	D6 @ 150 c/c	D6 @ 125 c/c	D6 @ 100 c/c	D6 @ 75 c/c

Table 8.9

Values of Ultimate D-load Capacity for Circular R.C. Pipe Culverts in KN/m/m Inside Diameter.

Inside Diameter = 1500 mm

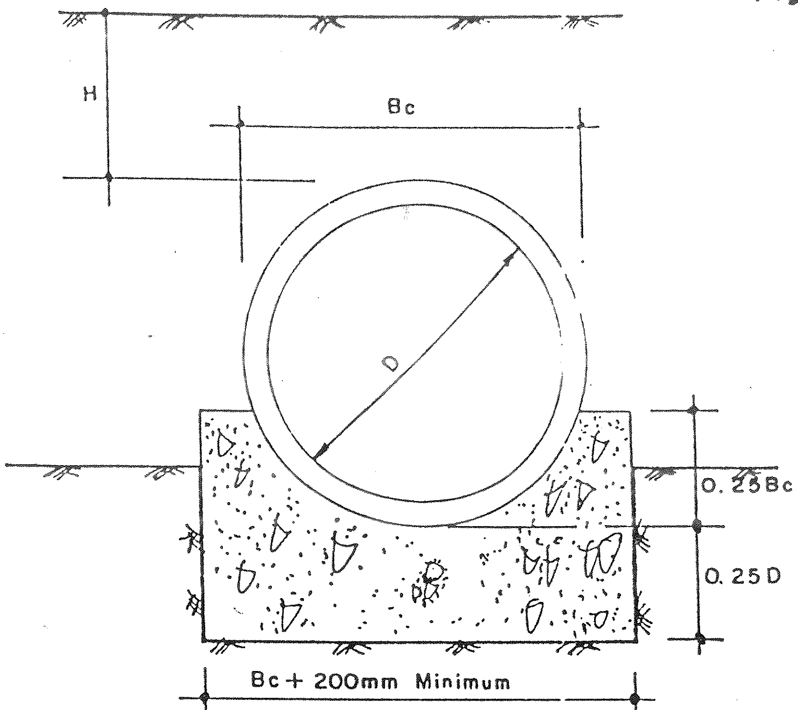
Ring Reinforcement	Wall thickness, mm				
	50	75	100	125	150
D6 @ 75 c/c	5.8	9.4	13.0	16.7	20.4
D6 @ 100 c/c	4.6	7.3	10.1	12.7	15.6
D6 @ 125 c/c	3.8	6.0	8.1	10.3	12.3
D6 @ 150 c/c	3.3	5.1	6.9	8.6	10.5
D6 @ 175 c/c	2.8	4.4	6.0	7.6	-
D6 @ 200 c/c	2.5	3.9	5.2	6.6	-
D10 @ 75 c/c	-	-	24.8	32.9	40.7
D10 @ 100 c/c	-	14.0	20.0	25.9	32.0
D10 @ 125 c/c	-	11.8	16.6	21.3	26.0
D10 @ 150 c/c	6.2	10.2	14.2	18.0	22.1
D10 @ 175 c/c	5.6	8.9	12.4	15.8	19.0
D10 @ 200 c/c	5.0	8.0	11.0	13.9	16.9
D12 @ 100 c/c	-	-	-	-	52.3
D12 @ 125 c/c	-	-	-	35.1	43.7
D12 @ 150 c/c	-	-	-	30.3	37.7
D12 @ 175 c/c	-	-	-	26.7	32.9
D12 @ 200 c/c	-	-	-	23.9	29.2
Longitudinal Reinforcement	D6 @ 250 c/c	D6 @ 150 c/c	D6 @ 125 c/c	D6 @ 100 c/c	D6 @ 75 c/c



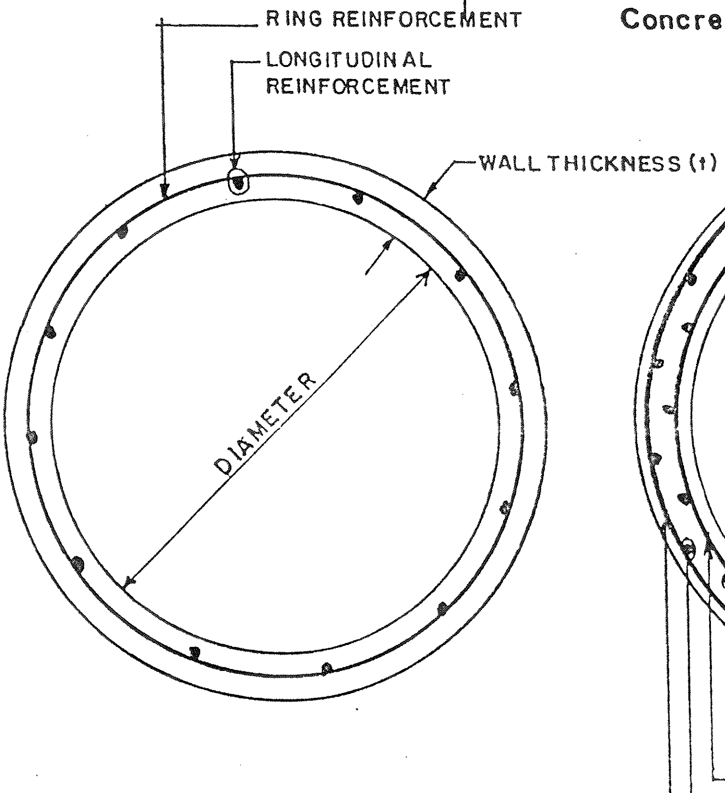
Various Installation Conditions

Figure 8.1

Figure 8.2

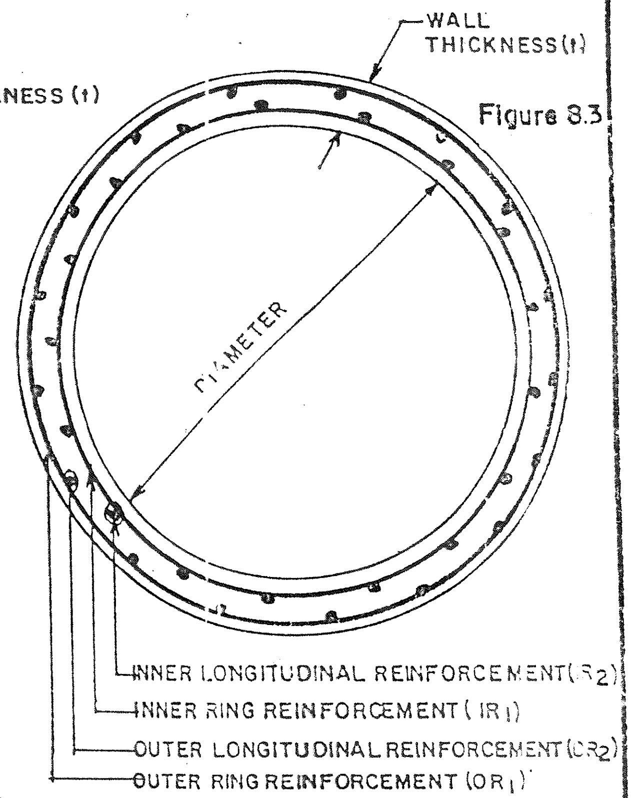


Concrete cradle bedding for pipe culvert



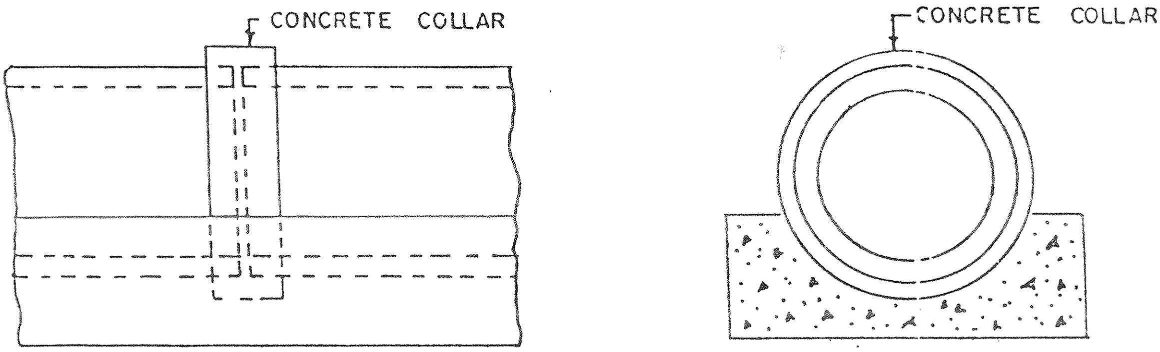
Sec. of a pipe with single layers of Ring and Longitudinal Reinforcement

Figure 8.3

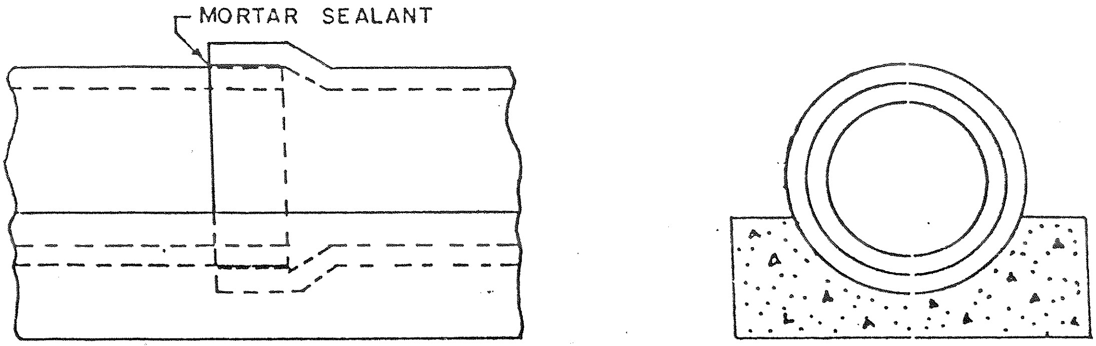


Sec. of a pipe with 2 (Two) layers of Ring and Longitudinal Reinforcement

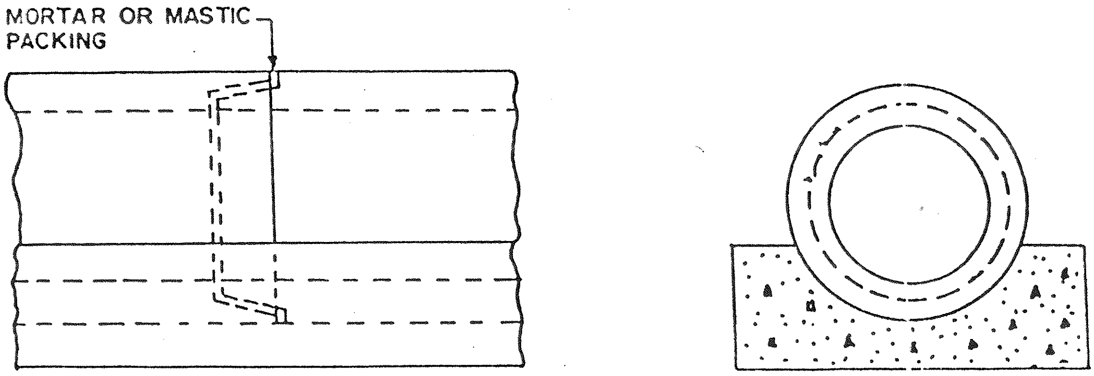
Figure 8.4



(a) Concrete Collar Joint



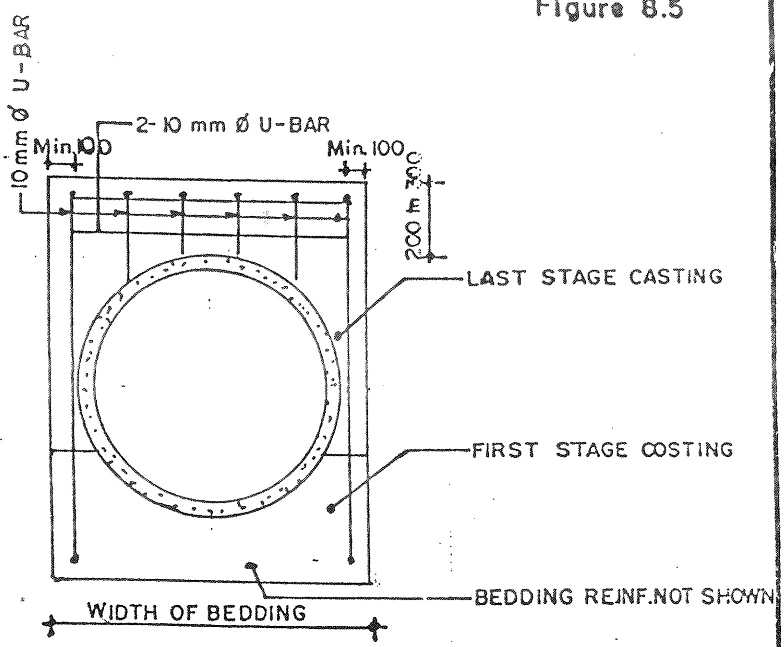
(b) Spigot and Socket Joint.



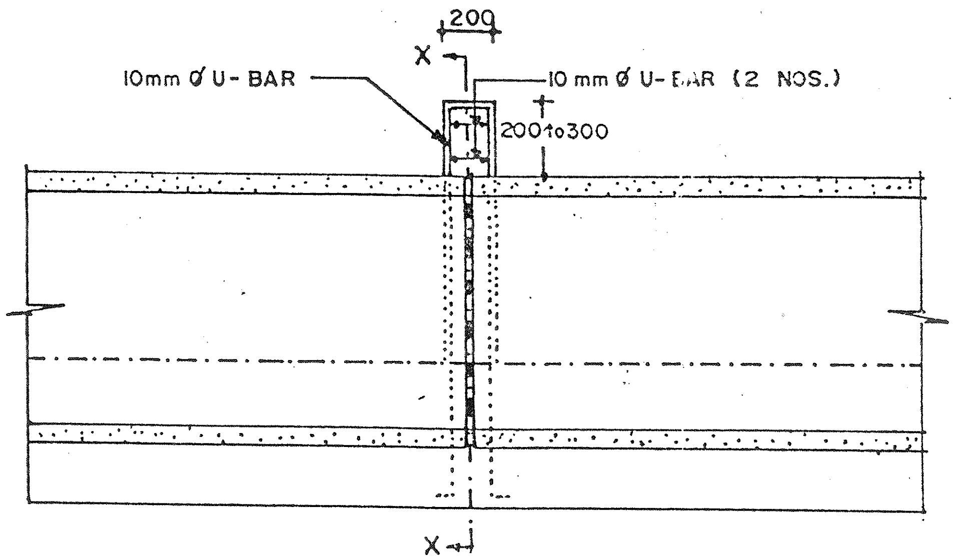
(c) Tongue and Groove Joint

Pipe joints

Figure 8.5



Section X-X



Details Of Cast-in Situ Rcc Pipe Collar Joint.

9.0 ROAD STRUCTURES

9.1 Bridges

9.1.1 Types of Bridge

According to loading, the bridge may be classified into the following four types.

- Type I : Pedestrian Bridge
- Type II : Light Traffic Bridge
- Type III : Medium Traffic Bridge
- Type IV : All Traffic Bridge

According to construction, types of bridge are as follows:

- Slab Bridge
- Simply supported RCC or PCC Girder Bridge
- Balanced Cantilever Bridge
- Continuous Span Bridge

The design criteria for slab bridges and simply supported RCC girder bridges have been described in the following.

9.1.2 Design Code of Practice

In the main, the design code of practice of the American Association of State Highway and Transportation Officials (AASHTO) has been followed. In some cases the code of practice of the Indian Road Congress (IRC) has also been followed.

9.1.3 Hydrological Analysis

9.1.3.1 Design Discharges

The design discharge for which the water way of the bridge is to be designed shall be as follows:

- For irrigation channels, the design discharge shall correspond to the full supply level of the channel.
- For drainage channels, the design discharge shall be taken as 1.5 times the design discharge.

9.1.3.2 Design Water Level

The design water level shall be determined with reference to the selected design discharge as mentioned above.

9.1.4 Hydraulic Analysis

9.1.4.1 Linear and Effective Linear Waterway of Bridge

The linear waterway of a bridge is the total length between faces of abutment. Effective linear waterway is the linear waterway of the bridge minus the width of any obstruction. (e.g. intermediate piers)

The linear and effective water way of a bridge may be determined as follows:

- For artificial channels (irrigation, drainage) the effective linear water way shall generally be such as to pass the full discharge at normal velocity.
- For natural channels in alluvial beds and having undefined banks, the effective linear waterway shall be determined from Lacey's regime formula as below.

$$L = CQ^{1/2}$$

where, L = regime width in metre.
Q = the design discharge in m^3/sec.
C = a constant usually taken as 4.8 for regime channels but it may vary from 4.5 to 6.3 according to local conditions.

In determining the ultimate waterway design, for each of the possible alternatives, consideration should be given to the amount of upstream afflux caused, any possible scour of bridge foundations, the height of abutments required and the overall cost of the bridge.

9.1.4.2 Scour Depth

The following theoretical method may be used to compute the scour depth of a natural channel in alluvial soil and where the effective linear waterway provided is not less than the regime width:

$$R = 0.47\ (Q/f)^{1/3}$$

where, R = normal scour depth in metres below the flood level corresponding to the value of Q adopted.
Q = the discharge adopted for the design (m^3/sec).
f = Silt factor.

The value of 'R' shall be suitably increased if the effective linear waterway (L) provided is less than the regime width (W), by multiplying by the factor $(W/L)^{2/3}$.

The maximum scour depth is generally taken as 1.5 times the normal scour depth for a single span bridge (with no piers) on a straight reach of river. A factor of 2.00 is used for multi span structures, or more complex approach conditions.

9.1.4.3 Afflux

The construction of wells and piers within the stream will obstruct the flow and thus produce an afflux or backwater effect which may be calculated by following the method described in "Hydraulics of Bridge Waterways" of Bureau of Public Roads, U.S. Dept. of Commerce.

9.1.4.4 Velocity Due to Obstruction

The increase in mean velocity due to obstruction of wells and piers may be calculated from the following formula:

$$V_1 = (2gh + \mu V^2)^{1/2} ,$$

where, V_1 = increase in velocity in m/sec.
v = average velocity upstream in m/sec.
μ = Kinetic energy coefficient 1.00.
h = afflux in metre.

9.1.5 General Features of Design

9.1.5.1 Geometry of Bridge Deck

Width of roadway

The width of roadway shall be the clear width measured at right angles to the longitudinal centre line of the bridge between the bottoms of the curbs. The following widths of roadway shall be followed:

Pedestrian bridge	: 1.85 m
Light Traffic bridge	: 2.75 m
Medium Traffic bridge	: 3.70 m
All Traffic bridge	: 3.70 m & 7.30 m

Side walk and Curb

The width of the sidewalk shall be clear width, measured at right angles to the longitudinal centre line of the bridge from the extreme inside portion of the railing to the top of the curb. Where sidewalks are warranted for pedestrian traffic, the width shall not be less than 45 cm. The face of the curb is defined as the vertical or sloping surface on the roadway side of the curb. The height of the bridge curb above the roadway shall not be less than 200 mm and preferably not more than 250 mm.

Railing

The material of the railings shall be concrete except where specifically required otherwise. The traffic railing shall be no less than 700 mm in height measured from the top of the roadway. The minimum height of a pedestrian railing shall be 900 mm measured from the top of the walkway to the top of the upper rail member.

Typical bridge deck sections are shown in Figure 9.1 and Figure 9.2.

Navigation Clearance

Where boat traffic is allowed, the minimum clearance measured from the design discharge level to the lowest point of the deck structure shall not be less than 1.50 m.

Free Board

Where no boat traffic is allowed, the free board shall not be less than 600 mm. The free board from design water level is to be measured to:

- the soffit of the girder in girder bridge.
- the soffit of the slab in slab bridge and culvert
- the bridge bearings in curved beams soffits of balanced cantilever bridge.

9.1.5.2 Roadway Drainage

The transverse drainage of the bridge roadway shall be achieved by means of a suitable camber in the roadway surface; and longitudinal drainage should be accomplished by camber or gradient. Over hanging portions of concrete decks should be provided with drip bead or notch.

9.1.5.3 Expansion Joints

To provide for expansion and contraction movements, floor expansion joints of not less than 20 mm shall be provided at the expansion ends of all spans and at other joints where they may be necessary. Typical details of joint are shown in Figure 9.3.

9.1.5.4 Diaphragms

In deck girder bridges, diaphragms shall be provided between the girders at the ends of bridge to provide lateral support. Intermediate diaphragm is provided at the centre of the span exceeding 12 m. The diaphragms are usually 250 mm to 300 mm thick with a nominal amount of reinforcement. The bottom level of diaphragms are to be set 100 mm to 150 mm above the bottom of girder.

9.1.5.5 Bearings

For slab bridges where span length is limited to 5 m, no special bearings are necessary. However, to ensure separation of the slab from the abutment seating a polythene sheet (minimum 2 layers) is to be provided on the seat before casting the slab. 20 mm Φ dowel bars shall be provided projecting vertically from the abutment at 600 mm centres as shear keys.

For R.C.C. girder bridges with lengths ranging from 5.00 m to 10.00 m, fixed bearings with steel plates can be safely used at both ends of span. One dowel bar of 25 mm for each girder shall be provided which will project vertically from the abutment. Typical details of bearings are shown in Figure 9.4 and 9.5.

For R.C.C. girder bridges with lengths ranging from 10 m to 15 m, a fixed bearing at one end and an expansion bearing at the other shall be provided. For the fixed arrangement, with no movement a vertical bar of 25 mm Φ is to be placed inside the girder and extended into the abutment. For an expansion bearing arrangement a lead sheet plate is to be placed between the two plates that are fixed to the girder and abutment, and no vertical bar should be included. Neoprene bearing may also be used.

9.1.6 Bridge Approach Gradient

Where the bridge is required to be raised above road level for navigational requirements, maximum upgradient towards the bridge or maximum down gradient away from the bridge shall not be greater than 3%.

The bridge shall be at the crest of the vertical curve joining up and down gradients and with a maximum rate of change of gradient of 6% per chain (30 m or 100 ft)

9.1.7 Loading

Structures shall be designed and proportioned for the following loads and forces where they exist. Other loads such as buoyancy, current pressure etc. have not been considered for bridges having only short spans.

- o Dead load
- o Live load
- o Impact or Dynamic effect of the live load
- o Wind loads
- o Longitudinal forces
- o Earth Pressure

9.1.7.1 Dead Load

The following unit weights shall be used in computing the dead load.

Structural Concrete	: 23.60 KN/M^3
Plain Concrete	: 22.60 "
Brick Masonry	: 18.90 "
Water	: 9.80 "
Moist Earth	: 18.10 "

9.1.7.2 **Live Load**

(a) *Roadway loading*

The live load shall consist of the weight of the applied moving load of vehicles appropriate to the following types of roadway (on which basis the bridges shall have been planned).

Type of roadway	Type of bridge	Loading
Village footpath	Pedestrian bridge	4.00 KN/m^2
Village Road	Light Traffic Bridge	H_{10}
Feeder Road	Medium Traffic Bridge	H_{15}
District Council Road and Highway	All Traffic Bridge	H_{20}, $H_{20}S_{16}$

(b) *Sidewalk loading* : Sidewalk floors shall be designed for a live load of 4.00 KN/m^2.

(c) *Curb loading* : Curb shall be designed to resist a lateral force of not less than 7.30 KN/m applied at the top of the curb.

(d) *Railing loading* : Railing loading shall be as per AASHTO code of practice.

Where H_{10}, H_{15}, H_{20}, $H_{20}S_{16}$ Truck loading refer to AASHTO code of practice.

9.1.7.3 Impact

The impact amount is expressed as a fraction of roadway live load stress and shall be determined by the Formula

$$I = \frac{15.24}{L+38} \quad (L \text{ in metre}),\ I \le 0.30$$

(Where I is the factor to be applied to the live loading or stress to assess the effect of impact)

9.1.7.4 **Wind Loads**

For the standard design of girder and slab bridge, the following wind load forces for a wind velocity of 160 Km/hour (100 miles/hour) may be used for the design of the substructure. The forces are:

<u>Wind load on Structure</u>

2.39 Kn/m^2, transverse
0.575 Kn/m^2, longitudinal

Both forces shall be applied simultaneously

<u>Wind loading on Live load</u>

1.46 Kn/m, transverse
0.584 Kn/m, longitudinal

Both forces shall be applied simultaneously, and be considered as additional to any worst loading combination, as described at 7.1.10 below.

9.1.8 **Longitudinal Forces**

Provision shall be made for the effect of a longitudinal force of five percent of the live load in all lanes carrying traffic travelling in the same direction. The centre of gravity of the longitudinal force shall be assumed to be located 1.80 m above the floor slab and transmitted to the sub-structure through the superstructure.

The longitudinal forces due to friction at the expansion bearings shall also be provided for in the design.

9.1.9 Earth Pressure

Structures which retain earth fill shall be designed and proportioned to withstand earth pressure as given by Rankine's formula.

A live load surcharge equal to not less than 0.60 m of earth shall also be considered. Where an adequately designed RCC approach slab supported at one end by the bridge is provided, no live load surcharge need be considered, (but the appropriate loads would have to be considered in the relevant structural design).

9.1.10 Load Combinations

The following loading conditions and combinations shall be considered in analysis and design of the various structural members of the bridge structure.

Structural Member	Loading or Combination of Loading
(a) Railing	Dead load + Live Load
(b) Post	Live load
(c) Curb	Live load
(d) Sidewalk	Dead load + Live load
(e) Deck Slab (Slab Bridge)	Dead load + Live load + Impact
(f) Deck Slab (girder Bridge)	<u>Case-1</u> Dead load + Live load on side walk and Railing only <u>Case-2</u> Dead Load + Live load + Impact on roadway only
(g) Girder	Dead load + live load + Impact
(h) Abutment	<u>Case-1</u> Dead load + Earth Pressure <u>Case-2</u> Dead load + Live load + Earth Pressure
(i) Wingwall	Dead load + Earth Pressure
(j) Pier	<u>Case-1</u> Dead load + Live load + Impact <u>Case-2</u> Dead load + Wind load <u>Case-3</u> Dead load + Live Load + Impact + 30% Wind load + Wind load on Live load + Longitudinal force from Live load.

For Case-3 loading, the pier shall be designed with 33% increase of allowable stress.

9.1.11 Distribution of Loading

Effective Span Length

For simple spans, the effective span length shall be the distance between centre of supports but not to exceed the clear span plus thickness of slab.

For slabs continuous over two supports and monolithic construction, clear span shall be used in calculating distribution of loads and moments.

Edge distance of wheel load

In designing slabs the center line of the wheel load (half of axle load) shall be assumed to be 0.30 m from the face of the curb. If curbs or side walks are not used, the wheel load shall be assumed to be 0.30 m from the face of rail.

In designing sidewalks, slabs and supporting members, a wheel load located on the sidewalks, shall be assumed to be 0.30 m from the face of the rail. Combined stresses arising from dead load, live load and impact load for this loading shall not be greater than 150 percent of the allowable stresses.

Bending Moment

For girder bridges, with the deck slabs spanning between two main girder supports and where main reinforcement is placed perpendicular to traffic, the live load bending moment per metre width of slab shall be determined by the following formula:

$$B.M.\,(KN\text{-}m/m) = \frac{(S+0.61)P}{9.74}$$

For girder bridge deck slab, continuous over three or more supports and where reinforcement is placed perpendicular to traffic, the following formula shall be applied for both positive and negative live load bending moment:

$$B.M.\,(KN\text{-}m/m) = \frac{0.8\,(S+0.61)P}{9.74}$$

where,

S = effective span as defined above
P = Load on one rear wheel of truck (KN)

For a simple span slab bridge where main reinforcement is placed parallel to the traffic, distribution of wheel loads in the transverse direction of the slab is computed as follows:

$$B = 1.219 + 0.06S,$$

where,

B = width (in metre) of slab over which the wheel load is distributed

In no case, however, shall this width of distribution exceed 2.134m.

The maximum live load bending moment per metre width of slab, without impact, is computed by the following formula;

H18 (KN) loading :

LLm (KN-m/m) = 13.14S.

Where,

S = effective span as defined above.

The moments for other loading cases shall be determined with proportionate of above values.

Shear & Bond Stress

Slabs designed for bending moment in accordance with the foregoing shall be considered satisfactory in bond and shear.

Distribution Reinforcement

The distribution reinforcement amount shall be the percentage of the main reinforcement steel required for positive moment as given by following formulas:

Where main reinforcement parallel to traffic:

Percentage = $55/S^{1/2}$, with maximum of 50%

Where main reinforcement perpendicular to traffic:

Percentage = $121/S^{1/2}$, with maximum of 67%

where,

S = the effective span as defined above in metre.

9.1.12 Longitudinal Edge Beams

Edge beams shall be provided for all slabs having main reinforcement parallel to the traffic. They shall be designed to resist a live load moment of 0.10 P S,

where,

P = wheel load in N.
S = effective span length , in metre.

9.1.13 Cantilever Slabs

For the wheel load of a truck placed on the cantilever slab, the bending moment per m of slab shall = PX/B (N-m),

where,

P = wheel load in N
X = distance in metre from load to point of support.
B = width of slab in metre over which a wheel is distributed which is given as follows :

The distribution for each wheel load on an element parallel to the traffic is,

$$B = 0.35X + 0.98 \text{ (m) but shall not exceed } 2.134 \text{ m.}$$

Each wheel on an element perpendicular to the traffic shall be distributed over width :

$$B = 0.8X + 1.143 \text{ (m)}$$

For railing load on a cantilever slab, the bending moment per m of slab = PX/B (N-m), in which X is the distance in m from the centre of the post to the point of interest. The effective width of slab (B) resisting post load is equal to the following :

where no parapet is used	$B = 0.8X + 1.143$ (m)
where parapet is used	$B = 0.8X + 1.524$ (m)

Railing and wheel loads are not to be applied simultaneously.

9.1.14 Distribution of Loading & Design of Concrete Girders

9.1.14.1 Position of Loads for Shear

In calculating end shears and reactions in longitudinal girders, no longitudinal distribution of wheel load shall be assumed for the wheel or axle adjacent to the end at which the stress is determined.

Lateral distribution of the wheel load shall be that produced by assuming the floor to act as a simple span between girders.

9.1.14.2 Bending Moment in Longitudinal Girder

For interior girders the live load bending moment shall be determined by applying to the girder the fraction of a wheel load (both front and rear), as follows:

- Girder designed for one traffic lane, the fraction of wheel load is S/1.981.
- Girders designed for two or more traffic lanes, the fraction of wheel load is S/1.829.
- If S exceeds 1.829 m in case of one traffic lane and 3.048 m in case of two or more traffic lanes, then the load on each girder shall be the reaction of the wheel load, assuming the flooring between the girder to act as a simple beam.

Where S is effective span length as previously defined.

9.1.15 Foundation Design

Where scour is not a concern and the allowable bearing capacity of the soil is adequate to adopt shallow foundations, the minimum depth of foundation shall be 1.20 m.

In cases where scour may occur, the foundations of the bridge piers and abutments should be taken to 4/3 X maximum scour depth or the maximum scour depth + 1.20 m whichever is greater.

For calculating the bearing capacity of deep foundations, the effective depth of foundation shall be considered to be below the design maximum scoured bed level.

The foundation design with respect to bearing capacity and settlement has been elaborately analyzed in chapter 5.

9.2 Culverts

9.2.1 Design Discharge/ Design Water Level

The design discharge and water level for determining the opening area of a culvert shall be estimated by the methods stated in article 9.1.3.1.

9.2.2 Hydraulics of Flow Through a Culvert

The culvert flow may be either open channel or conduit.

In the case of open channel flow, the flow through the culvert is given by :

$$Q = Cbd\ (2g\ \Delta h)^{1/2}$$
$$\Delta h = Ke/2g(Vs^2 - Vc^2) + Ko/2g(Vs^2 - Vc^2),$$

where,

Vs & Vc = Velocity through structure and channel (both U/S and D/S) respectively.
Ke & Ko = Inlet and exit loss respectively which may be taken as follows:

Values of Ke are :

0.20 (30° - 45° Wingwalls)
0.30 (90° - Wingwalls)

and for Ko are:

0.30 (30° - 45° Wingwalls)
0.75 (90° - Wingwalls)

In the case of conduit flow, the flow through the culvert is given by

$$Q = CA\ (2g\ \Delta h)^{1/2}$$
$$\Delta h = v^2/2g\ (Ke + Ko + 2Ln^2g/R^{4/3})$$

Conservative values for the entry and exit loss co-efficients may be taken as follows:

$$Ke = 0.50$$
$$Ko = 1.0$$

The commonly used value for the roughness "n" for concrete pipes is 0.014.

9.2.3 Waterway Opening

In the following "length" refers to the dimension of the culvert which is parallel to the flow, and "width" refers to the dimension perpendicular to the flow i.e. cross-sectional width.

For a culvert flowing partly full, a suitable width for the rectangular section of a box culvert is about the same as, or slightly less than, the bed width of the trapezoidal channel. Generally, 70% fluming can be taken for design purposes i.e. Area of rectangular opening = 70% area of trapezoidal drain section at the same depth. Under such conditions, the velocity through the structure will then be 1.43 x Velocity in the channel.

For a culvert to be constructed on an undefined channel and flowing partly full, the headloss shall be the guiding factor in determination of the opening width. In such cases, the headloss in general, should not exceed 30 cm.

For culverts flowing full, the headless through the culvert shall be limited to 15 cm for determining the opening size of the box or pipe.

9.2.4 Scour Depth

The regime scour depth (R) for determining the cutoff depth at either end of the culvert invert slab shall be estimated by -

$$R = 1.35\ (q^2/f)^{1/3},$$

where,

q = Unit discharge (m^3/sec/m)
f = Silt factor

The maximum scour depth R is given by 1.25 times and 1.50 times the regime scour depth R at upstream and downstream ends of the culvert respectively.

9.2.5 General Features of Design

9.2.5.1 Types of Culvert

Culverts may be rectangular (Box), square (Box) or circular (Pipe) cross-section and may be either cast-in-situ or precast.

9.2.5.2 Culvert Size

The diameters of pipe culverts shall be 600 mm, 900 mm, and 1200 mm. The culvert shall comprise of a single row of pipe (i.e. not consist of pipes in parallel). Any culvert required to be in excess of 1200 mm diameter shall be designed as a Box Culvert.

For box culverts the height-span ratio of the opening shall be within the range 1:1 and 1:1.50.

The clear height of the Box culvert shall be limited to 6.00 m.

9.2.5.3 Length/Width of Culvert

The length of any pipe culvert shall not be less than the crest width of the road. The length of any box culvert shall be clear width of road.

To determine the culvert length/width the crest width of the different types of road shall be taken as follows;

Feeder Road	:	7.30 m
Rural Road (R1)	:	4.90 m
Rural Road (R2)	:	3.70 m
Rural Road (R3)	:	2.50 m

9.2.5.4 Curb and Railing

For Box Culverts the height of curb above the road shall not be less than 200 mm and preferably not more than 250 mm.

The minimum height of Railing shall be 900 mm measured from the top of the culvert to the top of the upper rail member.

9.2.5.5 Cutoff Wall

A cutoff wall should be provided across the full width of the culvert under both the Wingwalls and the base slab both upstream and downstream. Except where otherwise required from scour considerations, the minimum depth of cutoff wall shall be 1.00 m.

9.2.5.6 Head Wall and Wing-wall

Headwalls shall be located parallel to the road way, usually at or near the point where top of culvert meets the sloping embankment surface.

Wingwalls shall be used as extensions of headwalls for large structures to prevent the road materials from encroaching on the waterway, as well as to increase the hydraulic efficiency of the culvert. The angle between the channel axis and the wingwalls should be set to be within 30° to 45°.

9.2.5.7 Joints in Box Culvert

Large wingwalls shall be cast usually with a definite joint at the culvert headwall, (on the basis that lateral pressures will not normally cause any damage to the culvert).

9.2.5.8 Depth of Soil cover

The minimum depth of soil cover over the pipe crest shall be 600 mm.

9.2.6 Structural Design of Box

The members of the R.C.C. Box culvert shall be analyzed for the following three loading conditions:

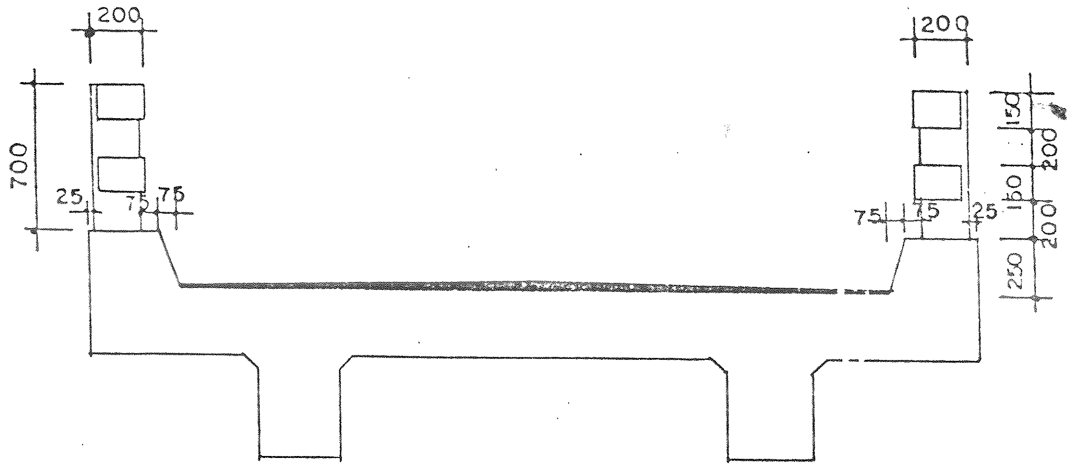
a. Culvert empty - Full live load
b. Culvert empty - No live load
c. Culvert full - Full live load

The members of the box shall then be designed for the maximum shear, moment and axial forces derived from such analysis.

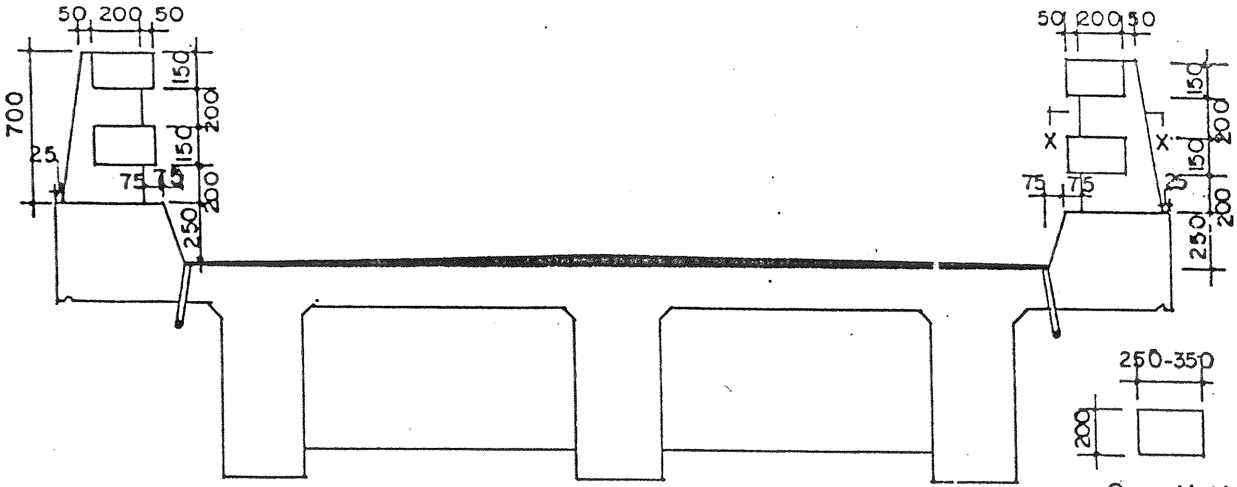
References:

1) SRP, Design Criteria, BWDB System Rehabilitation Project, October, 1993
2) AASHTO Specifications

Figure 9'1



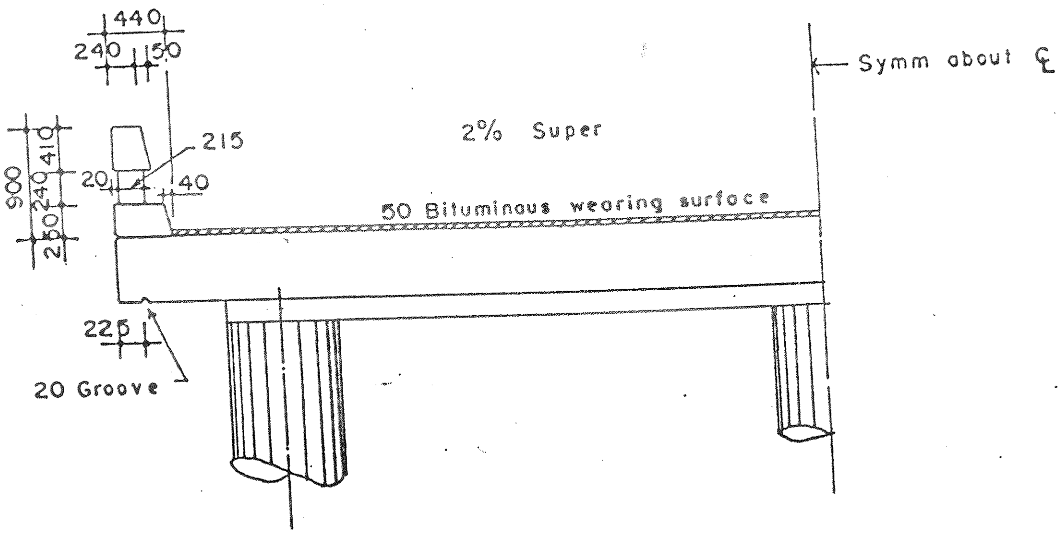
Light Trafic Bridge



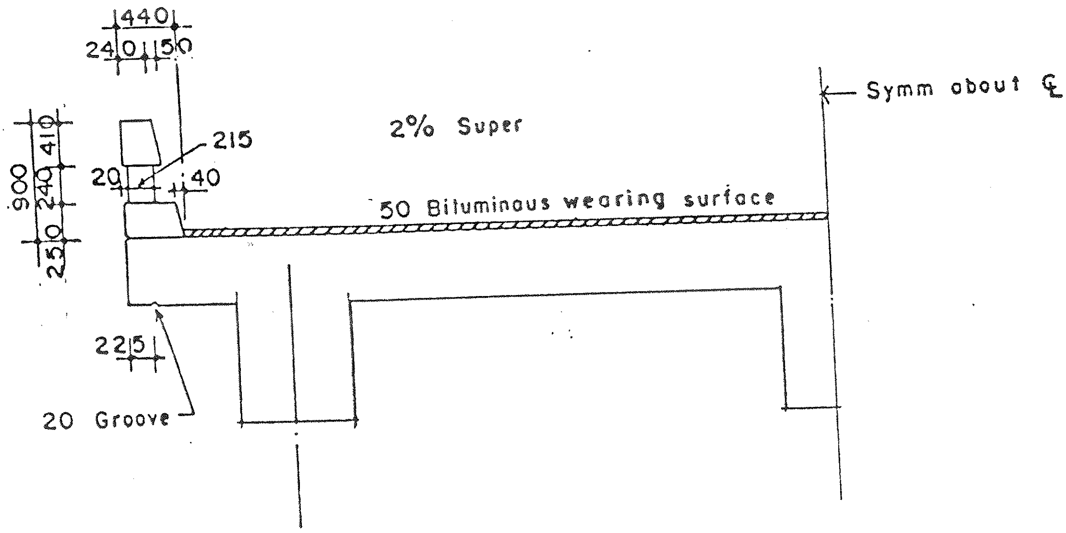
Medium & Heavy Trafic Bridge

TYPICAL SECTIONS OF TRAFIC BRIDGE

Figure: 9.2

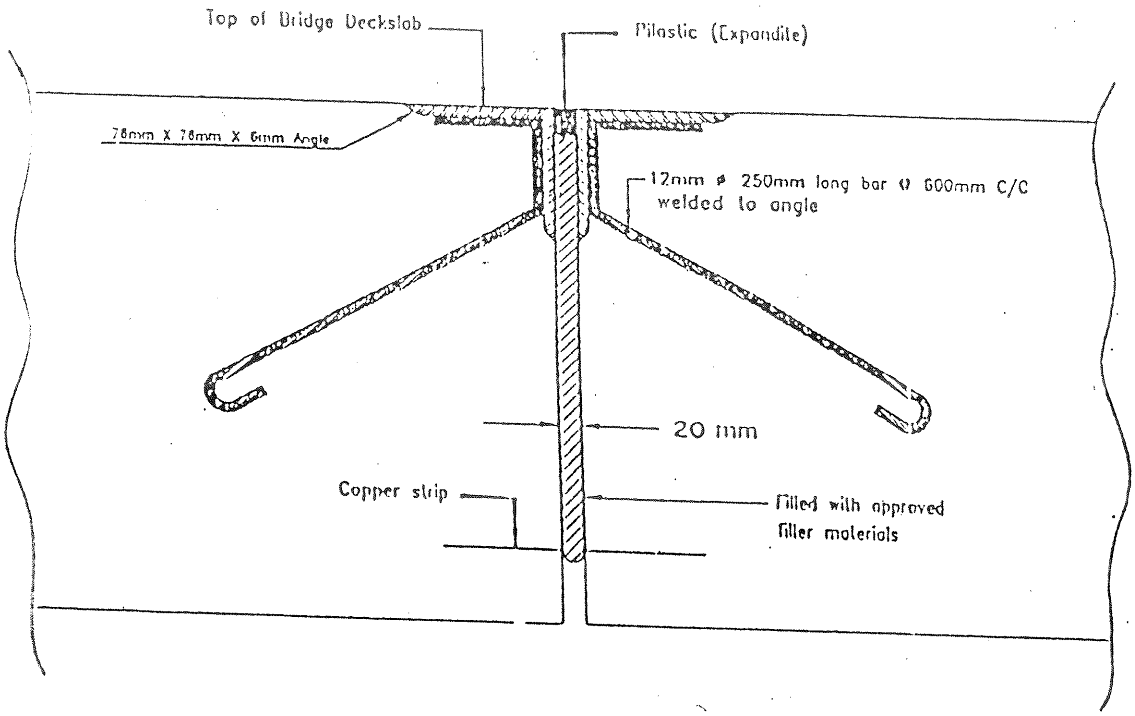


LONGITUDINALLY REINFORCED



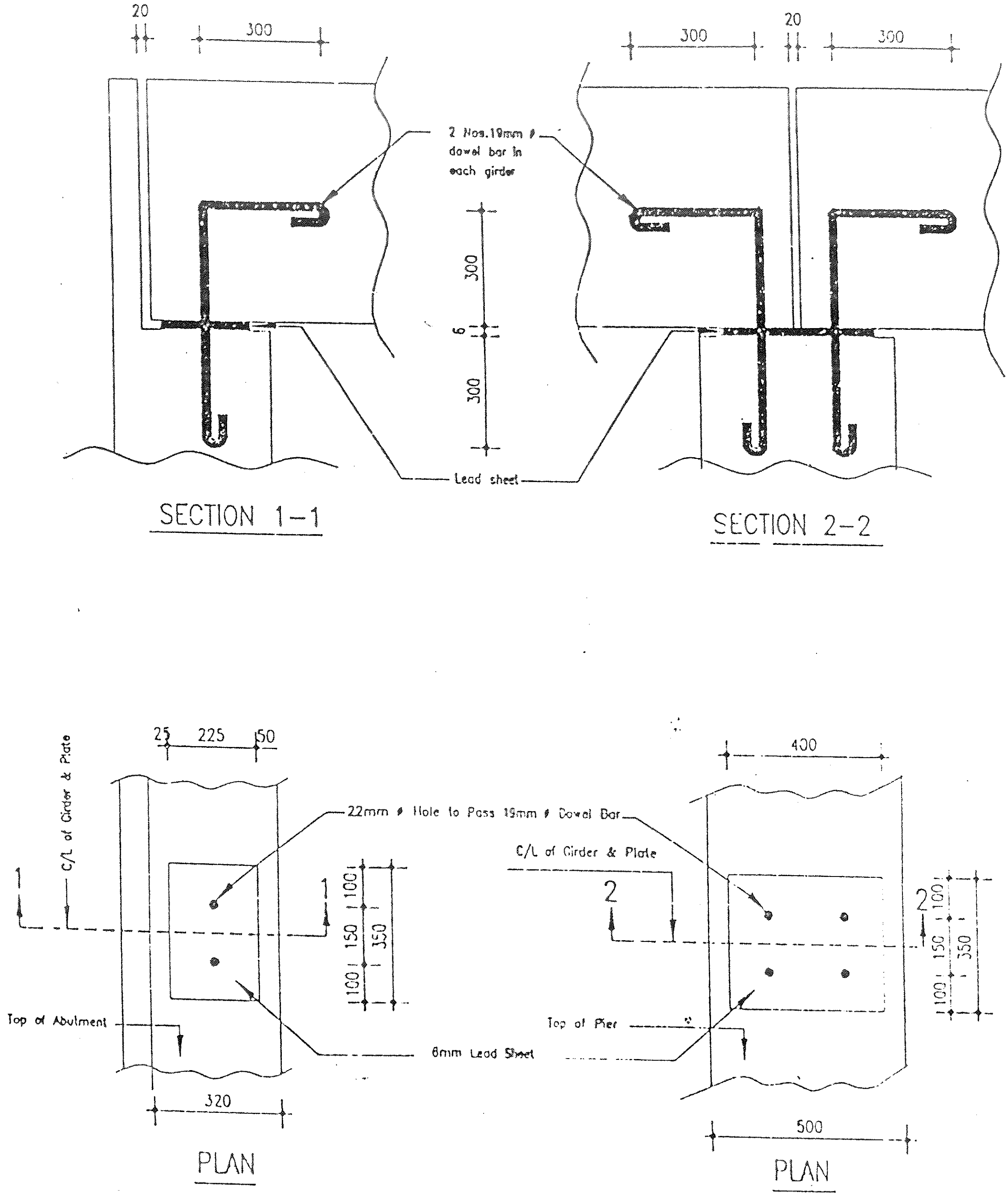
TRANSVERSELY REINFORCED

TYPICAL SECTION OF SLAB BRIDGE



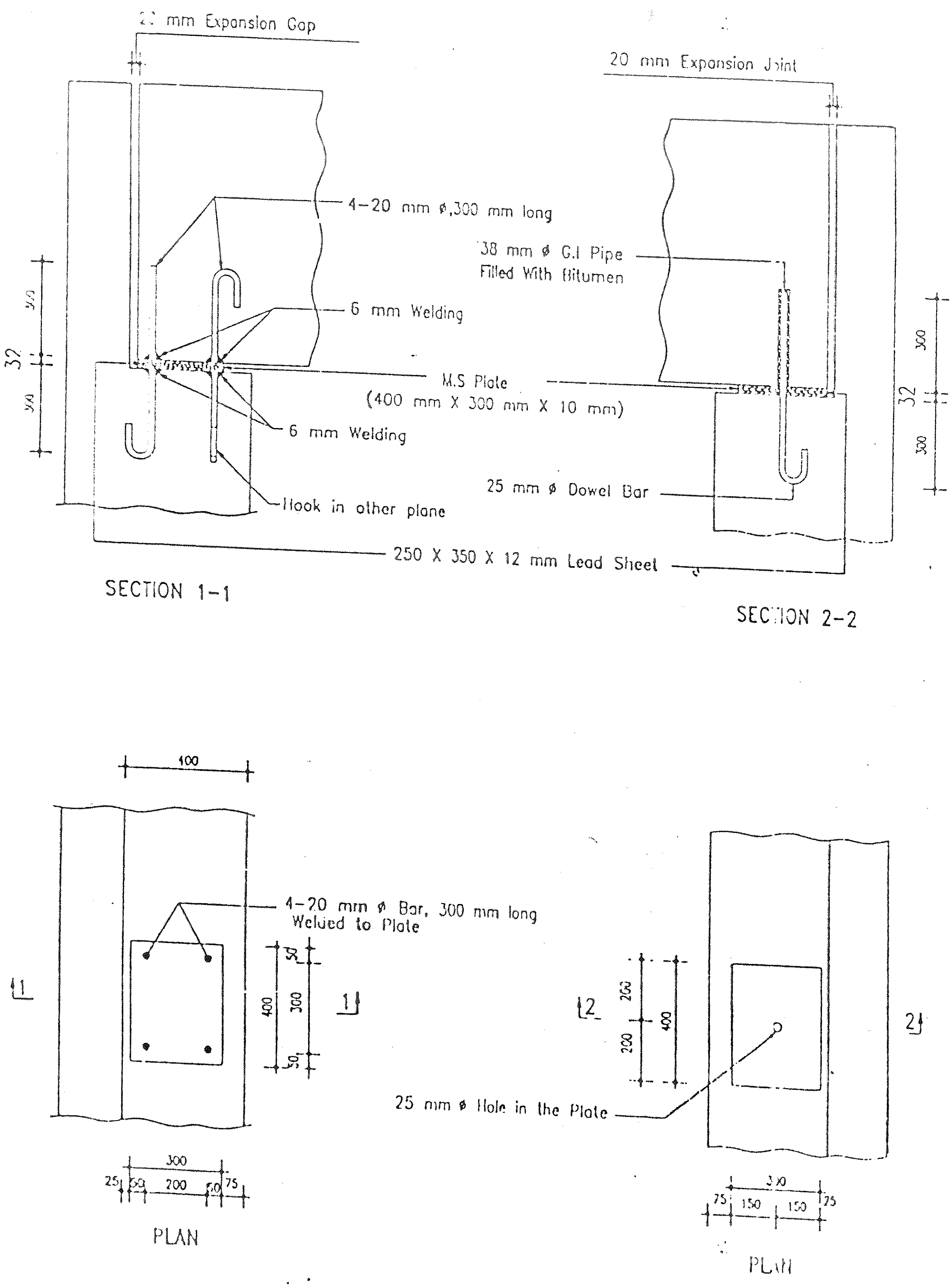
DETAILS OF JOINT ON BRIDGE DECK SLAB

Figure 9·4



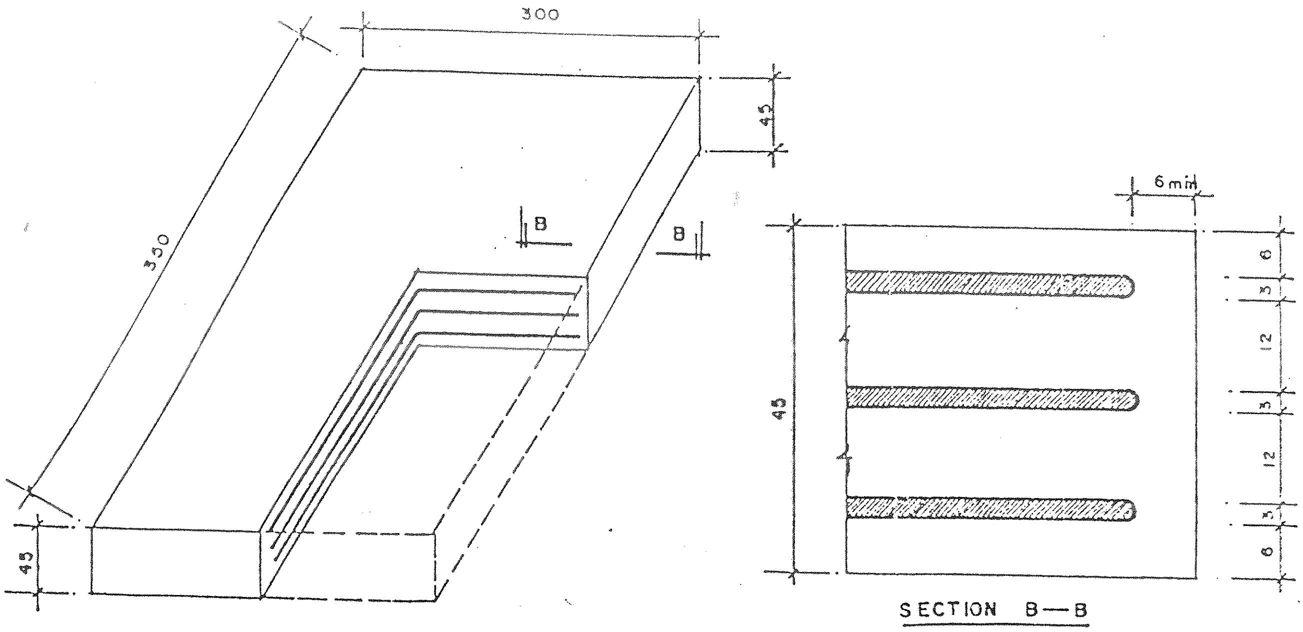
TYPICAL DETAILS OF FIXED BEARING

Figure 9·5



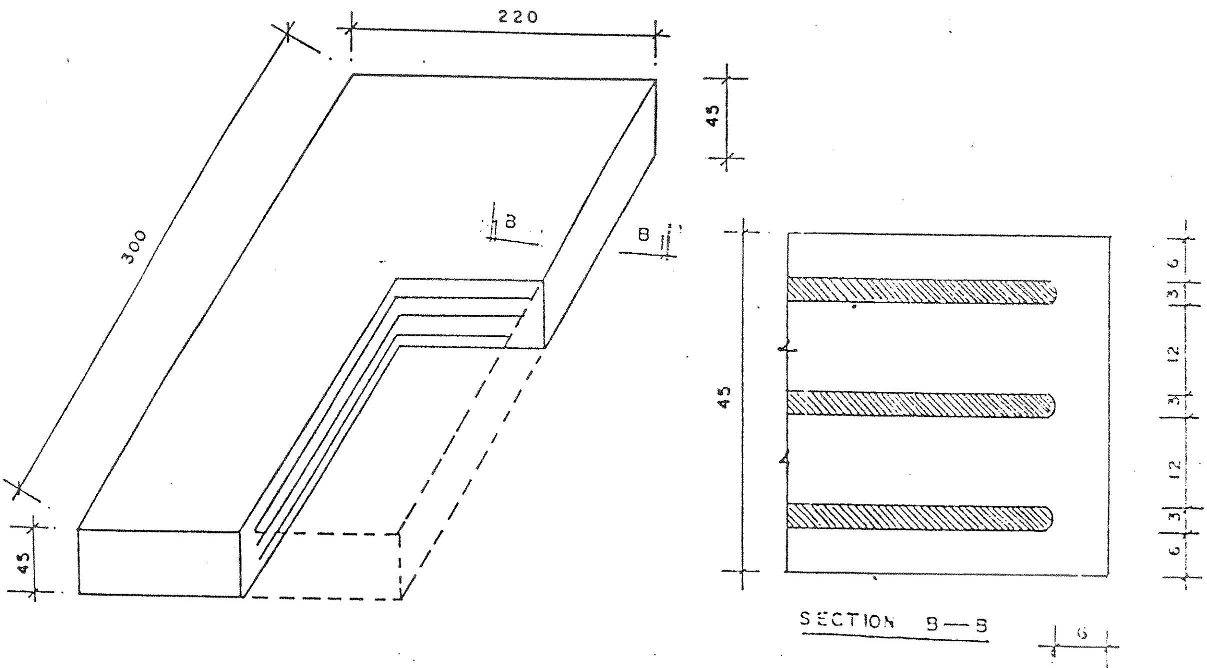
TYPICAL DETAILS OF EXPANSION & FIXED BEARING

Figure 9·6



ELASTOMER (NEOPRENE) BEARING PAD

SCALE. 1 : 10



ELASTOMER (NEOPRENE) BEARING PAD

10.0 BANK PROTECTION AND RIVER TRAINING WORKS

10.1 Purpose and Objective of River Training Works

The purpose of river training is to stabilize the channel along a certain alignment with certain cross section for one or more of the following objectives:

(a) Safe and expeditious passage of flood flow.
(b) Efficient transportation of suspended and bed load.
(c) Stable river course with minimum bank erosion.
(d) Sufficient depth and good course for navigation.
(e) Direction of flow through a certain defined stretch of the river.

10.2 Types of River Training Works

Type of works to be done for the purpose of training a river or a reach thereof is dependent on the objective and engineering principle chosen to be adopted in the river training programme. The usually adopted types of river training works are described below:

10.2.1 Bank Revetment

This type of River Training Works involve a protective cover of a suitable hard material applied on the slope and toe of the river bank so that the bank soil is protected from the actions of erosive forces of flowing water and dynamic actions of waves. Revetments are of two types:

(a) *Open joint type* in which joint gaps between individual hard material remains open allowing free flow of water.

(b) *Close-joint type* in which joint gaps between individual hard material are sealed.

10.2.2 Groyne

Spurs/Groynes can be constructed of timber, steel or concrete piles and of earth, rock, boulders or stones. The terms spurs and groynes are synonymous, however, solid constructions are sometimes called groynes.

(a) Based on submergence criteria, spurs/groynes are classified as:

- Submersible spurs.
- Non-submersible spurs/groynes.

(b) Based on materials and method of construction, spurs/groynes are classified as:

- *Permeable spurs* are porous construction and allow significant part of impinging flow to pass through the body of the structure. Porosity of the structure may be not less than 40-50 percent.

- *Impermeable spurs/groynes* are solid constructions. Impermeable spurs/groynes usually have armoured earthen construction.

(c) From functional view points, Spurs/Groynes are classified as

- *Attracting type*, where due to the intervention by the spur/groyne, incident flow of the river is attracted towards the bank.

- *Repelling type*, where the spurs repel the incident flow of the river in a desired direction away from the near bank. These types of groynes are aligned at upstream facing directions.

- *Deflecting type*, where the spurs, out of their intervention, change the direction of incident flow with a relatively local effect.

10.2.3 Guide Bundh

Guide bundhs are artificial banks constructed along planned alignments with the purpose of guiding river flow in a desired direction. Generally, guide banks are used in pairs.

10.2.4 Artificial Loop Cut

Loop cuts are sometimes made artificially as acts of river training to modify the course of the river. Loop cuts become successfully effective if made at fully developed loop, physical indication of which is that the prop and cut alignment will be tangential to the main direction of flow at the two ends of the loop.

The short cut (loop-cut) can be made using two different methods. The first method consists of dredging the full ultimately expected cross-section with a bed level corresponding to the average of bed elevations of upstream and downstream transition points. The other method is to dredge a pilot channel, leaving the remaining clearing to the river itself. The cross-section and the longitudinal profile of the pilot channel should be such that there is sediment transport even at low stages.

The primary requisite of alignment is that it be smooth and free from bends that are either too abrupt or too flat or other irregularities in the bank line. The existing meander pattern, the overall slope and channel length should be generally maintained.

Straight reaches should be avoided. The straightening of a truly meandering channel over long reaches should never be attempted.

10.2.5 Island

An artificially constructed island or an existing natural island trimmed to required streamline shape and protected by pitching (revetment) on all sides causes deepening of the river bed in the vicinity through scouring.

10.2.6 Retards

Retards (also termed pallasiding) are structures longitudinal to the flow and constructed in the river infront of the bank to be protected.

10.2.7 Porcupine

Porcupines are deployed in staggered arrangements in rows and columns to cover the under water surface of slope and toe of river bank desired to be protected from erosion.

Porcupines are effective in sediment laden tidal rivers of coastal area where velocity of flow are moderate.

10.2.8 Other Methods

Bandals : Bandals are spur like bamboo fence units constructed in groups across side channels in river bed to check flow through channels and induce deposition.

Biological Treatment : Planting of water resistant trees along reaches of flood plains.

10.3 Hydraulic Factors

For design of structural element of river training works, hydraulic factors such as water level, discharge, flow velocity, wave height and anticipated scoured bed level of river have to be established. The criteria for selecting each of these hydraulic factors are described in the following sections:

10.3.1 Water Level

The following data of hydrological analysis are important for design purpose :

Flood levels	-	1:100 Years 1:50 Years 1:20 Years / Bankfull level Average year level
Low water Level	-	Lowest during dry season
	-	Average lowest in various dry ✓ months

10.3.2 Discharge

The design discharge may be taken from available analysed records or may be computed from available data.

The design discharge for various conditions may be taken as follows:

- For major rivers : 1 in 100 year or 50 year discharge.
- For minor rivers : Bankfull Discharge
- For flashy rivers : Dominant discharge which may be bankfull discharge or 70% of 1 in 20 year discharge whichever is higher.

10.3.3 Current

The revetment used to protect an erodible surface against erosion has to withstand the drag forces generated by the velocity.

The usual practice in design is to apply rules of thumb to estimate local velocities. The local velocity at the boundary of a straight channel is assumed to be two thirds of the cross sectional mean velocity. The velocity at the outer bank of a severe bend can be upto four thirds of the cross sectional mean.

It has been shown both theoretically and experimentally that the local velocity around the periphery of a cylindrical pier in a channel is twice the mean velocity in the channel. In the absence of any other field or experimental data, it is reasonable to assume that the revetment at the nose of groynes should be of a size that will be stable against velocities that are at least twice the mean velocity upstream.

The average velocity may be obtained from measured data of a discharge measurement site. Otherwise, if the discharge and corresponding water levels are known, the average velocity may be computed from surveyed river cross section at point of interest.

10.3.4 Waves

The wind waves would usually govern the design of any protection work at the slopes of river training works. With respect to protection two aspects of wave have to be considered:

- Runup of waves against slope which might overtop the upper limit of protection
- Erosive forces of breaking waves against the slope causing erosion

To evaluate wave height the following factors that create waves in open water must be analyzed:

- Design wind direction
- Effective fetch
- Wind velocity and duration

The relationships are based on the Sverdrup-Muck theory as modified by Bretschneider and are considered accurate and reliable enough for riprap design.

In most instances deep water waves are generated in front of an embankment.

The following table summarises the characteristics of the waves that will be generated for variety of conditions:

Table 10.1
Characteristics of the Waves

Wind speed (m/sec)	Minm duration of wind (hrs)	Fetch length (km)	Wave height (m)	Wave period (secs)
15	1.00	5.0	0.7	2.8
	1.75	10.0	0.9	3.3
	2.25	15.0	1.2	3.8
30	0.75	5.0	1.3	3.5
	1.50	10.0	1.8	4.5
	2.00	15.0	2.0	5.0

10.4 Scour

The protection of toe of any river training is of prime importance as regards to the stability of overall slope protection against scour.

Lacey's regime formula is widely used to find out scour depth in unconstricted alluvial rivers. The empirical regime formula due to Lacey is given below :

$$D_m = 0.47 (Q/f)^{1/3}$$

where,

D_m = the scour depth at design discharge
Q = the discharge
f = Lacey's silt factor
= $1.76\ (d_m)^{1/2}$
d_m = weighted mean diameter of sediment particles in mm.

The water surface level corresponding to Q must be known in order to determine scour levels from the above equation. To estimate the maximum natural scoured depth, a multiplying factor must be added. The table shown below gives co-efficients recommended by the Indian Code (after Lacey) based mainly upon consideration of cross sectional shape.

Table 10.2

Empirical Multiplying Factors for Maximum Scoured Depth

Nature of Location	Factor
Straight reach of channel	1.25
Moderate bend	1.50
Severe bend	1.75
Right angle abrupt turn	2.00
Noses of piers	2.00
Alongside cliffs & walls	2.25
Noses of guide banks	2.75

In working out the silt factor f, it is important to see as a which sand sample it pertains.

10.5 Safety Factor and Procedures for Slope Stability Analysis

The Swedish Slip Circle Method, which supposes the surface of rupture to be a cylindrical one, is a comparatively simple method of analysing slope stability and may be used.

It is estimated that next to the common erosive action of an exposed slope falling water levels in the river will introduce the most severe risks of loss of micro- and macro-stability.

10.6 Planning Principles

10.6.1 General Criteria

The main function of a bank protection work is to provide a substantial interface between the water flow and the containing ground. To achieve this the improvement must satisfy the following basic requirements e.g. *Stability, Flexibility, Durability, easy maintenance, Safety and Environmentally acceptable.*

These requirements are achieved through an initial deterministic design and a subsequent probabilistic verification.

10.6.2 Physical Model Test

Finally the different parameters of the selected river training works may have to be determined and verified by conducting a physical model test.

10.7 Layout and Dimensioning of Bank Protection Works

10.7.1 Revetment

A revetment is a type of protection of a river bank or an embankment which covers continuously the entire slope of the bank or the embankment including the portions extending into the river bed.

Regarding the geometrical dimensions, a distinction is to be made between a revetment placed on an embankment and a revetment on the river bank. In the later case the alignment of the revetment has to follow the run of the river bank which is to be graded to required slope by excavation, filling, dredging and/or hydraulic backfill.

In case the revetment has to protect the river side of an embankment being an artificial bank the alignment has to be determined in such a way that high current attack is avoided. The crest of the revetment is as high as the elevation of the embankment, the level of which shall be in case of permanent structures sufficient above the high water level of 100 years return period taking into account possible wave run-up.

The crest width should be sufficient to provide a road for supervision and transportation of materials for maintenance and emergency works.

A berm on the river side of the embankment should be provided to facilitate the execution of construction and maintenance works, to improve the stability and/or to a have a good transition between different types of revetment. Its level should be above such a water level allowing construction and maintenance in the dry during a certain period of time annually. The width of the berm depends on stability analysis and dimension of construction and maintenance equipment.

10.7.2 Groyne

10.7.2.1 Selection of Type of Groyne

A first step in the planning process of a bank protection with groyne is the selection of the type of groyne :

- a single or a series of groynes;
- a permeable or a impermeable groyne, or,
- a submerged or a non-submerged groyne.

Groyne can be used single or in series. Their use in series arises if a long reach is to be protected and depends also on the flow, deflecting or repelling. Furthermore, the flow pattern between groynes built in series can be influenced by an optimized layout plan as are the location and of scour. Permeable groynes have an advantage if placed in a series, because of several permeable groynes, the reduction of the flow velocity near the bank will be enhanced by properly selected spacing of the groynes.

In a standard layout of a series of groynes all the groynes are similar regarding length, cross-section, orientation and the shape of the head of the groyne. However, often the most upstream groyne is attacked by the flow stronger than the other downstream groynes, and in that case the design of that groyne should be adjusted. In principle a selection from different options can be made for this adjustment : either the length of the groyne can be reduced, or the orientation can be changed or the top layers can be stronger than those of the other downstream groynes.

Single groyne protect the bank downstream of the groyne over a relatively short distance against bank erosion. Due to their sensitivity to change directions of flow attack, single groynes are not recommended for general application.

10.7.2.2 Length and Spacing of Groyne

The length of groyne is governed by the shape of the cross-section of the river and the extent of protection of the bank required. The latter leads obviously to a correlation with the spacing of groynes when constructed in series. In case a single groyne is planned, its length must be sufficient to protect the bank against erosion in the required area.

Generally the length of the groyne is selected between 10% to maximum 50% of that width. The length is often limited by the wish not to construct the groyne in the deepest part of the channel close to the thalweg. Along outer bends often series of relatively short groynes are built, because of the great water depths along the outer bend (bend scour and local scour).

Experiences with groynes in Indian rivers show that a blockage of 30% of the channel width at bankfull stage can be used as a guideline (Varshney et al., and Jogleker, 1971).

The ratio between length and spacing between the groynes is one of the most important factors of the effectiveness of a groyne field. In case groynes are spaced too far apart, the current may return to the bank before the following groyne in the system is able to influence the flow direction, resulting possibly in bank erosion or even loss of the next downstream groyne. If groynes are spaced too closely together, the groyne system would be too costly for its effectivity.

General practice is to relate the spacing of groynes to their length. The spacing depends not only on the length of the groyne , but also slightly on the orientation to the flow velocity, the bank curvature, and purpose of the groynes.

General practice is a spacing of about 2 to 3 times the length of groynes. This rule includes some safety for the bank erosion. In favourable conditions and without additional safety a single groyne can protect 4 to 5 times the length of the groyne An Indian guideline suggests a spacing of the groynes by 0.1 to 0.15 of the meander length of an outflanking channel. If the meander length is 15 to 30 times the channel width and length of the groyne is 30% of the channel width, then the spacing is 0.6 to 1.5 times the length of the groyne.

For impermeable and permeable, non-submerged groynes along a straight bank, a spacing of 2.5 to 3 times the length of the groyne is recommended for bank protection. This recommendation includes some safety.

On concave outer banks, groynes are to be placed closer together than on straight banks and the design ratio should be reduced to 2 to 2.5. On a convex bank the spacing between the groynes can be slightly more than the recommended spacing along a straight bank (as a first indication a design ratio of 3 to 3.5)

The ratio of spacing to the length of groynes required for bank protection only is less than that required for navigation channels.

The length of the most upstream groyne of a series of groynes should be less than the standard length of the downstream groynes, because of the strong attack on this upstream groyne. The reduction in length depends on each individual situation and no general recommendation can be given.

10.7.2.3 Shapes of the Groyne Heads

The main types of groyne heads of impermeable and not-submerged groynes are described in the following :

(a) Bell head

A strongly protected head is also called a bell-head. A bell head is often used if the direction of the approach flow is more or less fixed.

(b) T-head and L-head

Straight groynes with a rectangular vane at their heads are T-head or L-head groynes, they give more guidance to the flow and define the separation of the flow at the downstream side of the vane. The vane is oriented in such a way that it guides the flow. A T-head vane has two wings with an equal length. If the groyne blocks a relatively high percentage of the cross section (for example more than 30%) a T-head reduces the risk of erosion at the opposite bank downstream of the groyne. A long upstream wing can reduce the attack on the shank of the groyne and allow a reduction of the strength of the top layer of the shank.

(c) Hockey-stick head

Groynes bent like a hockey stick at their head are known as hockey shaped groynes, which is a combination of a short L-head with a bell head. A hockey-stick is applied if a very strong attack on the head is expected, stronger than in case of a bell head, and if a small reduction of this attack allows the selection of a less strong top layer of the head.

Since no general rules are available for determining the most convenient shape of the groyne head as well as other dimensions depending on a number of different design parameters, it is in many cases advisable to undertake physical model tests.

10.7.2.4 Cross section

A typical cross-section of a groyne depends on many considerations, one of the most important ones being whether the groyne should be conceived as permeable or impermeable.

(a) Permeable Groyne

Permeable groynes are generally constructed of timber, steel or reinforced concrete piles driven or sunk into the river bed in one or several rows. The individual vertical piles are mainly subject to horizontal loads, either on their total length by flow and wave attack or at changing heights due to floating debris, etc. In order to absorb the decisive horizotal loads, static and dynamic, it is advisable to have at least two rows of piles braced to each other by traverses and diagonals, the dimensions of which should be designed in accordance with standard rules of structural engineering. The spacing between the piles should be at least one diameter, hence the minimum permeability would be 50% of the area. For a standard design a maximum permeability of 70% is recommended, because of the effect on the flow field. Two parallel rows of piles can be placed in two different positions relative to each other. No recommendation is given for a certain arrangement.

The driving depth of the piles depends mainly on the subsoil conditions, the local scour hole and the pile characteristics. A falling apron reduces the local scour between the piles and reduces the depth of piles driven into the soil. The height of the piles is normally 1 meter above the maximum design water level. In special cases the height of the piles can be reduced to below this level which would result in a submerged permeable groyne during high floods.

The dimensions of the piles are determined according to the methods applied in structural and soil mechanical engineering and the design conditions have to be carefully assessed for each individual case.

To prevent local scouring close to the pile row a bed protection or a light falling apron of rip-rap or boulders is recommended. By increasing the permeability of the pile row towards the head of the groyne the maximum depth of the local scour hole can be significantly reduced, this scour reduction being the main reason for applying permeable groynes.

(b) *Impermeable groyne*

Standard cross sections of the shank of an impermeable groyne have a trapezoidal shape: two sides slopes and a horizontal top. The top of an impermeable groyne should have a minimum width of about 3 m, so that a truck can pass for maintenance. If the sides of the shank have a protective top layer there slope should be designed as steep as the stability of the top layer allows to minimise the area of the top layer. This results in slopes of 1:2 to 1:3 at the shanks. A light falling apron or bed protection is recommended at the upstream side of the shank of the groyne.

The side slope around the head of the groyne is same as the side slope of the shank if no important scouring is expected. If a deep scour hole is predicted, then the depth of the scour hole can be reduced by flattening the side slopes of the head of the groyne to about 1:3 to 1:5 at the head of an impermeable earth built groyne.

Due to different construction methods above and below the low water level, the designs of the top layers of inclined slopes are often different above and below design low water level. The top layer below low water level must have a rather flat slope (1:3 to 1:4) for stability reasons or to reduce the scour depth, whereas the slope above design low water level may be steeper to save space and construction material. The transition from one top layer design to another is often less resistant to flow attack than the actual top layers. Therefore, a berm with a width of at least 1.5 m is designed at the transition.

The radius of the groyne head is determined by the effect of the head on the deflection of the flow, and the minimum radius may also be determined by the size of the top layer elements.

The crest of an impermeable groyne can be designed at standard high water level with a gentle slope of 1 : 100 towards the river along the axis of the groyne.

The crest of a permeable groyne may be either horizontal or have the same gentle slope as in case of an impermeable groyne. In a special case, e.g. for deep water and/or strong flow and wave attack, a steeper slope of the crest may also be appropriate.

It must be stressed that the above mentioned slope are not only depending on the hydraulic loads, but also on the construction material of the core and protection of the slopes. In any case detailed stability investigations are required in order to establish the required slope, as well as estimations of settlements of the earth built dam depending mainly on the subsoil conditions.

The existing river bed around a groyne should be protected from scouring by a bed protection or a falling apron. A permanent bed protection has a length of about 1.5 times the scour depth at the downstream edge of the bed protection.

10.7.2.5 Orientation of Groyne

Groynes facing downstream are not suitable for bank protection surfaces, since the current may attack the root of the next downstream groyne endangering not only the root and the surrounding bank area, but the whole groyne itself. Groynes placed normal to the flow may protect only a small area. However, groynes facing upstream deflect the flow away from the bank, and they are able to protect bank areas upstream and downstream of themselves. Thus deflecting groynes seem to be best suited for bank protection or sedimentation purposes. Design guidelines recommened angles of 100° to 120° to the flow, however, also the form of the bank, that means concave or convex, must be taken into consideration. In straight reaches groynes may be pointed upstream as much as 105° to 110°. Typical design of a groyne is shown in fig.10.1.

10.7.3 Guide Bundh

The Guide Bundh can be named according to the shape of the curved head, namely, straight and elliptical guide bundhs with circular or multi radii curved head (fig.10.2). Elliptical guide bundhs have been found more suitable in case of wide flood plain rivers as compared to straight guide bundhs. Due to gradual change in curvature in the former case, the flow hugs the guide bundhs all along its length as against separation of flow occurring in case of straight guide bundhs after the curved head which leads to obliquity of flow. Elliptical guide bundhs have also been found to provide better control on development and extension of meander loop towards the approach embankment. Any other shape warranted by site conditions and supported by hydraulic model studies may be adopted.

10.7.3.1 Length of Guide Bundh

The length of the guide bundh on the upstream should normally be kept as 1.0 L to 1.5 L. In order to avoid heavy river action on the guide bundh, it is desirable that obliquity of flow to the river axis should not be more than 30° (fig.10.3).

For wide alluvial belt the length of guide bundhs should be decided from two important considerations, namely, the maximum obliquity of the current and the permissible limit to which the main channel of the river can be allowed to flow near the approach embankment in the event of the river developing excessive embankment behind the training works. The radius of the worst possible loop should be ascertained from the data of the acute loops formed by the river during the past. In case of river where no past surveys are available, the radius of the worst loop can be determined by dividing the radius of the average loop worked out from the available surveys of the river by 2.5 for rivers having a maximum discharge upto 5000 m^3/s and by 2.0 for a maximum discharge above 5000 m^3/s.

The length of the guide bundhs on the down stream side should be kept as 0.25 L to 0.40 L.

10.7.3.2 Curved Head and Tail of Guide Bundh

The function of the curved head of upstream guide bundh is to guide the flow smoothly and axially to the structure keeping the end spans active. The radius of the curved head of guide bundh should be kept as small as possible consistent with the proper functioning of the guide bundh. A radius of the curved head equal to 0.4 to 0.5 times the width of the barrage between the abutments have usually been found to give satisfactory results, the lower and upper limits of the radius being 150 and 600 m, unless indicated otherwise by model studies.

Radius of curved tail normally ranges from 0.3 to 0.5 times the radius of curved head.

The angle of sweep of curved head ranges from 120° to 145° according to the river curvature. For curved tail it usually varies from 45° to 60°.

In case of elliptical guide bundhs, the elliptical curve is provided upto the quadrant of ellipse and is followed by multi-radii or single radius circular curve. In case of multi-radii curved head the larger radius adjacent to apex of ellipse is generally kept 0.3 to 0.5 times the radius of the curved head for circular guide bundhs, with angle of sweep varying from 45° to 60° and the smaller radius equivalent to 0.25 times the radius of curved head for circular guide bundhs with sweep angle of 30° to 40° (Fig.10.4). The shape should be finalised on the basis of model studies.

10.7.3.3 Cross Section of Guide Bundh

Top Width : In order to permit the carriage of material and vehicles the width of shank of guide bundhs should be kept 6 to 9 m at formation level. At the nose of guide bundhs, the width may be increased suitably to enable the vehicles to take turn and for staking stones.

Freeboard : A freeboard of 1.0 to 1.5 m should be provided above the highest flood level (HFL) for 1 in 500 year flood or above the afluxed water level in the rear portion of the guide bundhs calculated after adding velocity head to HFL corresponding to 1 in 100 year flood/standard project flood at the upstream nose of the guide bank, whichever is higher.

Side slope : The side slopes of guide bundhs depend upon the nature of the river bed material of which they are made and the height. A slope of 2:1 may generally be found adequate.

10.8 Design Principles For Structural Elements

Whatever may be the type of structure used in river training works e.g. bank revetment, groyne or guide bundh, the protection works can be divided into two main parts i.e. protection of bank itself and protection of toe of the bank. The protection of bank is done by revetment using selected materials and the protection of toe is done by launching apron or driven piles.

The following general criteria should be considered during design of any protection works :

- the revetment does not slide under frequently occurring hydraulic loads
- the revetment including filter layers and subsoil must be in equilibrium as a whole

- the component of the weight of the revetment normal to its face should be greater than the uplift pressure caused by water
- the surface particles of revetment should have enough resistance against wave and current attack
- the toe of revetment shall be stable against probable maximum scour in the river bed.

The structural elements of the protection works need to be selected to fulfil the above criteria are as follows :

- size and thickness of revetment materials in open type revetment
- thickness of revetment in close type revetment
- thickness and gradation of riprap
- thickness and gradation of granular filter
- type of fibre filter
- dimension of launching apron
- depth of pile used as toe protection.

10.9 Stability of Revetments under Current Attack

10.9.1 Loose Units

A number of equations relating the size of revetment material to the velocity impinging on it, has been developed.

The equation which are considered to be the most suitable are recommended and presented below : (for more details and explanation, see detail see **"Guide to Planning and Design of River Training and Bank protection works"**).

1. *Neill* This is in the form of a design graph (Fig. 10.5) for spherical stones (specific gravity = 2.65) and for bank slopes between horizontal and vertical, 1(V) : 2(H). The following equation has been fitted to the curve.

$$D = 0.034\ V^2 \qquad \text{(metric unit)}$$

2. *JMBA [Ref. 10]*

$$D_n = \frac{0.7V^2}{2(S_s-1)g} \cdot \frac{2}{\log(6h/D)^2} \cdot \frac{1}{[1-(\sin\theta/\sin\phi)^2]^{0.5}}$$

(FPS or metric unit)

In these Equations :

- W = Weight of individual stone (Lb. wt.)
- D = Diameter of stone (ft or m)
- D_n= Dimension of cube (ft or m)
- V = Velocity (ft/s or m/s)
- h = Depth of water (ft or m)
- S_s= Specific gravity of stone
- θ = Slope of bank (°)
- Φ = Angle of repose of revetment materials (°)
- g = Gravitational acceleration (ft/s² or m/s²)

10.9.2 Stone Gabions and Mattresses

(1) The following formula to determine the grain diameter of stones for filling of gabions and mattresses is given by PIANC (1987a)

$$d_{n50} = h * [\frac{u_{cr}}{B\sqrt{k'\psi_{cr} g \Delta_m h}}]^{2.5}$$

where,

d_{n50}	= grain diameter = $(W_{50}/\rho_s)^{1/3}$	(m)
h	= water depth	(m)
k'	= slope reduction factor	(-)
u_{cr}	= critical velocity = $[1 - (Sin^2\alpha)/(Sin^2\xi_s)]$	(m/sec)
ξ_s	= angle of internal friction of material	
ψ_{cr}	= Shields parameter = 0.1 in case of initiation of movement	
B	= coefficient of flow conditions = 5 to 6 for major turbulent flow and other bends of river = 7 to 8 normal turbulence of rivers = 8 to 10 for uniform flow and minor turbulence	
Δ_m	= relative density = $(\rho_s - \rho_w)/\rho_w$	(-)
ρ_s	= density of protection material	(kg/m³)
ρ_w	= density of water	(kg/m³)

The thickness of the mattress can be related to the stone size. According to PIANC (1987a) it is sufficient to use two layers of stones in a mattress.

$$t_m = 1.8\ d_{m50}$$

in which,

t_m = thickness of mattress (m)

(2) The following equation is again derived from the general Pilarczyk formula and valid for wire mattresses :

$$D_n = \frac{\overline{u}^2}{\Delta_m\ 2g} \cdot \frac{0.03}{\psi_{cr}} \cdot K_h \cdot K_s$$

where,

$\bar{u}$	=	current velocity	(m/sec)
ψ_{cr}	=	Shields parameter	(-)
K_h	=	Depth factor	(-)

$$= \frac{2}{\left[\log\left(6\frac{h}{D}\right)\right]^2}$$

h	=	water depth	(m)
D	=	D_n = nominal thickness of protection unit (m)	
K_s	=	Slope factor	(-)

$$K_s = \cos\alpha \sqrt{\frac{1 - \tan^2\alpha}{\tan^2\xi_s}} - \sqrt{1 - \frac{\sin^2\alpha}{\sin^2\xi_s}}$$

In which,

α	=	angle of slope	(degree)
ξ_s	=	angle of internal friction between cover layer and sub layer	(degree)

Since the stones are retained by the wire mesh, the Shields parameter can be increased from ψ_{cr} = 0.03 to ψ_{cr} = 0.06 to 0.10 in case of initiation of movement.

For the estimation of thickness of a geotextile mattress the following equation can be applied :

$$d \geq \frac{\phi \bar{u}^2}{\Delta_m 2 g} * K_h * K_s$$

where,

d	=	thickness of mattress	(m)
Φ	=	0.8 for continuous protection	(-)
	=	1.0 for exposed edges	(-)
K_s	=	slope factor	(-)
K_h	=	depth factor	(-)

10.10 Stability of Revetment Under Wave attack

10.10.1 Loose Units

Various methods and equations are available to determine the size of revetment material required to resist wave forces. Pilarczyk & Hadson Equations are recommended to use for computing revetment size against wave attack which are presented below :

(1) *Hudson* : $W = \dfrac{H^3 \rho \tan\theta}{k\Delta^3}$

(2) *Pilarczyk* : $D = \dfrac{H_s}{(S_s - 1)} \cdot \dfrac{1}{\beta} \cdot \dfrac{E^{1/2}}{\cos\theta}$

Where,

W	=	Weight of revetment material
D	=	Cubic dimension
H_s	=	Significant wave height
β	=	Strength coefficient, 3 for cubes and 2 for randomly dumped cubes
θ	=	Bank slope angle
E	=	Wave breaking parameter
	=	$1.25\ T/H_s^{0.5}\ \tan\theta$
T	=	Wave period (sec)
f	=	A coefficient related to the amplitude of the wave and the depth and angle of the slope.
ρ	=	Density of revetment material
Δ	=	Relative density of revetment material
	=	$(\rho - \rho_w)/\rho_w$
ρ_w	=	Density of water
k	=	A coefficient varying from 3.2 for smooth quarry stone to 10 for tetrapod
S_s	=	Specific gravity of revetment material

(3) The empirical form of Hudson's equation in fps unit with k=3.2, is

$$W_{50} = \frac{19.5\, G_s\, H_s^3}{(G_s - 1)^3 \cot\theta}$$

The solution of equation is shown graphically in fig 10.6, and may be used for preliminary estimation of sizing riprap rock on wave consideration.

Among the above formulae, the Pilarczyk equation is currently used in many big river training works.

10.10.2 Gabion and Mattress

The gabion or mattress of the thickness d must be stable as a unit. The thickness can be related to the stone size D_n. In most cases it is sufficient to use two layers of stones in a mattress and

$$d = 1.8\ D_n$$

(1) The thickness d of the gabion/mattress as well as the nominal diameter D_n of the stone fill can be determined by means of the Pilarczyk formula :

$$d = \frac{H_s \quad \xi_z^{0.5}}{\Delta_m \psi_u 2.25 Cos\alpha}$$

where,

ξ_z = breaker similarity index
ψ_u = 2 to 3
Δ_m = relative density of the system
Δ = relative density of fill material
= $(1 - n)\ \Delta_m$
α = slope angle

And

$$D_n = \frac{H_s \quad \xi_z^{0.5}}{\Delta_m \psi_u 2.25 Cos\alpha}$$

in which,

ψ_u = 2 to 2.5
Δ_m = 1.65

(2) For the estimation of the thickness of the gabions/mattress PIANC (1987a) gives the following equations :

$$t_m = \frac{H_s}{2(1-n)\Delta_m \ Cot\alpha} \qquad for \quad Cot\alpha < 3.0$$

$$t_m = \frac{H_s}{4(1-n)\Delta_m \ (Cot\alpha)^{1/3}} \qquad for \quad Cot\alpha \geq 3.0$$

where,

t_m = thickness of mattress
H_s = significant wave height
n = porosity of stone = 0.4

However, these equations are only used for significant wave heights of less than 1.00 m.

10.10.3 Grouted Stones

In case of grouted stone used as cover layer on a slope revetment the proper execution of grouting is of outstanding importance. Creating of a completely impermeable surface must be avoided because it may introduce extra lift forces.

(1) **Pilarczyk** recommends for the application of his general formula as follows:

$$D \geq \frac{H_s \ \xi_z^b}{\Delta_m \ \psi_u \ \phi \ \cos\alpha}$$

Where,

D	=	specific size or thickness of protection unit	(m)
H_s	=	significant wave height	(m)
Δ_m	=	relative density of a system unit	(-)
Φ	=	stability factor	(-)
ψ_u	=	system determined stability upgrading factor	(-)
	=	1.0 rip-rap as a reference	
	>=	1.0 for other revetment systems	
	=	1.05 for surface grouting (30% of voids)	
	=	1.50 for pattern grouting (60% of voids)	
α	=	slope angle	(degree)
ξ_z	=	breaker similarity index	(-)
	=	$\tan\alpha * (H_s/L_o)^{-0.5}$	
	=	$\dfrac{1.25 \times T \times \tan\alpha}{H_s^{1/2}}$	

In which,

L_o	=	wave length (m) = $\dfrac{gT^2}{2\pi}$	
T	=	average wave period	(sec)
b	=	in the range of 1/2 to 2/3	

10.10.4 Open Stone Asphalt

(1) The thickness of a cover layer of open stone asphalt exposed to wave attack can be determined by the following formula, PIANC (1987a):

$$d_b = 0.75 * \left[\frac{27}{16} * \frac{1}{1-V^2} * \left(\frac{P}{\sigma_b}\right)^4 * \left(\frac{S}{C}\right)\right]^{1/5}$$

where,

d_b	=	thickness of cover layer	(m)
σ_b	=	asphalt stress at failure	(N/m²)
P	=	wave impact	(N/m)
S	=	stiffness modulus of the asphalt	(N/m²)
V	=	Poisson's ratio for asphalt	(-)
C	=	modulus of subgrade reaction	(N/m²)

The following thicknesses of open stone asphalt layers on river banks can be estimated as a guidance :

Small rivers :

d_b	=	10 to 15 cm	(in situ)
	=	8 to 12 cm	(prefabricated mattress)

Large rivers :

d_b = 15 to 25 cm (in situ)
= 15 cm (prefabricated mattress)

(2) The recommendations of Pilarczyk for the application of his general formula to determine the thickness of a stone asphalt layer are as follows :

b = 2/3
ψ_u = 2.0 for open stone asphalt placed on geotextile on sand
= 2.5 for open stone asphalt placed on sand asphalt as sub layer. In this case the thickness of the system may be defined as the total thickness of both layers.

10.11 Thickness and Grading of Riprap

10.11.1 Riprap Thickness

Opinions of different authorities regarding the thickness of slope pitching are given below :

- U.S.Army Corps of Engineers (1991), recommends that thickness of protection should not be less than the spherical diameter of the upper limit W_{100} (percent finer by weight) stone or less than 1.5 times the spherical diameter of the upper limit W_{50} stone, whichever results in greater thickness.

- California Highway Division (1970) recommended that there should be at least two layers of overlapping stones so that slight loss of material does not cause massive failure.

- ESCAP (1973) recommends thickness of protection should be at least 1.5D, where D is the diameter of normal size rock specified

- Spring (1903) recommended thickness of stone in inches for covering by rough, heavy and loose stone for pitching from low water upwards as shown in Table 10.4.

- The thickness of stone pitching and soling for permanent slopes required at head, body and tail of guide bank for river flowing in alluvial plains as recommended by Gales (1938) is tabulated in Table 10.5.

- Inglis (1949) recommended following formula to compute thickness of stone required,

$$T = 0.06\ Q^{1/3}$$

Where,

T = Thickness of stone riprap in ft and
Q = Discharge in cfs

The Inglis formula apparently gives excessive thickness for discharge above 1.5 million cusecs.

The thickness determined above should be increased by 50 percent when the riprap is placed under water to provide for uncertainties associated with this type of placement.

10.11.2 Riprap Grading

Size distribution of a riprap mixture should form a smooth grading curve without a large spread between minimum and maximum sizes. Stones should be reasonably well graded. Table 10.3 shows grading specifications (Ref.- 12) for three classes of riprap which have been considered suitable for a fairly wide range of stream flow situations.

The size obtained from figure 10.5 should be taken as median diameter (D_{50}), which means that 50% by weight of the mixture should be larger.

10.12 Design of Toe Protection

10.12.1 Toe Scour Estimation and Protection

Lack of protection of the toe of revetment against undermining is a frequent cause of failure of revetment. Accordingly, protection of the toe of revetment by suitable method is a must.

10.12.2 Toe Protection Methods of Revetment

Toe protection of revetments may be provided by following methods:

a. *Extension to maximum scour depth:* Lower extremity of revetment placed below expected scour depth or founded on non-erodible bed materials.

b. *Placing launchable stone:* Launchable stone is defined as stone that is placed along expected erosion areas at an elevation above the zone of attack.

Launch slope is less predictable if cohesive material is present, since cohesive material may fail in large blocks.

10.13 Dimension of Launching Apron

Among the various methods, launching apron has been considered to be most economical and common method of toe protection of revetment. Adequate quantity of stone for the apron has to be provided for ensuring complete protection of the whole of the scoured face. This quantity will obviously depend on the apron thickness, depth of scour and slope of the launching apron. This has been considered below :

10.13.1 Thickness of Launching Apron

Spring (1903) recommended a minimum thickness of apron equal to 1.25 times the thickness of stone riprap of the slope revetment. He recommended further that the thickness of apron at the junction of apron and slope should be the same as that laid on the slope but should be increased in the shape of a wedge towards the river bed.

Since the apron stone shall have to be laid mostly under water and cannot be hand placed, thickness of the apron at junction according to Rao (1946) should be 1.50 times the thickness of riprap in slope. The thickness of river end of apron in such case shall be 2.25 times the thickness of riprap in slope.

The face slope of the launching apron may be taken as 2:1 for loose stone as suggested by Spring (1903) and Gales (1938). The dimension of launching apron proposed by different authors may be seen in Fig 10.7.

10.13.2 Size of Stone

The required size of stone for launching apron may be the same as for the size in slope revetment considering stream velocity as governing factor.

10.13.3 Length of Launching Apron

The general practice as recommended by Inglis (1949) is to lay the apron over a length of 1.5D, where D is the design scour depth below the position of laying.

10.13.4 Quantity of Stone in Apron

Knowing the thickness of apron, the depth of maximum probable scour and the slope of the launched apron, the quantity of apron stone can be assessed. For dimensioning and estimating quantity of stone in apron, Fig. 10.8 may be used.

10.13.5 Recommended Shape

It is recommended to use the shape of launching apron as suggested by T.S.N. Rao (Fig.10.8) considering the condition of stone provided at the junction of the apron and shape which appears to be more logical.

10.14 Design of Terminations and Transitions

The experience shows that damage to bank protection often starts at terminations, transitions and joints. As a general principle the revetment in transition zones or at joint should be of strength equal or greater than the adjoining systems.

This can be achieved by increasing the thickness of the cover layer at joints and transitions.

10.15 Design of Filter

The stability of the whole revetment depends on the type and composition of the filter layer. A filter should prevent excessive migration of soil particles, while allowing relatively unimpeded flow of liquid from the soil. The function of a filter is:

- to prevent migration of subsoil particles out of the bank slope (Retention Criteria) and
- to allow at the same time movement of water through the filter (Permeability Criteria).

The filter may consists of one of the following types of material:

- granular filter, made of loose, bounded or packed grains, and

- fibre filter, made of synthetic or natural materials.

The design of the filter layer must also take realistic account of construction constraints and consolidation of the subsoil following the construction.

Important soil parameters to be calculated using results of the soil tests are:

- coefficient of uniformity, C_u, from particle size distribution:

$$C_u = d_{60}/d_{10}$$

- linear coefficient of uniformity, C'_u, from particle size distribution :

$$C'_u = d'_{50}/d'_0 = d'_{60}/d'_{10} = d'_{70}/d'_{20} = d'_{100}/d'_{50} = (d'_{100}/d'_0)^{1/2}$$

- coefficient of curvature, C_c, from particle size distribution:

$$C_c = \frac{d_{30}/d_{10}}{d_{60}/d_{30}} = \frac{(d_{30})^2}{d_{60} \times d_{10}}$$

- hydraulic conductivity of soil, can be measured from directly permeability testing; if testing is not possible, then the hydraulic conductivity can be estimated from the particle size distribution. (fig.10.9)

10.15.1 Granular Filter

Conventional gravity filter design requires consideration of both the retention capability and the permeability of the granular filter.

The inverted filter shall be designed using the following criteria, (Ref : DRAINAGE PRINCIPLES AND APPLICATIONS - by International Institute of Land Reclamation and Improvement Page -174 & 175).

(a) The gradation of filter should conform to the following rule,

$$\frac{d_{15}\ \textit{filter material}}{d_{85}\ \textit{base material}} \leq 5$$

$$\frac{d_{50}\ \textit{filter material}}{d_{50}\ \textit{base material}} \prec p$$

$$\frac{d_{15}\ \textit{filter material}}{d_{15}\ \textit{base material}} \leq q$$

Filter type	p	q
For homogenous and round grains	5-10	5-10
For homogenous sharp grains (Sylhet sand)	10-30	6-20
For graded grains (sized khoa, stone chips)	12-60	12-40

(b) The sieve curves of all layers should be almost parallel in the area of the smaller fractions.

(c) Minimum layer thickness

Sand - 0.10 m
Gravel - 0.20 m
Stone - 2 times stone diameter

An analysis of the retention criteria shows that if the pore spaces in granular filters are small enough to retain the coarsest 15% (i.e. d_{15} size) of the adjacent soil, then the majority of the finer soil particles will also be retained.

10.15.2 Fibre Filter

The criteria for geotextile filter selection are as follows:

- retention criteria to ensure that the geotextile openings are small enough to prevent excessive migration of soil particles.
- permeability criterion to ensure that the geotextile is permeable enough to allow liquids to pass through it without significant flow impedance.
- anti-clogging criterion to ensure that the geotextile has enough openings so that if soil becomes entrapped within the geotextile and clogs a few openings, the permeability of filter will not be significantly hampered.
- survivability criterion to ensure that the geotextile survives its installation.
- durability criterion to ensure that the geotextile is durable enough to withstand adverse chemicals, ultraviolet exposure, and abrasive environments for the design life.

PIANC (1987a) provides following general criteria for design of fibre filter :

(a) Retention

- for soils with a uniformity coefficient less than 5: $0.05\ mm < O_{90} < 0.7\ d_{90}$
- for soils with a uniformity coefficient grater than 5: $0.05\ mm < O_{90} < d_{90}$

(b) *Permeability*

$$\Psi > 5 \times 10^{3} * k_s * i$$

Where,

Ψ	=	permittivity	(1/s)
k_s	=	permeability of the subsoil	(m/s)
i	=	hydraulic gradient in the soil	

10.16 Placement of Riprap

Common methods of riprap placement are hand placing; machine placing, such as from a skip, dragline, or some form of bucket. Hand placement produces the most stable riprap revetment. This reduces the required volume of hard material.

The layer thickness and stone size may be increased somewhat to offset the short comings of placement method. Thickness of underwater placement should be increased by 50% to provide for the uncertainties associated with the type of placement.

Table - 10.3

Suggested Stone Rip-Rap Grading for Stream Bank Revetment

Class - I

Nominal 12 inch diameter or 80 lb weight
Allowable local velocity up to 10 ft/sec

Grading Specification

	100%	smaller than	18	inches	or	300 lb
At least	20%	larger than	14	inches	or	150 lb
At least	50%	larger than	12	inches	or	80 lb
At least	80%	larger than	8	inches	or	25 lb

Class - II

Nominal 20 inch diameter or 400 lb weight
Allowable local velocity up to 13 ft/sec

Grading Specification

	100%	smaller than	30	inches	or	1500 lb
At least	20%	larger than	24	inches	or	700 lb
At least	50%	larger than	20	inches	or	400 lb
At least	80%	larger than	12	inches	or	70 lb

Class - III

Nominal 30 inch diameter or 1500 lb weight
Allowable local velocity up to 15 ft/sec

Grading Specification

	100%	smaller than	48	inches	or	5000 lb
At least	20%	larger than	36	inches	or	2500 lb
At least	50%	larger than	30	inches	or	1500 lb
At least	80%	larger than	20	inches	or	400 lb

Table - 10.4

Spring's Table to Compute Thickness of Stone on Slope

River bed materials as classified by Spring	Thickness in inches for river slopes in inches per mile					Remarks
	3	9	12	18	24	
Very Coarse	16	19	22	25	28	The stone pitch prevents sand under neath from being sucked out by high velocity flow. More rationally stone pitch thickness should be based on velocities.
Coarse	22	25	28	31	34	
Medium	28	31	34	37	40	
Fine	34	37	40	43	46	
Very Fine	40	43	46	49	52	

Table - 10. 5

Gale's Table to Compute Thickness of Stone on Slope

River	Rivers with discharge 0.25 to 0.75 million cusec		Rivers with discharge 0.75 to 1.5 million cusec		Rivers with discharge 1.5 to 2.5 million cusec	
Parts of guide bank	Head	Body and Tail	Head	Body and Tail	Head	Body and Tail
Pitching Stone	3'-6"	3'-6"	3'-6"	3'-6"	3'-6"	3'-6"
Thickness of Soling Ballast	7"	7"	8"	8"	9"	9"
Total Thickness	4'-1"	4'-1"	4'-2"	4'-2"	4'-3"	4'-3"

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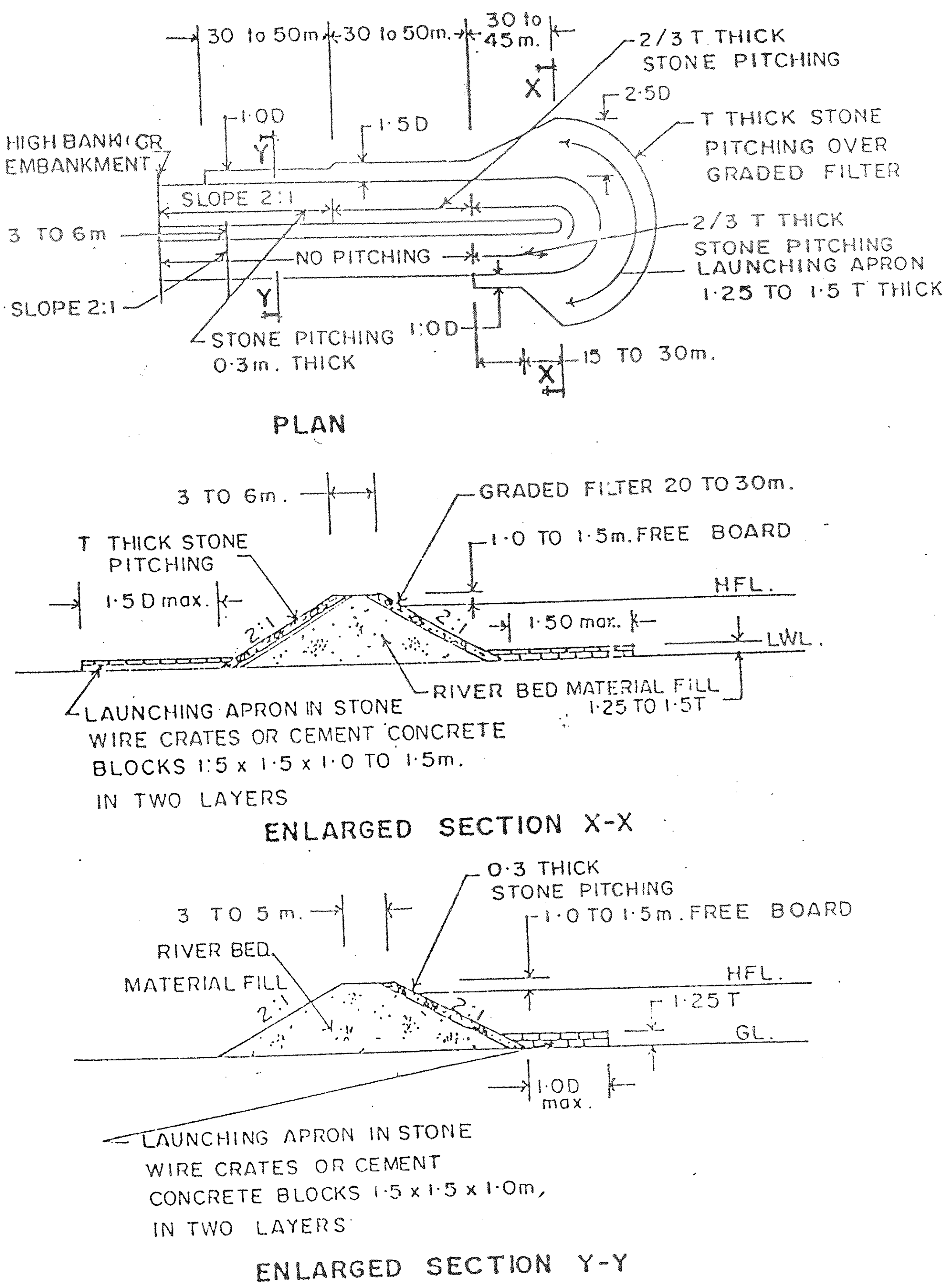
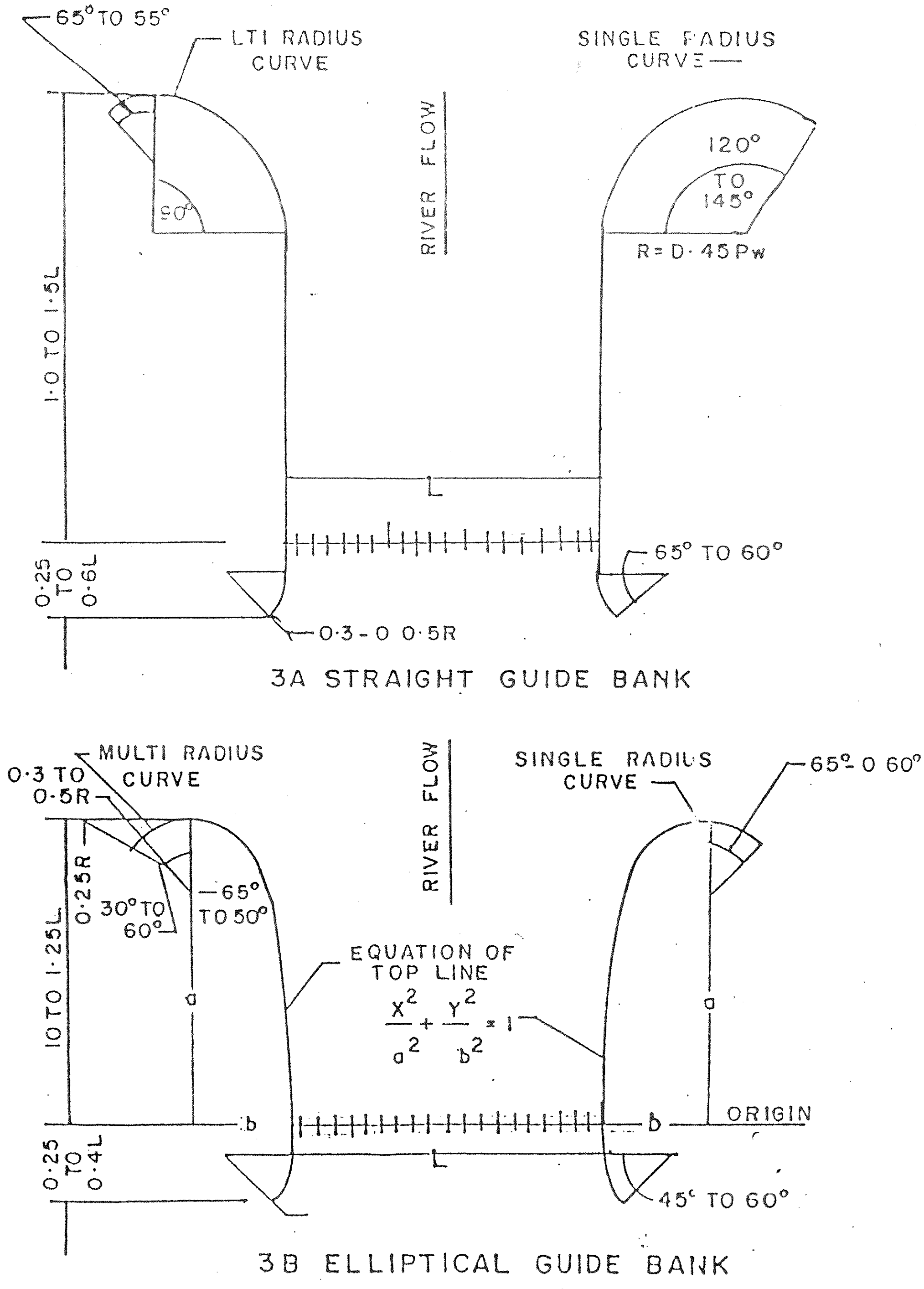


TYPICAL DESIGN OF SPUR

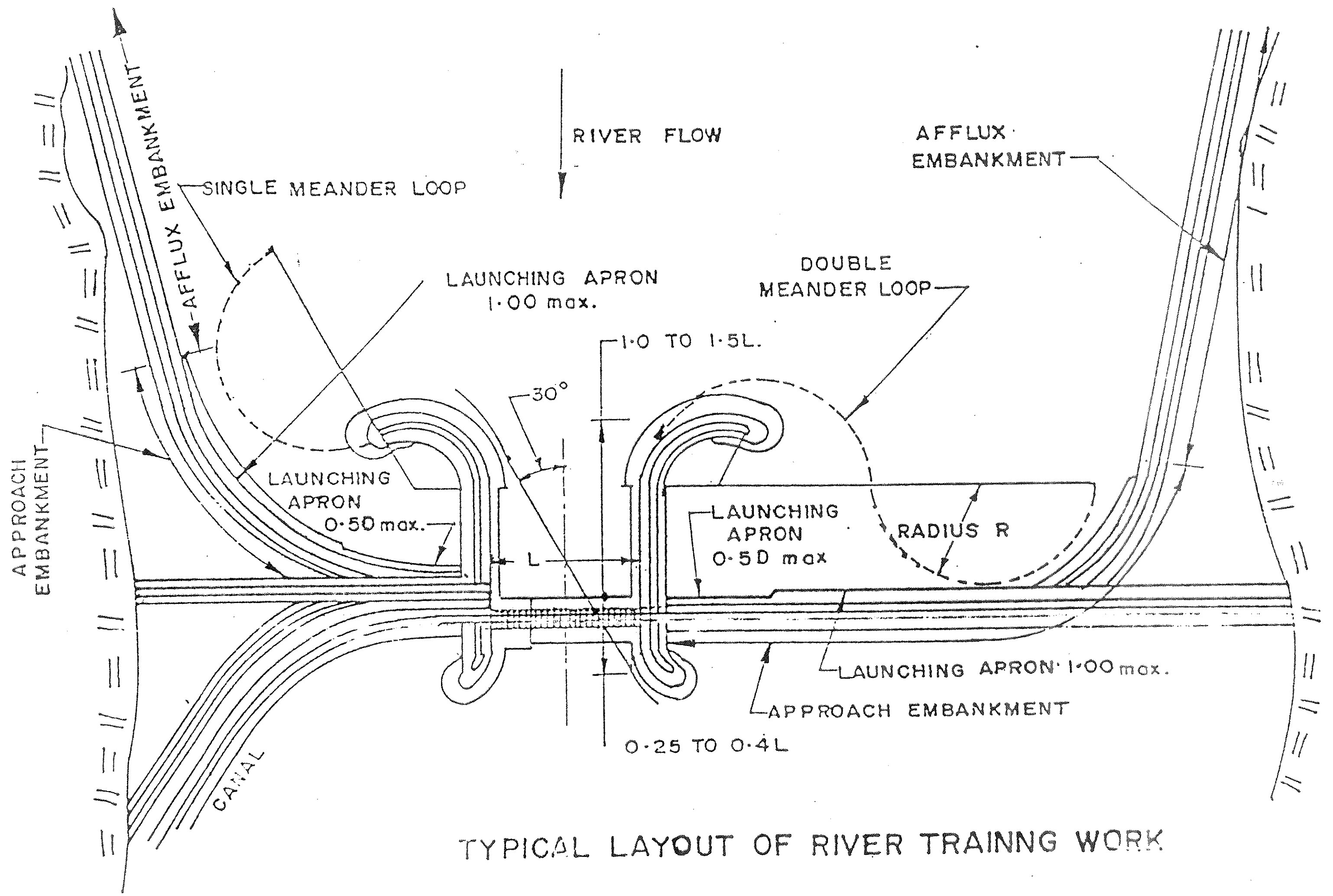
FIG·10·1



$$\frac{X^2}{a^2}+\frac{Y^2}{b^2}=1$$

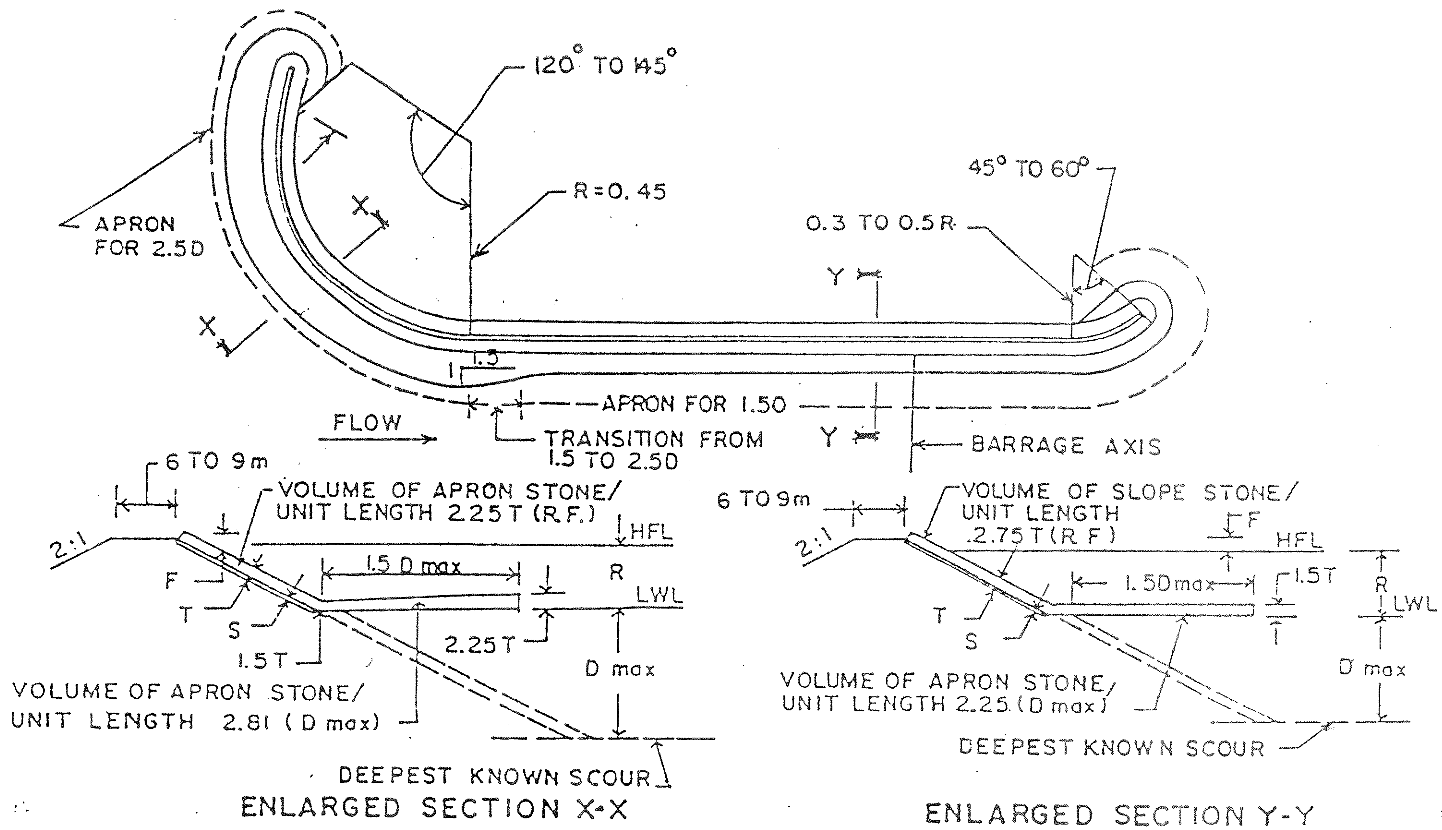
GEOMETRICAL SHAPE OF GUIDE BANKS

FIG.10·2



TYPICAL LAYOUT OF RIVER TRAINNG WORK

FIG. 10·3



LEGEND:

F Free board
R Rise of flood above low water level
D max Deppth of scour for culculation of apron stone
T Thickness of slope stone
S Thickness of soling (Filter)

DETAILS OF GUIDE BANK

FIG. 10-4

CURVES TO DETERMINE THE SIZE OF STONES NEEDED FOR SLOPE PROTECTION

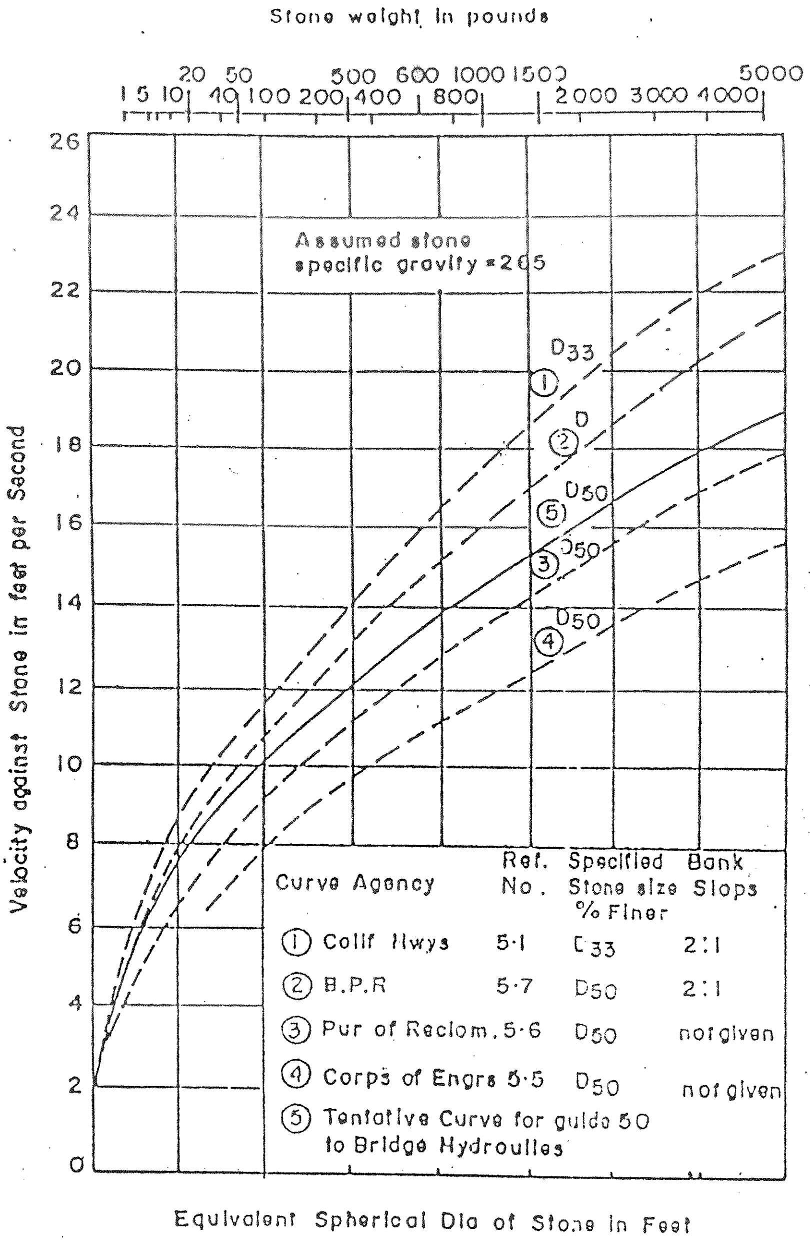
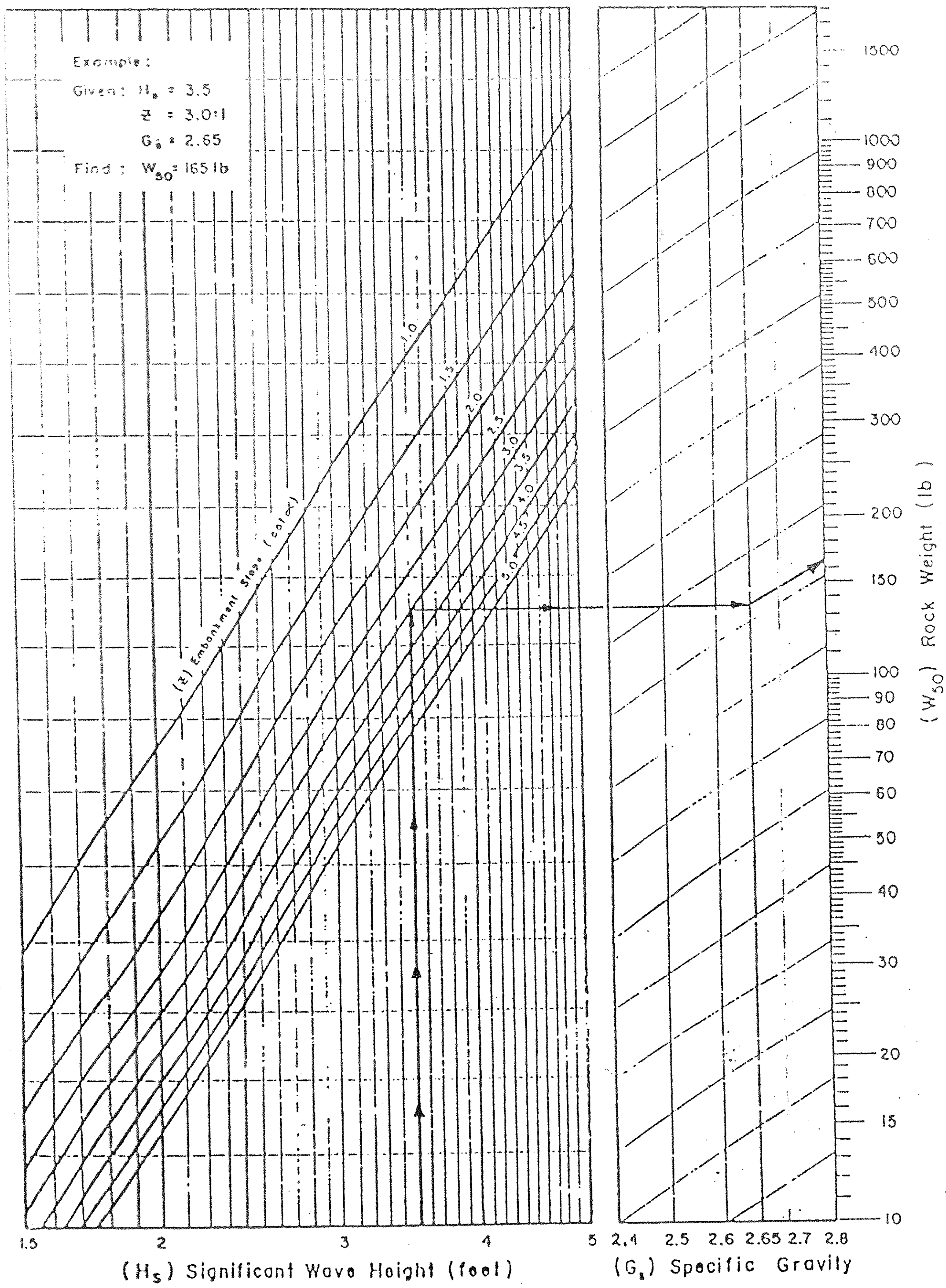


FIG.10-5



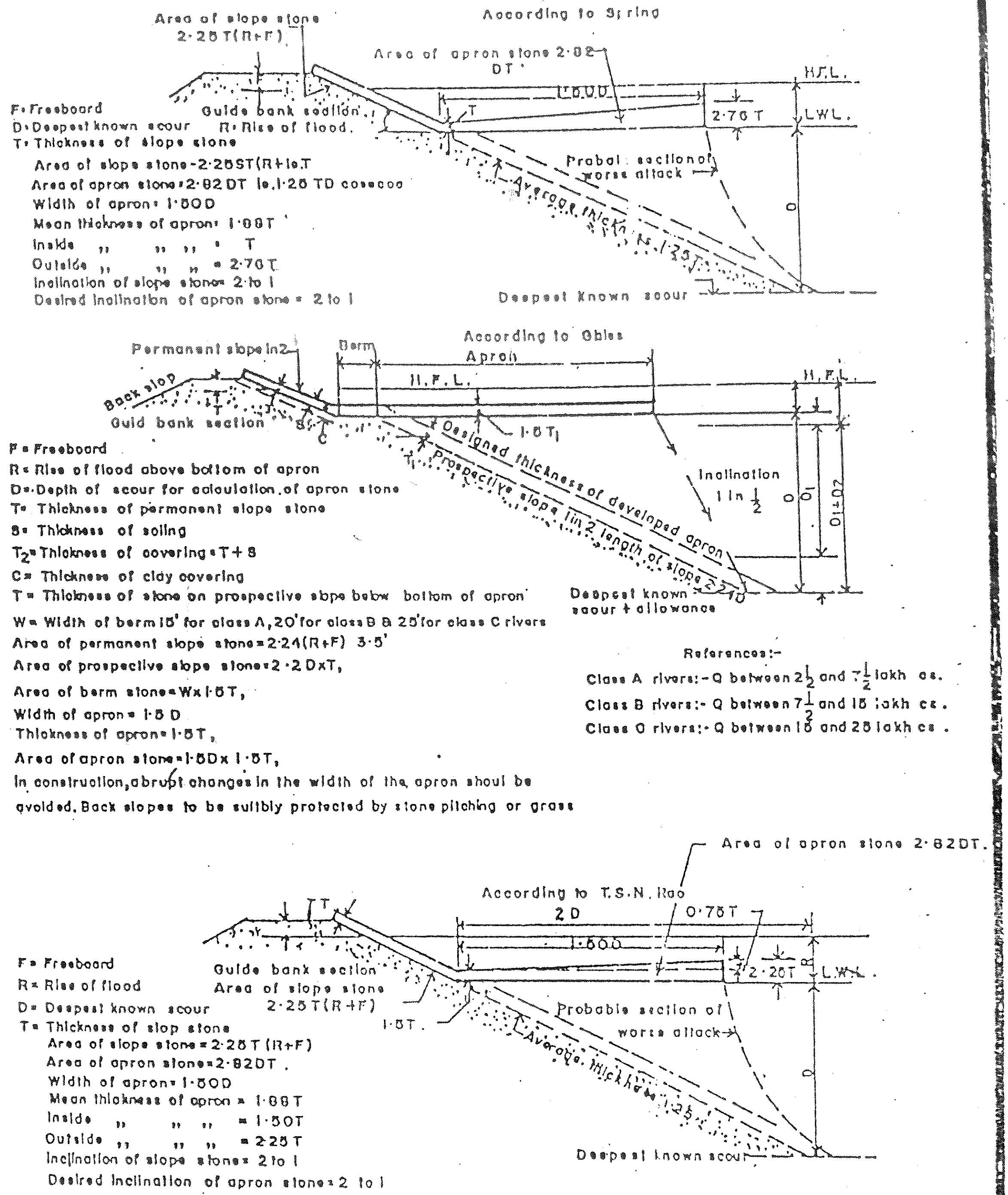
Equation (E 14); From Appendix E

$$W_{50} = \frac{19.5\ G_s\ H_s^3}{(G_s - 1)^3\ \cot\alpha}$$

FIG. 10-6

ROCK SIZE SELECTION

WNTC ENG. STAFF

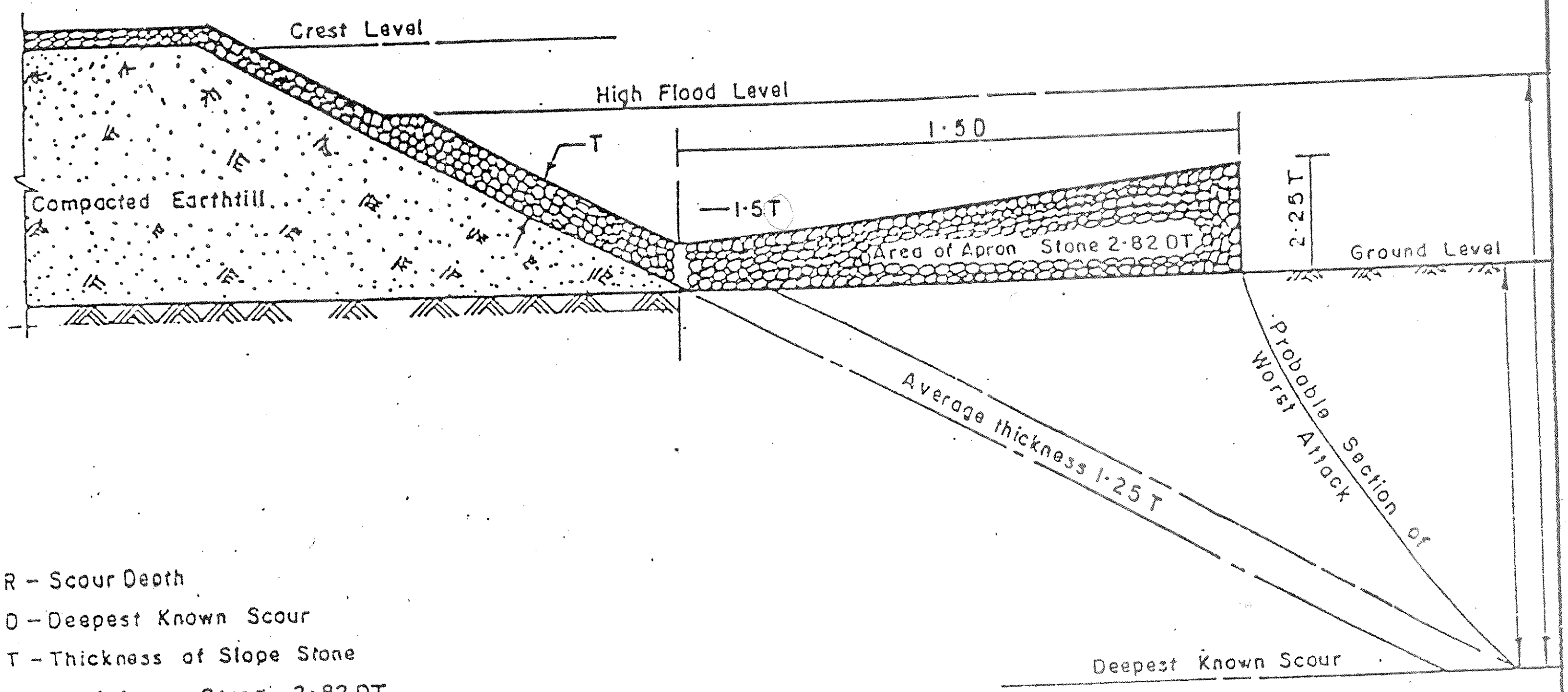


Dimensions of launching aprons according to Spring, Gales and Rao

(Source: Joglekar, 1971) FIG.10·7.

DIMENSIONS OF LAUNCHING APRON

(PROPOSED BY T. S. N. RAO)



R – Scour Depth

D – Deepest Known Scour

T – Thickness of Slope Stone

Area of Apron Stone – 2·82 DT

Width of Apron – 1·50 D

Mean Thickness of Apron – 1·88 T

Inside Thickness of Apron – 1·50 T

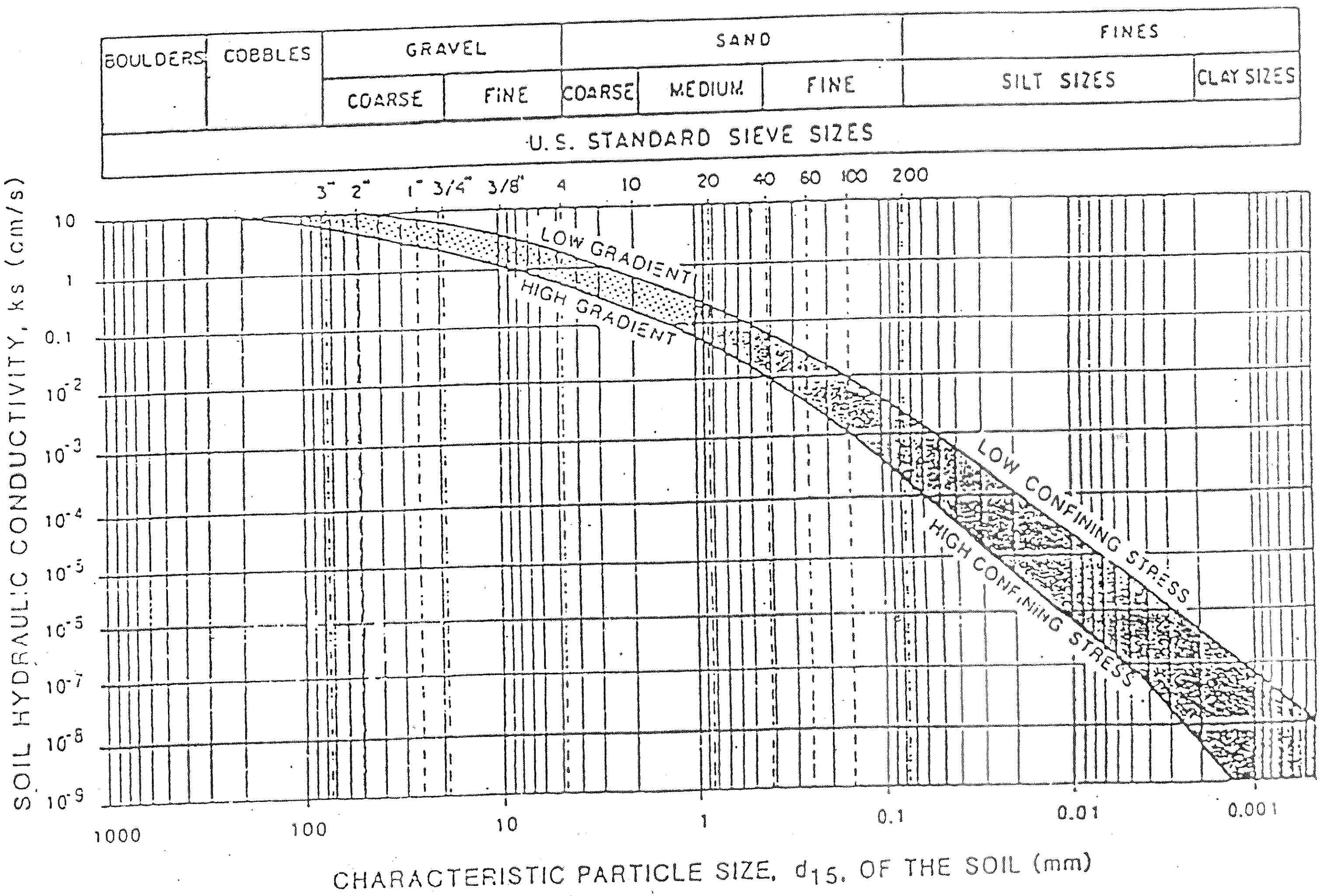
Outside Thickness of Apron – 2·25 T

Desired Inclination of Apron Stone 2 to 1

FIG–10·8

FIG.-10·9

TYPICAL HYDRAULIC CONDUCTIVITY VALUES



11.0 DESIGN CRITERIA OF GATES AND HOIST

11.1 General

To consider general design standards and criteria of hydraulic gates & hoists local physical, hydrological, climatic and operating conditions should be properly evaluated. The design standards and criteria of different types of hydraulic gates depend to a great extent on the functional and physical features of the gates.

11.2 Types of Gate

In the hydraulic structures for irrigation and flood control the following categories of gates are generally used.

A. Vertical lift gates

These gates are of two types:-

(a) Sliding Gates
(b) Fixed Roller gates

Type (a) and (b) are again sub-divided into four categories:

i. Single leaf gates
ii. Single leaf gates with flap
iii. Double leaf gates
iv. Double leaf gates with flap in the bottom.

B. Flap Gates

i. Simple flap gates.
ii. Flap gate with lifting arrangement.

C. Radial gates

11.2.1 Functional classification of gates.

Functionally the gates are classified as Crest gates, Stand by gates, Intake gates, Outlet gates, Spill-way gates, De-silting gates, Regulating gates, Lock gates, Tide gates, Counter flow preventive gates. Short descriptions are detailed hereunder:

[a] Crest Gates

Designed and installed to permit floods in dams to flow down safely.

(b) Stand by gates

Located in the inlet openings of discharge pipes in dams to be used for the servicing and inspection of orifice gates, discharge pipes etc.

[c] Intake gates

Required for the inspection and servicing of headrace channels, penstocks etc. in hydroelectric power plants.

[d] Outlet gates

Required for the inspection and servicing of hydraulic turbines, draft tubes etc. in hydroelectric power plants.

[e] Spillway gates

Required to permit floods to flow down safely.

[f] Desilting gates

Designed to sweep away sediments upstream of river weirs.

[g] Regulating gates

Designed to control upstream reservoir levels.

[h] Lock gates

Used for navigation.

[i] Tide gates

Used for the prevention of adverse tides.

[j] Counter flow preventive gates

Designed and installed where tidal lock gates are provided. This gate prevent the tidal gates from opening out when the lock is to be operated in reverse direction.

11.3 Selection Criteria of Type of gates.

The main purpose of a gate is to regulate or completely shut out the discharge entering or escaping from rivers or channels through hydraulic structures/regulators. Selection criteria of type of gates to be used are described below.

11.3.1 Sliding Gates.

These gates slide in vertical grooves enlarged in vertical direction. It consists of simple structures, supported in grooves between piers or abutments. No roller is attached to these gates. Structures having opening area of maximum 10m² may be selected for use of Slide gates.

11.3.2 Fixed Roller Gates.

For greater spans and higher heights to reduce friction, rollers are fixed to the gates at their ends. The improved designs of such gates now make it possible for them to be used for larger spans.

11.3.3 Flap gates.

Flap gates are generally used to control water flow in one direction only and to prevent reverse flow.

11.3.4 Radial Gates.

Radial gates are so named because they are made to the shape of a portion of a cylinder and rotate radially about a horizontal axis. Generally for greater span of regulators this type of gate is selected.

The following are the guide lines for selecting the type of gate to be designed for a particular structure.

SL NO.	TYPE OF GATES	CONDITIONS OF SELECTION
1.	SLIDE GATE	GATE SURFACE AREA $10 M^2$ MAXIMUM.
2.	FIXED ROLLER GATE	GATE SURFACE AREA IS MORE THAN $10 M^2$. PARTICULARLY USED FOR LOCK GATES.
3.	DOUBLE LEAF GATE	SPECIAL PURPOSE GATES. FLAP IN THE BOTTOM GATE IS PROVIDED WHERE REQUIRED.
4.	FLAP GATE	WHERE UNIDIRECTIONAL FLOW OF WATER IS TO BE CONTROLLED. LIFTING ARRANGEMENT IS PROVIDED WHERE REQUIRED.
5.	RADIAL GATE	FOR LARGER VENT OPENING, COMPARISON OF COST AND OPERATIONAL ASPECT ARE CONSIDERED WHILE SELECTING RADIAL GATE OVER FIXED ROLLER GATE.

For special cases, selection criteria for gates shall be separate.

11.4 General Structural Design of Hydraulic Gates.

11.4.1 Design method.

Design method consists of the following steps:

i. Hydrostatic load calculation at different conditions.

ii. Distribution of the hydrostatic load on the different members/components.

iii. To calculate and find out the dimensions of the sections on the basis of the load as stated in (ii).

iv. Working or allowable stress design method will be followed for the design of all steel or other member.

Allowable stresses of some commonly used materials for design and fabrication of different hydro-mechanical structures are given in Table 11.1 and 11.2.

Table 11.1

Allowable Stresses

MATERIAL	Working stress (Kg/cm²)		
	TENSION	COMPRESSION	SHEARING
STRUCTURAL STEEL	840	840	560
COLD ROLLED STEEL (RESULTING STRESSES DUE TO COMBINED AXIAL AND TORSION IN GATE STEMS)	840	840	700
COLD ROLLED STEEL (RESULTING STRESSES DUE TO COMBINED FLEXURE AND TORSION IN GATE STEMS)	700		560
CAST STEEL HARD	1050	1050	840
CAST STEEL MEDIUM	875	875	700
CAST STEEL SOFT	700	700	560
ROLLED TOBIN MANGANESE BRONZE	1120	1050	700
PHOSPHORUS OR GUN BRONZE	525	700	490
BRONZE NUTS ANCHOR BOLTS	420		105
PINS SAE 1045 STEEL (FACTOR OF SAFETY 4)	1600		

Table 11.2

Bearing Stresses (Kg/cm²)

MATERIAL	STRESSES KG/CM²
CAST IRON ON CAST IRON	25
CAST IRON ON CAST STEEL	30
BRONZE ON BRONZE	35
STEEL THREADS ON SHAFTS	525
PIN BEARING PRESSURE ON BRONZE BUSHINGS	250
BEARING PRESSURE ON CONCRETE GROUT	35
BOND BETWEEN ANCHOR BOLTS AND CONCRETE	60
ANCHOR BOLTS	420
BEARING ON CONCRETE GROUT	35

11.4.2 Factor of Safety

The most commonly used values for factor of safety based on the ultimate strength are listed in Table 11.3.

Table 11.3
Factor of safety

Material	Kind of Load				
	Steady Stress	Repeated Stress in one direction		Repeated stress, reversed	
		Gradual applied	Shock	Gradual applied	Shock
Brittle Metal C.I. and alloys	4	6	12	8	14
Ductile metal W.I. & Mild Steel	3	5	10	6	12
Cast Steel	3	5	10	8	15
Hard Steel	4	6	12	9	14
Copper and other soft metals & alloys	5	6	10	8	15
Timber	5	8	15	10	18
Leather	8	10	12	12	15
Masonry and Brick-work	10	15	20	20	30

11.4.3 Average properties of some Metals and Alloys

Properties of metals and alloys frequently used in design and fabrication of different types of hydro-mechanical structures, such as, gates, valves, conduits etc. are given in Table 11.4.

Table 11.4

Properties of Metals and Alloys

MATERIAL		UNIT WEIGHT, KG/M^3	MODULUS OF ELASTICITY E 10^6 KG/CM^2
1.	ALUMINUM	2720	0.70
2.	COPPER	8900	1.19
3.	LEAD	11350	0.14
4.	NICKEL	8900	2.10
5.	TUNGSTEN	19300	3.50
6.	BRASS, YELLOW HARD	8460	10.50
7.	CAST IRON GREY	7200	0.91
8.	CAST IRON DUCTILE	7200	1.75
9.	PHOSPHOR BRONZE	8800	1.12
10.	STEEL 0.2%C, HOT ROLLED	7830	2.10
11.	STEEL 0.2%C, COLD ROLLED	7830	2.10
12.	STEEL 1%C, HOT ROLLED	7830	2.10
13.	STEEL 1%C, HARDENED & TEMPERED AT 430^0 C.	7830	2.10
14.	STEEL 0.38~0.43%C, HARDENED AND TEMPERED	7750	2.03
15.	STAINLESS STEEL	7910	2.03
16.	EPOXY, GENERAL PURPOSE	1120	455 (TENSION)
17.	RUBBER, NATURAL (MOLDED)	920	--

11.5 Main Components of Gates

11.5.1 Main Components of Vertical Lift Gates.

(a) Main Horizontal beam
(b) Top & Bottom beams
(c) End or Side vertical beams
(d) Vertical sub beams
(e) Skin plate
(f) Side roller
(g) Side roller bush
(h) Side roller fixing shaft
(i) Guide roller
(j) Rubber seal etc.

11.5.2 Main Components of Flap Gates.

(a) Main Horizontal beam
(b) Top & Bottom beams
(c) End or Side vertical beams
(d) Vertical sub beams
(e) Skin plate
(f) Gate fixing lever
(g) Gate fixing pin
(h) Pin and lever fixing bracket
(i) Pin & Bush
(j) Rubber seal etc.

11.5.3 Main components of Radial Gates.

(a) Main Horizontal beam
(b) Top & Bottom beams
(c) End beams
(d) Horizontal sub beams
(e) vertical sub beams
(f) Skin plate
(g) Rubber seal
(h) Wire rope fixing bracket
(i) Side guide roller & fixing arrangement
(j) Arms
(k) Arm fixing boss
(l) Trunion pin
(m) Trunion fixing bracket
(n) Trunion girder
(o) Trunion anchorage girder, if necessary

11.6 Design Loads.

Following loads are considered depending on the type, size and the prevailing design conditions at the gate location.

11.6.1 Sliding Gates:

Static water load is taken into consideration because the gates are generally operated by manual means.

11.6.2 Fixed Wheel roller Gates and Radial Gates:

The hydrostatic load at each condition by established formula developed by different standards of gate design and by general hydraulic conditions are stated below.

(a) Maximum static water load at normal condition
(b) Hydraulic load at earth quake time
(c) Hydraulic load at wind time
(d) Earth pressure for siltation
(e) Load exerted by the wire rope tension on the different components (for radial gates only)

11.6.3 **Flap Gates:**

(a) Static water load at fully closed conditions
(b) Steady state loading
(c) Dynamic loading
(d) Transient loading

The loads (b),(c)&(d) are analysed at operating condition when water jet impinges on the gate which in turn causes extra load or reactions on the gate members as well as gate levers and pins.

After mathematical analysis of the loading conditions stated in (I), (II) & (III) the maximum load is found out. This maximum load is distributed on the different components of the gate. As per distributed load, the suitable sections of the components are selected on the basis of bending and shear stress relationship as the case may be. Generally, structural steels are used as main beams, horizontal and vertical sub beams, top, bottom and end beams etc. The allowable bending stress of structural steel is σ_b= 1000 to 1200 Kg/cm². The different sections such as angles, H-beams, I-beams, channels and in some cases flat plates are used as the main components of the gates. From the bending and shear stress relation, a suitable section should be selected in such a way that the gate is placed within the side grooves properly.

11.7 **Design Condition of Vertical Lift Gates:**

(i)	U/S or R/S	HWL	=	EL. H m.
(ii)	D/S or C/S	LWL	=	EL. H_1m.
(iii)	Invert level		=	EL ±
(iv)	Soffit level or gate height		=	a
(v)	Operating Deck		=	EL
(vi)	Gate span or vent width		=	b
(vii)	Gate height		=	a

(when gate height is not equal to soffit height).

(viii) Sealing system	:	water tight on four (At submergible condition) sides.
(ix) Hoisting system	:	Gear pedestalor hand wheel operated screw system.
(x) Corrosion allowance	:	1.5 to 2 mm.
(xi) Deflection on main beam	:	1/800 of span length.
(xii) Specific or unit wt.of water	:	1 ton/m³.

11.8 **Basic Equations for Load calculations.**

These equations are generally used for larger gates, such as, fixed wheel roller gates, navigation lock gates and radial gates.

11.8.1 Silt pressure on vertical lift gate

Co-efficient of silt pressure.

$$C_e = \frac{1-\sin\theta}{1+\sin\theta}$$

Where,

C_e = co- efficient of silt pressure.
θ = Angle of internal friction of silt deposits
= 20° (Generally taken in our country).

Horizontal force of silt pressure at any point.

$$P_e = C_e \; w_1 \; d \quad (t/m^2)$$

Where,

P_e = Horizontal force of silt pressure (t/m^2)
W_1 = Under water unit weight of silt deposit (t/m^3)
= 0.927 t/m^3 (Generally taken in our country).
d = Depth measured from silt deposit surface to any point (m).

Total silt pressure.

$$P_e = \tfrac{1}{2} \, . \, W_1 . \, h_e^2 . \, C_e . B \quad \text{in tons.}$$

where,

h_e = Height of silt pressure (m).
B = Span width (m).

11.8.2 Water Pressure during Earthquake.

Hydrostatic pressure during earthquake and dynamic water pressure during earthquake are considered as the hydraulic pressure acting on the gate leaf. (Fig. 11.1)

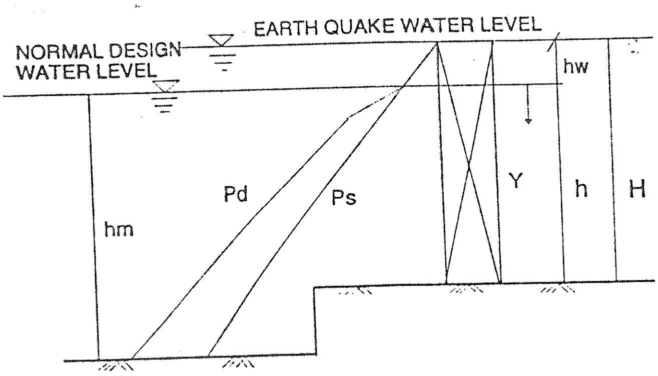


Fig. 11.1

(a) Hydrostatic pressure during earthquake

$P_s = \frac{1}{2} W(h_w+h)^2 . B$ tons.

Where,

h_w = Height of waves due to earthquake (m).

$= 0.0612 \sqrt{(V.F)} + 0.762 - 0.27 \sqrt[4]{F}$

where, F = Fetch in Km
V = Wind velocity (generally taken 110 Km/Hr in our country)

(b) Dynamic water pressure during earthquake

This dynamic water pressure shall be determined by the use of Westergard's formula.

$P_d = 7/8$ w.k. $\sqrt{(h_m . y)}$ Ton/ m²

P_d = Dynamic water pressure in water depth at point y.
k = Design seismic coefficient.
h_m = Distance from water surface to foundation rock during earthquake (m).

Total dynamic water pressure,

$P_d = 7/12 . K . h_m^{1/2} . h^{3/2} .B$ tons

(c) Total water pressure during earthquake

$P = P_s + P_d$.

11.8.3 Comparing water pressures at normal times and water pressure during earthquake.

Taking into account the fact that the allowable stress during earthquake is increased by 50% over that at normal times, the water pressures at normal times and those during earthquake at each point of gate leaf shall be compared and the case of highest load shall be taken as a basis for structural design.

11.8.4 Earth load on the gate for siltation (on radial gates)

(a) At normal time

Earth load is calculated by the formula.

$P_e = \frac{1}{2} C_e . W_e . H^2_e$ t/m

Where,

P_e = Earth load in ton/m.

C_e = Active earth pressure factor
= $Tan^2(45° - \theta/2)$

θ = Angle of internal friction = 20°.

H_e = Earth height in m.

W_e = Unit wt. of earth in water generally taken as 0.927 t/m^3

C_e = Tan^2 (45° - 20/2)
= 0.49 ≈ 0.5

P_e = ½ x 0.5 x 0.927 x H_e.
= 0.232 H_e t/m.

Total load on gate,

P_e = 0.232 H_e x B, Where, B = span width in m.

(b) Earth load on the gate during earthquake time

The earthquake magnitude on the land is K and the earthquake magnitude in water would be K'.

$$K' = \frac{K-r}{r-(1-\alpha)}$$

Where, r = Specific gravity of earth.
α = Void ratio of earth
K = Earthquake magnitude = 0.15

The angle of internal friction of earth within water decreases by an amount $\tan^{-1}K$ during the time of earthquake.
That is θ' = Angle of internal friction of earth with water during earthquake = $\tan^{-1}K$.

On the above consideration, co-efficient of active earth pressure in water during earthquake is

$$C_e = Tan^2\{45 - ½(\theta - \theta')\}$$

$$= Tan^2\{45 - ½(\theta - \tan^{-1}K')\}$$

$$= Tan^2\{45 - ½(20 - 8.53)\}$$

$$= 0.60$$

So, P_{e-e} = ½ x 0.6 x 0.927 H_e ton/m
= 0.2781 H_e ton/m

Total, P_{e-e}= 0.2781 H_e. B tons

11.9 Selection of skin plate thickness (by Timoshenko's formula).

The maximum bending moment per unit length when the periphery clamping plate is subjected to uniformly distributed load is produced in the centre of each side.

$M_1 = \alpha . h . a^2 \quad t.cm$
$M_2 = \beta . h . b^2 \quad t.cm.$

Where,

a = Length of short side of plate (cm)
b = Length of long " " (cm)
h = Uniformly distributed load (t/cm^2)
α, β = Co-efficient determined by b/a given below.

B/A	1.00	1.20	1.40	1.60	1.80	2.00	>2.00
A	0.0513	0.0639	0.0726	0.0780	0.0812	0.0829	0.0833
B	0.0513	0.0554	0.0568	0.0571	0.0571	0.0571	0.0571

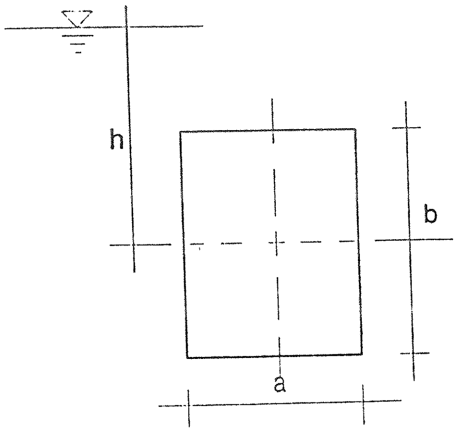


FIG. NO-11.2

Bending moments calculated from the above relation would be different for different panel loading. The maximum bending moment should be taken as design maximum bending moment. Having the maximum bending moment the thickness of the skin plate is to be calculated from the following relation.

Bending stress $\qquad \sigma_a = \dfrac{6 . M_{max}}{t^2} \qquad ton/cm^2$

Considering allowable bending stress σ_a = 1200 kg/cm^2 the thickness t shall be calculated. 2 mm corrosion allowance shall be added to the calculated value to have, the actual thickness of the skin plate to be used.

11.11 Design of main beam section(for vertical lift gates)

Bending moment, $M = \frac{W(2l-B)}{8}$ *t-cm.*

Where,

W = Hydraulic load exerted in ton.
l = Effective span between rollers in cm.
B = Width under pressure in cm.

Bending stress, $\sigma_b = M/z$ (t/cm²)

where,

z = Section modulus for the main beam in cm³.

Shearing force.

The maximum shearing force is produced on the supports as shown below:-

F = W/2 ton.

Shearing stress,

$\tau = F/A_{web}$. (t/cm²)

Where,

A_{web} = sectional area of webs at supports (cm²)

Deflection:

Deflection in the span center is obtained approximately by the following formula.

$$\delta = \frac{5}{384} X \frac{wl^4}{E.I} \quad cm.$$

where,

E = Modulus of elasticity.
= 2.1 x 10^6 kg/cm² for structural steel.
I = Geometric moment of inertia for the main beams (cm⁴)

Degree of Deflection,

$\Lambda = \sigma/l < 1/800$, which is allowable deflection as stated earlier.

When I beam is used as main beam then the following dimensional criteria are followed (fig. 11.3):

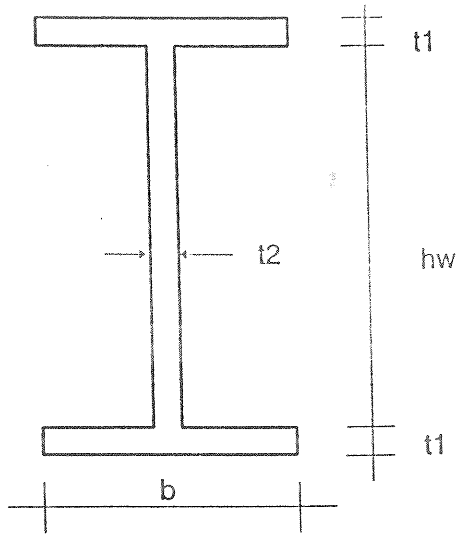


FIG. NO.-11.3

Compression flange

$b \le 24\ t_1$

Tension Flange

$b \le 30\ t_1$

When vertical stiffeners are not provided then,

$h_w \le 60\ t_2$ for material

SS41, SM 41 (as per JIS)

$h_w \le 55\ t_2$,

When vertical stiffeners are provided

$h_w \le 160$ t2.......... SS41

$h_w \le 136\ t_2$ ....SM 50(JIS)

When proceeding with design work, close attention must be given to the location and number of the main beam in view of the correlation between the beams and location and number of the rollers.

11.11 Design of roller

Calculation for the rollers shall be made using the following formula

$$P_a = KBD > P$$

where,

P_a= Allowable load for the roller(kg)
P = Working roller load (kg).
K = Coefficient of roller wheel pressure(kg/cm^2).
B = Effective roller tread width (cm).
D = Roller diameter (cm).

The coefficient of roller wheel pressure, k, is proportional to the roller hardness.

$$K = 0.0015(H_B)^2 \text{ kg/cm}^2$$

Where, H_B is the hardness of the roller material.

The type of roller used for hydraulic gates is heavy in load and rotates at a low speed, differing in feature and performance from commonly used bearings.

While the liquid lubrication system was employed formerly, according to which a bronze bush is set in a roller and grease is forced in, in recent years oilless bearings have come to be used more widely. The reason why the oilless bearings of the solid lubrication type has come to win popularity is that the liquid lubrication system does not permit, sufficient distribution of lubricant to the shaft assembly because of heavy load and low-speed rotation.

11.12 Design of Roller bush.

The allowable bearing pressure for the roller bush are standardized as follows :

For generally used grease lubrication.

$$P/(D.B) \leq 150 \text{ kg /cm}^2.$$

where , p = Imposed load on each roller, kg.
D = Diameter of the roller shaft, cm.
B = Length of the bush or width of the roller, cm.

For the oilless bearing.

$$P/(D.B) \leq 250 \text{ kg/cm}^2$$

The above values are also applicable to the trunnion pin of the radial gates.

The use of an oilless bearing makes it a rule not to lubricate, and there may be cases where lubrication is done only to lower the efficiency of a bearing.

Since the built-in lubrication system is not effective in rust prevention, for the bearing of a roller shaft it will be necessary to select such materials for which corrosion protection is taken into consideration, including the use of stainless steel, chromium plating etc.

11.13 Physical and fixation criteria of the roller wheel.

The fixed roller wheels shall be symmetrically positioned about the vertical centre axis of the gate.

The tracks shall be slightly crowned in order that the point of contact remains in the central portion of the track even though the side beams have rotated slightly due to the bending of the main beams. Hence, the stresses shall be analyzed for point contact.

The design of the wheel involves a tri-axial stress condition. The Hertzian contact stresses shall be analyzed according to the procedure outlined in the IS 4622:1978 [3,7]

The following conditions shall apply:

(a) The max shear stress shall be less than 240% of Brinell Hardness Number (BHN) of the track

(b) In estimating the wheel load W a 50% overload factor shall be adopted to cater for misalignments in the wheels.

(c) The permissible point contact stresses shall be 240% of the ultimate tensile stress:

The minimum wheel tread, T is obtained from the expression

$$T = \frac{W}{R_1(0.169BHN-15.174)} \quad \ldots\ldots (1)$$

Where, the wheel radius R_1, is determined from physical considerations or from the diameter D as shown in section 2.10. This expression allows a factor of safety of 2 on the bearing stress.

Let R_2 = Radius crowning of wheel (or track)
E = Modulus of elasticity
μ = Poissions ratio (=0.30)
α = Depth of max shear stress
a = Semi major axis of ellipse of contact

$$\Omega = \frac{4R_1R_2(1-\mu^2)}{E(R_1+R_2)} \quad \textit{Elastic and shape factor}$$

K = b/a and

τ_{max} = Maximum shear stress.

The contact stress ρ_{max}. is determined by:

$$\rho = 1.5\ W/(\pi.a.b)$$

Then, the condition that must be satisfied in the wheel design are as follows:

(a) The wheel tread is greater than the result of Equation (1).
(b) τ_{max} < 2.41 BHN
(c) ρ_{max} < allowable stress (240% of ultimate tensile stress) and
(d) Depth of penetration of hardness > 2 α

11.14 Wheel tracks

Wheel tracks shall provide a smooth surface for the wheels to roll. The hardness of wheel track surface shall be kept upto 50 points BHN higher than the wheel to prevent excessive track wear

The bearing stress in the second stage concrete, P_c is found from equation

$$P_c = \frac{0.2813 W E_c}{E_s I w^2}$$

Where,

E_c and E_s are the modulus of elasticity in concrete and steel respectively,

I = second moment of area of the track and
w = width of track base.

The distance of track centre line from the inner face of the regulator barrel shall be checked for shear. The second stage concrete mix design shall be specified by the engineer in charge.

The length of influence in the concrete from each wheel is given by:

$$L = 1.5\ W / wP_c$$

If L is more than the distance between wheels, there will be an overlap of the pressure distribution under adjacent wheels, and stresses must suitably superposed for the concrete design.
In bending , the stress in the wheel track shall be determined from Equation:

$$\sigma = \frac{0.5 W E_s I}{z E_c w}$$

Where, Z is the section modulus, cm^3.

11.15 Impact stress factor of Flap gates.

It shall be assumed that the dynamic load caused by the hydrostatic and dynamic loading would be applied simultaneously. Hence, the components of loads caused during dynamic equilibrium and transient loading on the flap gates are to be combined (as per working paper 4).

$$R_x = (1.5 \sin\theta \cos\theta + 0.680 \cos\theta + 2.134 \sin\theta).W \quad \text{.........(1)}$$

and

$$R_y = (1 - 1.5 \sin^2\theta - 0.680 \sin\theta + 2.134 \cos\theta).W \quad \text{.........(2)}$$

where,

R_x and R_y are the horizontal and vertical forces acting on the pin.
W is the self weight of the gate

The resultant is given by

$$R = (R^2_x + R^2_y)^{1/2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

R is found to have a maximum value of approximately 3.11 W. This value shall be used for assessing the dynamic loading on the pin. Considering the fact that this force is transmitted to the pins of total weight W_p through the impact of the gate of weight W, the actual impact force imparted would be several fold magnified due to elastic effect.

Assuming the conservation of kinetic and strain energies, this impact force R_i can be estimated to be

$$R_i = 3.11 W \left[\frac{W}{W_p} \left(\frac{3W}{3W + W_p} \right) \right]^{1/2} \qquad (4)$$

As an illustrative example for a standard gate of 1.52 X 1.83 m (5′ x 6′) estimated weight of 385 Kg (850 Lbs) having 2 pins of total weight 0.75 Kg (1.65 Lbs) , the impact loading on the pin is found to be 266 KN (59840 Lbs) or approximately, 70 times the weight of the gate.

Since the exact gate weight is not known before design, the expression:

W = A(34 + 0.057 A) may be used as a first estimate of the gate weight for design purposes.

where,

W = weight in Lbs &
A = Area in ft^2

11.16 Design Condition of Radial Gates.

(a) Design data required

(1)	Type :		Steel Radial gate.
(2)	Gate height	=	H in m.
(3)	Gate Radius	=	R in m.
(4)	Design head	=	H_2 in m.
(5)	Free board	=	H_1 in m (Generally taken 0.2 m to 0.35 m)
(6)	Sill level	=	EL ± a in m.
(7)	trunnion pin level	=	EL ± b in m.
(8)	Sheet pile sand height	=	EL ± d in m.
(9)	Wind velocity	=	70 to 100 miles/hour.
(10)	Water tight system	=	Three or four side rubber water tight system.
(11)	Hoisting system	=	Depending on hoisting load it may be operated electrically or manually or by both systems.
(12)	Hoisting speed	=	0.30 m (approx.) per min.
(13)	Wind pressure	=	Max. 50 Lb/ft^2
(14)	control system	=	Local and remote control or either one depending on the situations prevail at site or as per requirement of project planning.
(15)	Power source	=	380 to 400 Volt, 50 Hz
(16)	Safety factor against wire rope	=	More than 8 against straight tension.

(17) Bearing stress on concrete = 550 psi

(18) Main material of gate leaf and arms = Structural steel of SS41 grade.

(19) Seal material = Stainless steel of SUS304 grade.

(20) Span width = B in m.

(b) *Relationship of Radius and Height of Radial gate*

The radial gate geometry concerns the relationship of the skin plate Radius R to the gate height H, the locations of the bottom seal, bearing Centre, lifting gear and the angle of approach of the skin plate to the bottom sealing surface.

No readymade formula can give the best solution to a particular gate layout. However, for most of the crest spillway gates so far designed the value of R lies within the ranges R = 1.3H as per BS standard. This relationship varies from country to country. Some of which are stated below.

As per Hydraulic Gate and Penstocks Association of Japan,

$$R = H(1 \text{ to } 1.4)$$

As per Indian Standard of Radial gates

$$R = 1.25H$$

In our country specially in BWDB it is general practice to select a suitable value of R from the above stated ranges considering the structural features where the gates would be used and the hydrological conditions at the upstream and downstream of the gates.

(c) *The following factors are to be considered in the layout of radial gates.* (figure 11.4)

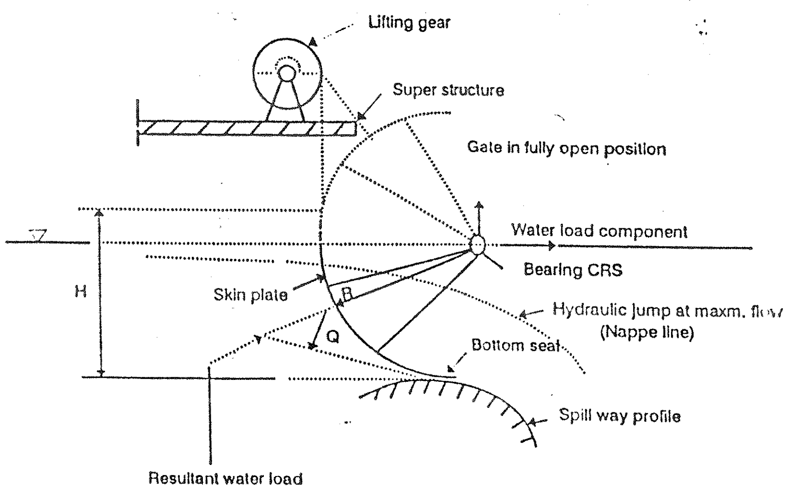


FIG. 11.4

[1] The level of the top of the skin plate is fixed by the max. water retention level and free-board that may be required to counter the waves. The bottom of the skin plate must seat on or downstream from the spillway crest to give a downward direction to the flow under the gate to ensure positive water pressures on the spillway profile, particularly at small gate openings.

[2] For structural reasons the radius R should be small. However, to avoid undue scuffing of the gate bottom seal during final closure, the angle should not be too acute. A reduction in R increases the up and downstream fleeting angle of the hoist ropes. Both of these factors are affected by R and by the location of the bearing centre.

[3] The pivot bearings should be well clear of the <u>nape</u> at maximum flow to avoid interference with discharge and possible damage from any debris that may be carried over. The position of the bearing relative to the crest determines the angle of the resultant water load on the anchorage.

[4] If a lifting gear utilizing wire ropes attached to the upstream face of the skinplate is used, the superstructure on which the gear is mounted can be kept low and hoist loads kept to a minimum.

The superstructure be positioned sufficiently far upstream to clear the raised gate. The effect of lifting gear position on the inclination of the ropes and the increase in lifting effort that may be required are to be considered.

[5] Also a special attention is given to determine the stability of the gates at fully closed and opened condition. Two types of moments act upon the gate viz. stabilizing moment and overturning moment. Sometimes at fully raised condition if the air flow at 100 to 160 km/hr or earthquake occurs then the overturning moment may be higher than the stabilizing moment for which the gate may hurt the superstructure or the roadways to D/S.

(d) Design and selection criteria of some main components of Radial gates.

[1] Gate leaf:

The function of the gate leaf is to transmit the hydraulic load imposed on the skin plate from the main beams to the trunnion pins through the auxiliary beams and via the arms. So the gate leaf must be rigidly constructed to be capable of resisting the frictional moment in the trunnion pins exerted during opening and closing operation. Further more, the requirements to be met are water tightness in the fully closed position and freedom from hindrances during discharge in a partially opened position, such as excessive vibration and other similar troubles.

[2] Skin plate:

Skin plate or the face plate is one of the main components of the gate leaf. It is made concentric to the pin and hence the resultant of the water pressure passes through the pin creating no moment to be overcome in hoisting the gate. This is the fundamental principle of the gate.

To allow for corrosion, the skin plate should have a minimum thickness of 6 to 8 mm. The panels of the skin plate created by the different horizontal and vertical stiffeners supported along the edges of the horizontal beams and vertical stiffener shall be assumed to be loaded uniformly by the hydrostatic pressure which act at the centroid of the respective panel. In the analysis, the edges are assumed to be rigidly fixed. This plate shall be analysed for two situations, i.e., its behaviour as panels with fixed supports as well as its bending as a flange member in combination with the main beams and stiffeners.

[3] Arms:

The geometric location of the gate arms must be such as to obtain approximately equal strut loads.
The individual arms struts are primarily loaded as a column but also carry high secondary stresses induced by the angular rotation of the joints at the gate leaf and arms. The arms are designed as a column with an l/r ratio equal to or less than 120 where r is the least radius of gyration. The lower arm approximately carries a load of 0.31w and the upper arm 0.21w, where w is the water load on the gate. The two members are connected to a hub. The arms must be designed to also absorb major bending and shear stresses.

[4] Pin Anchorage

Arms loads are transmitted through bearings into protruding trunnion pins and depending on the pier conditions, to heavy anchor girders or castings which are grouted in place. The trunnion pins must be designed for major bending and shear stresses and secondary shear stresses. Horizontal and vertical pin anchorage component forces are further absorbed by heavy anchor bolts which are mounted in two principal directions. These anchor bolts are of sufficient length to develop full bond strength and are formed to distribute the respective loads in the mass concrete.

So the effective operation of gates and seals depends to a great extend on accurate alignment of pin anchorages and bottom sill and side wall plates. Therefore, ample adjustment provision prior to as well as during gate erection are necessary.

(e) Design data used as standard and criteria of Hydraulic gates

On the basis of discussions made in articles 11.1, 11.2, 11.3, 11.4 regarding the functional and physical classifications of different hydraulic gates the following data are considered as design standard and criteria. These data are based on the foregoing discussions as well as various international standards covering this field and the standard engineering practice.

[1] Design Head (H).

To ensure satisfactory and maximum hydrostatic load calculation which imposed on the hydraulic gate at different periods of the year water level and discharge data are first analysed statically. To perform this work properly the hydrological data may be obtained from data files of surface water Hydrology Directorate-1. To determine the hydrostatic head of the structure, the hydrological data are analysed at different climatic and operating conditions for various return periods.

The following modes are generally considered depending on the structural and project requirements.

(i) Maximum flood by pass.
(ii) Drainage
(iii) Flushing.
(iv) No flow.

Analysing the water heads at above stated modes the critical condition or the maximum heads condition is sorted out. There are also exceptions in some cases or project such as hydro electric power plant and pump house. In these cases the maximum water retention head calculation methods differ widely. In hydroelectric power plant the maximum head of the water under which the turbines operates with full capacity is taken as design head of the spillway gates. In the pump house dry draft tube condition is taken as the maximum head of water when the gate is fully closed.

[2] **Gate height.**

The gate height depends on the type of gate used in the different structures. In the case of surface types gates where the head is not greater than the gate height, the gate height is determined as follows:

$$H = H1 + l1$$

where,
- H = gate height in mm.
- H1 = maximum water head in mm.
- l1 = Free-board in mm.

The value of free board depends on the type of gate and height of gate etc. In the case of sliding gate the free-board is generally taken as 100 to 200 mm but in the case higher height gates such as fixed roller and radial gate this value is generally taken as 150 to 300 mm. The value of free-board mainly depends on the span length and the height of the gates.

11.17 Arrangement Criteria of Main Beams of Radial Gate. (figure 11.5)

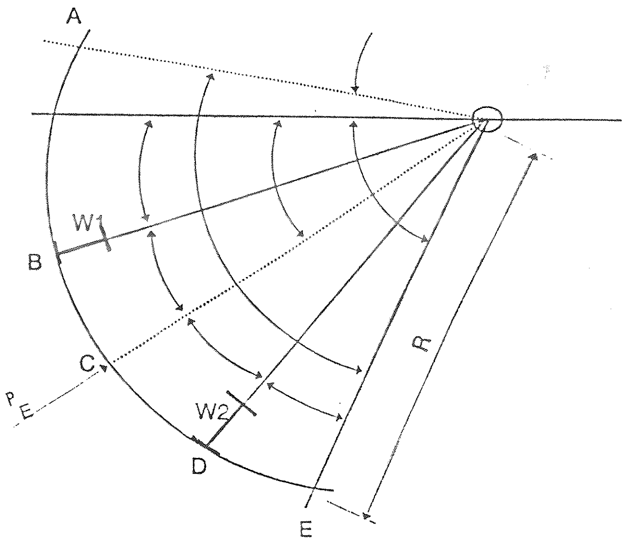


FIG. 11.5

Generally two numbers of main beams are used which would carry the total hydrostatic and other loads.

Each main beam is supported at two points by the gate arms. In terms of total arc length, the distance of the center lines of the lower and the upper arms or main beams are measured from the sill, along the inside surface of the face plate , which are as follows:

Arc(DE) = 0.1 to 0.13l
Arc(BD) = 0.5l
Arc(AB) = 0.3 to 0.37l where l = total arc length
Arc(AE) = l.

This spacing gives the maximum moment on the intermediate portion of the beams between the arms equal to, though of opposite nature to the moment at the supports and conduces to most economic design of the side beam which is treated as a condition.

11.18 Position of main beams and arms in Radial gates. (figure 11.6)

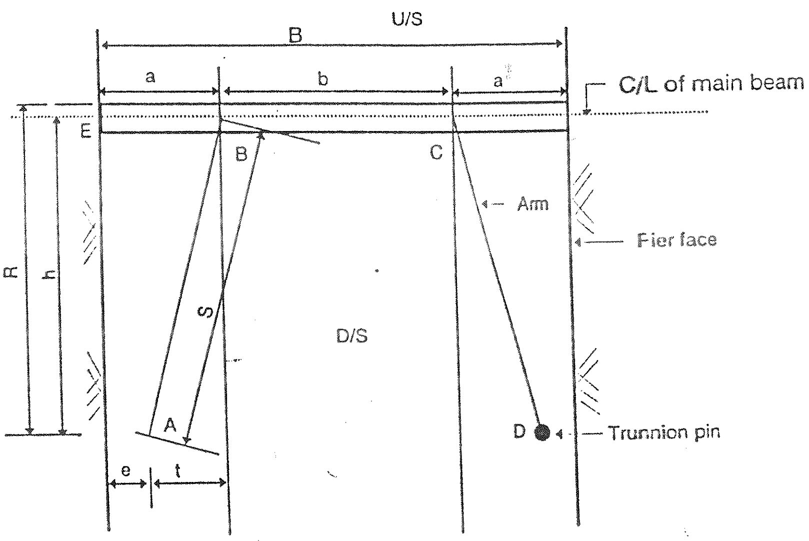


FIG. 11.6

At first we are to select the supporting position of the main beams in such a way that the deflection angle in that position becomes zero (0,0) and as a result of which we shall make structure so that the arms do not exert any bending moment owing to water pressure.

Therefore, in the above configuration the position of the different members are arranged as follows:

There is a numerical relation between the distance 'a' and total length of the main beam of the span length 'B' which is as,

a = 0.225 B. (The cantilever portion of the main beams)

e = 150 to 200 mm taken from experience and considering the size of the arm section and trunnion assembly such as bracket, base frame and the horizontal girder anchored inside the pier concrete.

$l' = a - e$

So arm length, $S = \sqrt{(l'^2 + h^2)}$

Where,

h = radius of the gate (R) - 1/2 of main beam section height.
l = length of the simply supported portion of the main beam.
= B -2a.

11.19 Allowable stress of main beams and arms of Radial gates.

a. *Main Beam:*

Generally the structural steel of SS 41, SM 41, SM 50 (as per JIS) etc. are used for main beam materials depending on the thickness of the flange and the web.

If thickness , $t \leq 16$, then

Yield stress, $\sigma_y = 25$ Kg/ mm² for SS41
$\sigma_y = 25$ Kg/ mm² for SM41
$\sigma_y = 33$ Kg/ mm² for SM50

If 't' is from 16 to 40 mm then,

$\sigma_y = 24$ Kg/ mm² for SS41
$\sigma_y = 24$ Kg/ mm² for SM41
$\sigma_y = 32$ Kg/ mm² for SM50

If t' > 40 mm then all the above stated values shall be less.

So, axial tensile stress, $\sigma_t = 0.6\ \sigma_y$ Kg/cm² and

Bending stress $\sigma_b = 0.60$ to $0.75\ \sigma_t$ Kg/cm²

We frequently use the following values for design

$$\sigma_t = 0.6 \times 24 \text{ Kg/mm}^2$$
$$= 1440 \text{ Kg/cm}^2$$

$$\sigma_b = 0.7 \times 1440 \text{ Kg/cm}^2$$
$$= 1000 \text{ Kg/cm}^2$$

Allowable shearing stress

$$\tau_a = 0.45 \times \sigma_t$$
$$= 0.45 \times 1440 \text{ Kg/cm}^2$$
$$= 648 \text{ Kg/cm}^2$$
$$\equiv 700 \text{ Kg/cm}^2$$

b. *Allowable stress of the Arm:*

Material: Structural steel of SS41 grade (t > 16 mm)

Yield stress, $\sigma_y = 2400$ kg/cm²

Allowable tensile stress, $\sigma_t = 0.6 \times \sigma_y$
$= 1440$ kg/cm²

But the arms experience compressive stress which shall be obtained as follows:

If, $l/b \leq 9/K$, then $\sigma_c = \sigma_t = 1440$ kg/cm²
If, $9/K < l/b \leq 30$, then $\sigma_c = 1440 - 11\ (K\ l/b - 9)$

where,

l = supporting length of the flange in cm
b = Flange width in cm
K = $\sqrt{(3 + A_w/2A_c)}$
A_w = Section area of the web plate, cm²
A_c = Section area of the flange plate, cm²

However, in case of $(A_w/A_c) < 2$ then, $K = 2$.

Knowing the above information from physical condition of the section and material, the allowable stresses are obtained.

11.20 Allowable stress of Trunion Pin & Bush.

a. Trunion Pin

Allowable bending stress,

$\sigma_b = 0.6\ \sigma_y$ where,

σ_y = Yield stress of the material used. Generally stainless steel of grade SUS 304 (JIS) is used as Pin material. There fore, σ_y is equal to 2100 kg/cm².

$$\sigma_b = 0.6 \times 2100$$
$$= 1260$$
$$\equiv 1200 \text{ kg/cm}^2$$

Allowable shearing stress, $\tau_a = 0.6 \times \sigma_b = 0.6 \times 1200 = 720 \equiv 700$ Kg/cm².

b. Pin bush

Surface pressure on the bush bearing

$$P_b = R / d_i.B$$

Where, R = Reaction force in kg,
d_i = Diameter of pin.
B = Length of bush.

$P_b < \sigma_s$ where, σ_s = Allowable surface pressure of the bush material
= 250 Kg/cm² (Considering the material oilless metal, no. 500 Spl as per JIS)

11.21 Rubber seal.

(a) Physical and Mechanical properties of rubber:

To prevent leakage of water, seals are provided at bottom and the sides, on the top as well where the gate is submerged. The seals are of various types in shape viz-P, Y, L and flat plate.

Also they are of many types in respect of materials from which they are fabricated or made from such as:

(1) Direct metallic contacts.
(2) Powered metal seals.
(3) Wooden seals.
(4) Rubber seals.

Whenever greater water tightness is required, rubber seals are most commonly used. The rubber lasts longer in wet situation and under low temperatures.

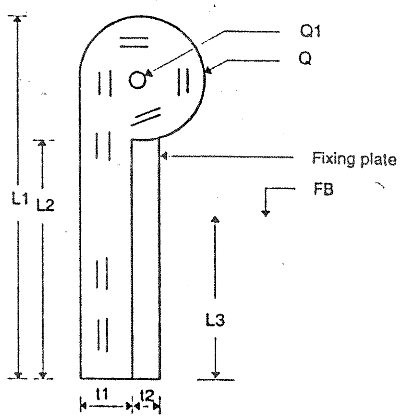
Rubber seals shall be molded solid section of Jor musical note type without any canvas inclusions. The material shall be compounded of natural rubber or a copolymer of butadiene and styrene, or a blend of both and shall contain reinforcing carbon black, zinc oxide, accelerators, anti-oxidants, vulcanizing agents and plasticizer. The physical characteristics shall meet the following specifications.

Tensile strength	: 20 N/mm^2
Elongation at break	: 45%
Durameter hardness (shore type A)	: 60-70
300% modulus	: 6 N/mm^2
Water absorption (max.)	: 5% by weight.
Compression set (max.)	: 30%

Tensile strength after oxygen
Bomb Aging ASTM D572 80% of tensile strength.
Tensile strength of vulcanized joints : 7 N/mm^2

(b) Nominal sizes of rubber seal being used:

There is no standard size of rubber that may be frequently adopted during design and drawing. But it is the general practice in BWDB that in most of the cases irrespective of vertical lift gate or radial gates 'P' type rubber seal of different sizes are used. In some special cases such as higher size roller or navigation lock gates 'Y' and 'L' type are provided for proper accommodation of the other parts of the gates into the gate groove. Also flat plate type rubber seals are sometimes used at the bottom of the vertical lift gates. The sizes i.e. the length and thickness of these seals are selected according to the sizes and design water head. So excepting 'P' type other types may not dimensionally be fixed as standard. For quick design and selection of the rubber seals, the following 'P' type rubber seals of different sizes may be provided.



"P" TYPE RUBBER SEAL

Fig. 11.7

(Ref fig.11.7)

TYPE	SIZE	PHYSICAL DIMENSION, MM.							REMARKS
		Q	Q_1	L_1	L_2	L_3	T_1	T_2	
P	A	50	20	100	50	25	20	6	M-8 FIXING BOLT(FB) @100 C/C MAY BE USED
P	B	40	15		60	30	15	6	,,
P	C	25	10	75	50	25	10	6	M-8 COUNTER SHANK HEAD BOLT WITH HEXAGONAL NUT @100 C/C MAY BE USED.
P	D	20	5	50	30	15	10	6	,,

11.22 Design of Hoisting Systems

11.22.1 General.

Hoisting system for vertical lift gates may be either two drum wire rope hoists or of hand wheel operated screw type depending on the hoisting load. These systems are generally operated manually, subject to the condition that the effort of a man or two allow the gate to be lifted easily. Motor operated multi stage gear reduction with a worm reducer is provided for higher lifting load.

A single point of hoist would be adequate for the smaller size of gates. For larger width, the gate shall be hoisted from two symmetrically located points.

Design considerations, procedures and calculation methods for the following types of hoisting arrangements are discussed.

(a) Pedestal type hand wheel operated screw system.
(b) Pedestal type hand lever with bevel gear set.
(c) Hand wheel operated two drum with spur gear and a worm reduction unit.
(d) Motor operated multi-stage spur with a worm reducer system etc.

11.22.2 Calculation of hoisting loads.

Besides the weight of the gate, the forces to overcome during hoisting the gate will be different from the force required to lower due to frictional and dynamic loads that come into play. The worst combination shall be considered for the design.

The loads that are relevant in the above considerations are the following:

11.22.2.1 Hoisting load of sliding gate.

(a) Self weight of the gate W_1 tons.(Including leaf, beams and other accessories)
(b) Load due to rubber seal friction (W_2)

(c) Water pressure on the top of the gate (if submergible) (W_3)
(d) Frictional force due to silt deposits (F_e)
(e) Buoyancy (F_b)
(f) Load during opening operation (Hoisting load)

$$W_h = W_1 + W_2 + W_3 + F_e + F_b$$

(g) Load during closing operation (Lowering load).

$$W_h = W_1 - W_2 - W_3 - F_e - F_b$$

For proper lowering $W_1 > W_h$ (If wire rope hoist is used).

11.22.2.2 Hoisting load on fixed wheel roller gate.

(a) Own weight of Gate leaf and ballast (Since a closing force of more than 20% of the own weight of gate leaf is desirable, in case of an insufficient closing force, ballast shall be placed on the gate leaf) W_1
(b) Friction force due to roller movement (F_w)
(c) Friction force due to rubber seal (F_r)
(d) Frictional force due to silt deposit (F_e)
(e) Buoyancy force (F_b)
(f) Uplift and down pull force due to overflowing water (F_u)
(g) Uplift and down pull force during discharge at the lower end (F_d)
(h) Load during operation (hoisting load)

$W_H = W_1 + F_w + F_r + F_e - F_b \pm F_u \pm F_d$ { '+' down ward load, '-' upward load.

(i) Load during closing operation (lowering load).

$$W_h = W_1 - F_w - F_r - F_e - F_b \pm F_u \pm F_d.$$

(j) The lifting or hoisting force comes to w_h tons theoretically. For practical purposes and also for safety 20%. additional load is to be considered. Therefore, total hoisting load will be 1.2 W_h, tons. The hoist equipment shall be designed in consideration of this hoisting load.

11.22.3 Stresses in screw.

A power screw is subjected to an axial load and to a turning moment. Due to these the following type of stresses are set in the screw.

a. General.

(1) Direct tension or compression due to axial load. The method of mounting the screw will determine whether the stress is tensile or compressive. If possible the thrust collar should be provided so that the screw is in tension.

(2) Shear stress due to axial load. This stress is usually not dangerous. Due to the axial load, the threads of screw will shear at the core or root diameter and the thread of the nut will shear at the major diameter which is nothing but the outside diameter of the screw. Therefore :-

$$F_s\ (screw) = \frac{W_h}{n \pi d_c t} \qquad (1)$$

$$F_s(nut) = \frac{W_h}{n \pi d_o t} \qquad (2)$$

Where,

W_h = Axial load on the screw hoisting load, Kg
n = No. of thread engagement.
d_c = Core diameter of Screw, cm.
d_o = Outer diameter of screw, cm.
t = thickness of the thread or width of the thread, cm.

(3) Compression between the thread surfaces due to axial load. This would result in bearing pressure between the thread surfaces. This bearing pressure is not uniform since under load, the nut and the screw are under different kinds of stresses, i.e., if there is tension in the screw, there will be compression in the nut and vice versa. The assumption of uniform bearing pressure simplifies the design.

b. *Allowable bearing pressure P_{ba} of Screw material.*

Allowable bearing pressure P_{ba} depends on the material combination of screw and nut which is given below:

Screw Material	Nut Material	P_{ba} Kg/cm^2
Steel	Cast Iron	40-70
Steel	Bronze	55-100

11.23 Condition of self locking of screw and its efficiency.

(a) Condition of self locking:

If $\phi \geq \alpha$, the screw will not overhaul i.e., the load will not move downward without the application of a torque directed downward. Such a condition is known as self locking or self braking.

(b) Efficiency of screw (For square thread):

Work done in raising the load, when one complete turn is given to the screw is :

$$2\pi T = W_h . \tfrac{1}{2} . d_m . \tan(\alpha+\phi) . 2\pi \qquad (1)$$

So, work input is :

$$2\pi T = W_h . \pi . d_m . \tan(\alpha+\phi)$$

Now efficiency of a screw thread is given as:

$$\eta = \frac{Useful\ \ work}{Work\ \ input}$$

Now when the screw moves through one revolution, the load moves up by a distance equal to pitch for a single start thread,

$$\eta = \frac{w_h p}{w_h \pi d_m \tan(\alpha+\phi)} \qquad (2)$$

now, $\tan \alpha = P/(\pi\ d_m)$.

So, $P = \pi\ d_m.\ \tan\alpha$

From equation (2),

$$Efficiency = \frac{\tan\alpha}{\tan(\alpha+\phi)}$$

$$Efficiency\ in\ lowering\ the\ gate, = \frac{\tan\alpha}{\tan(\alpha-\Phi)}$$

$\frac{d\eta}{dx} = 0$ For maximum efficiency,

and it will be seen that $\alpha = 45°$

$$From\ eqn\ \ (2)\ \ \eta_{max} = \frac{1-\tan\Phi}{1+\tan\Phi}$$

If for ordinary values of μ, curve is plotted between the efficiency and the helix angle, then the efficiency increases rapidly up to the helix angle of 20° and then the increase is slow and maximum is reached for helix angle of 40 to 50°. There is not much difference in efficiency between range of helix angle to 30 and 50°. As the helix angle increases it becomes difficult to cut the threads and the mechanical advantage decreases. In practice, helix angle more than 30° are seldom used.

Also, from condition of self locking, we know that
$\alpha \le \phi$, so, Efficiency, $\le \frac{1}{2}(1 - \tan^2\phi)$

Hence it is clear that for self locking, the efficiency cannot be more than 50%. It is also clear that if the efficiency is more than 50%, the screw will not remain self locking and it will overhaul.

(c) Co-efficient of friction of screw materials

The co-efficient of friction of the materials for the threads of power screws depends upon : materials for the nut and screw, workmanship in cutting the threads, lubrication, and degree of 'running in' of the threads and the unit pressure on the threads. If the unit pressure is less than about 1000 kg/cm2 and the rubbing velocity is less than about

15 m.p.m, the values for co-efficient of friction (μ) may be taken from table given below. If the screw is operated very slowly, then the co-efficient may be taken as 30% higher.

Values for μ

Screw material	Nut material			
	Steel	Brass	Bronze	Cast iron
Steel, dry	0.15-0.25	0.15-0.23	0.15-0.19	0.15-0.25
Steel, Machine oil	0.11-0.17	0.10-0.16	0.103-0.15	0.11-0.17
Bronze	0.08-0.12	0.04-0.06	-	0.06-0.09

Most power screws are made of steel with nuts of C.I. or bronze. The co-efficient of friction for plain thrust collars are given below:

Values of μ_c

Material	Running (M_c)	Starting (M_c)
Soft steel on C.I.	0.121	0.170
Hardened steel on C.I.	0.092	0.147
Soft steel on bronze	0.084	0.101
Hardened steel on bronze	0.063	0.081

11.24 Selection criteria of Gears.

For selecting the gears, the maximum allowable gear ratio, efficiency and pitch line velocities are to be studied which are given in the Table 11.5.

Table 11.5
List Of Gears

SL NO.	TYPE OF GEAR	GEAR RATIO	EFFICIENCY AT RATE POWER	MAXM PITCH LINE VEL. RPM
1.	SPUR	1 TO 10	95 TO 98	600
2.	HELICAL & HERRINGBONE	1 TO 15	95 TO 98	1500
3.	HELICAL AND DOUBLE HELICAL HIGH SPEED	1 TO 15	95 TO 98	9000
4.	CROSSED HELICAL	1 TO 10	95 TO 98	1200
5.	STRAIGHT BEVEL	1 TO 6	95 TO 98	300
6.	SPIRAL BEVEL	1 TO 9	95 TO 98	2400
7.	ZEROL	1 TO 9	95 TO 98	1200
8.	HYPOID	1 TO 9	95 TO 98	1200
9.	HIGH REDUCTION HYPOID	10 TO 90	80	-
10.	WORM	3.5 TO 90	50 TO 90	1800
11.	DOUBLE ENVELOPING WORM	3.5 TO 90	50 TO 90	1200
12.	FACE	3 TO 8	95 TO 99	1200
13.	SPIROID	10 TO 100	50 TO 97	1800
14.	HELICON	3 TO 100	50 TO 98	1800
15.	BEVELOID	1 TO 100	50 TO 95	1200

11.25 Some important parameters of Gears.

The following parameters are frequently used in selecting and designing the various type of gears.

[a] *Recommended series of modules mm:*

The following modules are always preferred for cutting the gear teeth very smoothly. If modules other than these are used the cutting of gear teeth would be difficult.(as per IS and JIS)

1, 1.25, 1.5 , 2.0 , 2.5 , 3.0 , 4.0 , 5.0, 6.0, 8.0, 10.0, 12.0, 16.0, 20.0, 25.0, 32.0, 40.0, 50.0.

[b] *Systems of gear teeth:*

The following systems of gear teeth are used in practice:

1. $14\frac{1}{2}°$ Full depth system
2. $14\frac{1}{2}°$ Composite system
3. $20°$ Full depth system
4. $20°$ Stub system

Use $20°$ full depth system, which helps in reducing the minimum number of teeth on the pinion as compared to other systems.

[c] *Minimum no. of teeth on Pinion for different teeth systems:*

$14\frac{1}{2}°$ Full depth system, is 31
$20°$ Full depth system , is 17
$20°$ Stub system, is 14
$14\frac{1}{2}°$ Composite system is 12

For gear drive used at low speed and for manual work, i.e., winches, the minimum number of teeth for $20°$ involute system can be reduced to 10.

[d] *Stress concentration Factor, K*

The stress concentration factor, K is a function of the ratio of fillet radius to the thickness and the ratio of teeth thickness to teeth height.

For most design purposes, it's value may be taken as,

K = 1.6 for full depth $14\frac{1}{2}°$ teeth
= 1.5 for $20°$ Full depth and stub teeth.

[e] *Peripheral Speed of Spur Gear:*

DEGREE OF ACCURACY	PERIPHERAL SPEED OF GEARS, M/SEC
1.0 - 6.0	> 15
6.0 - 7.0	7.5 - 15
7.0 - 8.0	5.0 - 7.5
8.0 - 9.0	2.5 - 5.0
10.0 - 12.0	< 2.5

[f] *Backlash for gears:*

In actual practice, for normal functioning of the mating gears and to allow for thermal and centrifugal expansion, the tooth is made somewhat thinner, to provide easier engagement of the corresponding tooth space. Hence the tooth thickness is less than the width of space, the difference is known as "Backlash".

Sometimes to tip and the root of the tooth profile are relieved. This will ensure smooth sliding into and out of mesh without knocking and also will take care of any misalignment.

Backlash for gears, mm.

Peripheral Speed, <8 m/s			Peripheral speed >8 m/s	
Modulus, mm	Backlash, mm		Module, mm	Backlash
	Minimum	Maximum		
20	0.75	1.25	8.00	0.40
16	0.50	0.85	7.00	0.38
12	0.35	0.60	6.00	0.36
10	0.30	0.51	5.00	0.28
8	0.22	0.40	4.00	0.23
6	0.20	0.33	3.50	0.22
5	0.15	0.25	3.00	0.21
4	0.13	0.20	2.75	0.20
3	0.10	0.15	2.50	0.19
2.5	0.08	0.13	2.00	0.18
2	0.08	0.13		
1.5 and finer	0.00	0.10		

11.26 Gear Material

Sl no.	Material	Condition(Finished gear)	Minimum Tensile Strength Kg/mm^2	BHN no.
1.	Malleable C.I.	-	28 -32	149 - 217
2.	Cast Iron	As cast and Heat treated	20- 35	179 - 300
3.	Phosphor Bronze	Sand cast, Chill cast, centrifugal cast	16 - 27	60 - 90
4.	Cast steel	-	55	145
5.	Forge Steel			
	a. Carbon Steel	Normalised, Hardened & tempered	50 - 80	143 - 248
	b. Carbon Chromium Steel	- Do -	80 - 100	225 - 285
	c. Carbon Manganese Steels	- Do -	55 - 80	170 - 230
	d. Manganese Molybdenum Steel	- Do -	70- 90	200 - 230
	e. Chromium Molybdenum Steel	- Do -	70 - 155	200 - 300
	f. Nickel Steel	- Do -	80- 90	230 - 255
	g. Nickel Chromium Steel	- Do-	145	444
	h. Nickel chromium Molybdenum Steel	- Do -	90- 155	255- 444
6.	Surface Harden Steel			
	a. Carbon Steel	-	55 - 71	145 - 520
	b. Chromium Steel	-	70 - 80	250 - 500
	c. Nickel Steel	-	70	200 - 300
	d. Nickel Chromium Steel	-	86	250 - 500
7.	Case Hardened Steels			
	a. Carbon Steel	-	50	140 - 650
	b. Ni steel	-	70 - 80	200 - 620

11.27 Proportions of Bevel gears.

The proportions of bevel gears are given as:

Addendum at larger end, a	= m.
Circular pitch, P_c	= π. m
Dedendum, d	= 1.2 m (At larger end).
Thickness of tooth	= face width = $P_c/2$ = 1.57 m
Working depth	= 2 m
Clearance	= 0.2 m
Outside diameter	= Pitch diameter + 2 a cos γ

$$D_o = D + 2a \cos\gamma$$

Inside diameter or Dedendum diameter

$$D_d = D - 2d \cos \gamma.$$

Pressure angle $\phi = 20°$

11.28 Hand wheel operated two drum with spur gear and worm reduction system.

(a) *General.*

In this type of hoists, the gate shall be self closing due to its own weight. The downward forces shall be at least 20% higher than the frictional forces. If this condition is not satisfied the gate shall be suitably ballasted and the ballast loading shall be reckoned in the structural analyses of the gate leaf.

This type of hoists are generally provided or installed to lift the roller gates of higher size and lifting load. Also where electric power is not available this type of hoists are used to lift the navigation lock gate or any other vertical lift gate having hoisting load approximately 5 tons or higher. Counter weight fitting provisions are made with this system that shall be discussed later on.

(b) *Main components.*

(i) Wire rope.
(ii) Drum.
(iii) Spur gear and pinion.
(iv) Worm wheel and shafts.
(v) Hand wheel.
(vi) Shafts to hold drum and gear.
(vii) Base frame on which the hoist arrangements are placed.
(viii) Counter weight, if necessary.

11.29 Selection of wire rope.

a. *Factor of safety (F_s)*

Hoisting load of the gate = W_h
No, of wire rope used N = 2.
So, load on each wire rope = $\frac{1}{2}W_h$ = W_d.

Let diameter of wire rope = D_r
Breaking strength = W_B

These D_r and W_B are selected from the standard specifications of wire rope for materials of plow steel or galvanized steel for the type of 6 x 37, in such a way that,

Factor of safety = W_B/W_D = F_s > 8 for straight tension of wire rope.

b. Criteria for selecting factor of safety. (F_s)

The factor of safety of wire ropes mainly depends on the velocity of the wire rope and the working load per rope. The criteria of the same are given below:

Velocity of the rope upto 30 mpm, F_s=7 (But for straight tension, we consider it as 8)
Velocity of the rope upto 90 mpm, F_s=18
Velocity of the rope upto 240 mpm, F_s=36
Maximum permissible working load per rope
Velocity from 120 to 240 mpm = 14 d^2,
Velocity from 45 to 90 mpm = 28 d^2,
Velocity < 30 mpm = $70d^2$

Where, 'd' is the diameter of the rope in cm.

c. Breaking load of some commonly used wire rope

Type of wire rope : 6 x 37

Material: Galvanized steel

DIAMETER OF ROPE (D)	BREAKING LOAD IN TONS	UNIT WEIGHT IN KG/M
10	5.41	0.359
12.5	8.45	0.561
14	10.6	0.704
16	13.8	0.920
18	17.5	1.16
20	21.5	1.44
25	33.8	2.25
28	42.4	2.82
30	48.7	3.23
33.5	60.7	4.03
40	86.6	5.75
50	135	8.98

11.30 Selection criteria of Drums.

As per standard minimum diameter of the drum D_d = 13 x D_r

Proposed diameter D_p= 27xD_r.

We shall use diameter of the drum D_d = 20xD_r.

In addition to the drum being adequately sized for the required mechanical advantage, the following conditions shall be satisfied.

(a) The number of grooves on the drum shall be adequate to take up the full length of the cable when fully raised, in one layer.

(b) When fully lowered, there shall be at least two full turns of the cable yet remaining as a lead off.

(c) The drum shall have flanges on both sides.

(d) The lead angle of drum grooves shall be less than 50°.

(e) The clearance between adjacent turns shall be 1.5 mm for ropes up to 12 mm diameter, 2.5 mm for diameters above 12 mm; for the range of loads under consideration.

The drum shall be designed for bending and crushing

11.31 Design of spur gear.

[i] Design Considerations

For the design of a gear drive, the following data is usually given:

(1) Horse power to be transmitted or Torque to be transmitted.
(2) Speed of the driving gear.
(3) Speed of the driving gear, or the velocity ratio.
(4) Centre distance.

While attempting the design of a gear drive, the try must be made to meet the following requirement:

(1) The teeth must be sufficiently strong so as to withstand the static loading as well as dynamic loading on the drive.
(2) The teeth should have good wear resisting properties for their long life.
(3) Economy of space and material.
(4) The alignment of gears and deflections of shafts must be taken into considerations since these effect the gear performance.
(5) Proper lubrication of the gears.

[ii] Design Procedure

Basically the design procedure for finding the tooth proportions consists of the following procedure.

(1) If the peripheral speeds are not given, these should be selected. These speeds are also selected or calculated from the prefixed value of gate lifting as well as no. of reductions and stages of reductions.

Its value may be taken as 180 to 300 mpm for a low rotary speed of the pinion and a low to moderate power transmission. For higher values of pinion speed and higher power, greater values for peripheral speed may be assumed. But for hydraulic gate lifting the lower peripheral speeds of pinion and gears are required.

(2) If the material for pinion and gear are also not specified, these should also be selected. For a low velocity, C.I. and low grade steel may be used but for a higher velocity, better grade materials should be selected.

(3) Design load, F_d is found out.

(4) To solve the Lewis equation, the preliminary values of 'f' and y are assumed from empirical relation and tables given in the later sections. The Lewis equation is then solved for module m and the nearest standard value is selected. It should be ensured that $F_b \geq F_d$.

(5) The approximate pinion diameter is determined from the selected value of V_m and the number of teeth n_g and n_p are found out.

(6) Exact values for Y are calculated both for pinion and gear.

(7) The beam strength equation is solved both for pinion and gear and for each the value of 'f' determined. Greater value is chosen.

(8) The weaker is checked for wear to ensure that $F_w \geq F_d$. A weaker gear is one with smaller value of $(f_b \times y)$, known as strength factor of gear.
where, f_b = Allowable bending strength of the gear material kg/cm^2

y = o.154 - 0.192/n for 20° Pressure angle, Full depth system.
= 0.175 - 0.841/n for 20° Pressure angle, stub tooth system.
= 0.124 - 0.684/n for 14½° Pressure angle.

[iii] Design stress

The value of the bending or flexural stress f_b may be taken as equal to the elastic limit of the material in bending divided by the factor of safety which may be taken as 1.5.

The ratio of the ultimate tensile strength of the gear materials to the elastic limit may be taken as shown below:

(a) Cast Iron: Not treated = 2.5 to 3.
Heat treated = 1.7

(b) Phosphor bronze - 1.8 to 2.

(c) Forged steels

Carbon steels : Not treated = 1.8
Hardened = 1.6 to 1.7
Case hardened = 1.8
Alloy steels = 1.2 to 1.25

(d) Cast steels : About 2.2.

[iv] Gear proportion

The gear construction may have different designs depending upon size and application. If the dedendum circle diameter D_d is just greater than the shaft diameter by an amount less than half the shaft diameter, the pinion is made integral with the shaft. If the diameter of the pinion is :

$D \leq (14.75\ m + 6)$ cm, Where m = module in cm.

Then the pinion is made solid with uniform thickness equal to the face width 'f'. The limit for a pinion or gear with a web is given by the equation:

$D \leq (23.5\ m + 8.5)$ cm.

The web thickness should be from 1.6 to 1.9 m. The web may be solid or with recesses to reduce its weight.

Large gears have arms joining the rim to the hub. The no. of arms depending upon the size of the gear may be decided as below:

4 to 5 arms for diameter up to 50 cm
6 for diameter from 50 to 150 Cm.
8 arms for diameter from 150 to 200 cm.
10 arms for diameter > 200 cm

[v] Gear teeth proportions of spur gears

Addendum, a = m.
Dedendum, d = 1.25m.
Clearance = 0.25m
Total depth = a + d
Working depth = Total depth - clearance.
Circular pitch $P_c = \pi\ m$
Width of space = ½(Circular pitch + Backlash)
Tooth thickness = Width of space - Backlash

[vi] Factor of safety for gears

Type of drive and load	
(a) For steady load on a single pair of gears	3
(b) For suddenly applied load on a single pair	4
(c) Steady load on gears of train beyond the first mesh	5
(d) Sudden load on gears of train beyond first mesh	6

[vii] Safe working or allowable bending stresses (f_b) for gear materials

Material	f_b, kg/cm²
(1) Cast Iron	560
(2) Cast Iron, good grade	700
(3) Cast Iron, High grade	1050
(4) Cast steel, untreated	1400
(5) Cast Steel, heat treated	1960
(6) Forged steel, untreated	1400 to 2100
(7) Forged steel, heat treated	2100 to 2450

(8) Alloy steels, case hardened	3500
(9) Cr- Ni steel, heat treated	4700
(10) Cr - va -steels, heat treated	5250

[viii] Lewis form factor Y

Lewis form factor Y depends on the no. of teeth on the pinion and the system of gear teeth used. The values given below are for 20° pressure angle, full depth system.

No. of teeth	Y	No.of teeth	Y
12	0.245	30	0.359
13	0.261	34	0.371
14	0.277	38	0.384
15	0.290	43	0.397
16	0.296	50	0.409
17	0.303	60	0.422
18	0.309	75	0.435
19	0.314	100	0.447
20	0.322	150	0.460
21	0.328	300	0.472
22	0.331	400	0.480
24	0.337	Rack	0.485
26	0.346		
28	0.353		

[ix] Face Width of the Gear

If face width is very long in relation to pitch, the load may not remain uniformly distributed which will be against the assumption made. On the other hand, a narrow face width will give a less smooth action and poor wearing qualities. The usual value is taken as : f = 3 to 4 P_c or 9.5 m to 12.5 m

[x] Design Conditions

All the parameters of the gears shall be found out or calculated satisfying the following conditions.

a. Flexural Strength:

$F_b \geq F_d$, where F_d is the design strength.

b Wear Strength:

$F_w \geq F_d$

For safety against breakages, the beam strength of the teeth should be greater than the dynamic load, i.e

For safety load, $F_b \geq 1.25\ F_d$
For pulsating load, $F_b \geq 1.35\ F_d$
For shock load, $F_b \geq 1.50\ F_d$

To satisfy the above conditions, the material combinations of the gear and the pinion and its physical and chemical properties are adjusted.

11.32 Calculation of hoisting load for radial gates.

Hoisting load of a radial gate is calculated by the following relation.

$$W = (w_1 r_1 + w_2 \mu_1 r_2 + w_3 \mu_2 r_3 + w_4 \mu_3 r_4 + w_5 \mu_4 r_5) / R \quad \text{tons}$$

Where,

W = total hoisting load, tons.
R = radius from trunnion pin centre to wire rope centre, m.
W_1 = gates self weight tons.
r_1 = radius up to the centre of gravity of the gate. m.

(It is generally calculated by mass-moment relation knowing the self weight of the gate as well as other members).

W_2 = Maximum water pressure and earth pressure tons.
M_1 = co-efficient of friction of the pin bush =0.2
r_2 = radius of pin, m.

W_3 = Water load flowing on watertight rubber seal

$$= \frac{\text{Water and earth pressure X rubber width X 2 (2 for two sides)}}{\text{span length}}$$

M_2 = co-efficient of friction of watertight rubber
= 0.7 (At running)
= 1.5 (At starting)

r_3 = radius up to watertight rubber, m.

W_4 = Horizontal reaction force, tons.
= $[(H_o + H_n) \times 2]$

M_3 = Co-efficient of friction of thrust washer of pin = 0.2

r_4 = radius of thrust washer, m.

W_5 = earth pressure, tons.

r_5 = radius up to the earth pressure line of action, m.

The design hoisting load shall be taken 10 to 20% higher than the actual, i.e.,

W_a = 1.2 x W tons. (Lifting force).

11.33 Arrangement of hoisting system for radial gate.

For central drive hoist system, the motor, reducer and drum are generally arranged as shown in fig. 11.8.

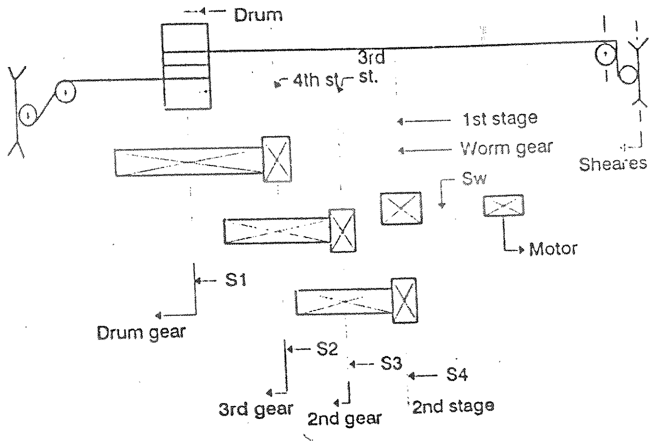


FIG. 11.8

In the figure:

S_1	=	drum shaft.
S_2	=	3rd spur gear shaft
S_3	=	2nd spur gear shaft.
S_4	=	worm gear shaft.
S_w	=	worm and motor shaft.

11.34 **Mechanical efficiency of the hoisting system for radial gate.**

Standard mechanical efficiency of each component of the system (as shown in fig.11.8):

Drum	=	0.95
Drum gear	=	0.95
Spur gear	=	0.95 x 2 (For two set)
Sheave	=	0.95 x 3 (For three nos.)

Worm = 0.50 (assumed for self locking system. Also may be calculated from the physical and chemical properties of the gear set)

Hence, total mechanical efficiency of the system is

$$\eta_m = 0.95 \times 0.95 \times 0.95 \times 0.95 \times 0.50$$
$$= 0.349$$
$$= 0.35$$

11.35 Motor capacity

Capacity of the motor is calculated by the formula:

$$P = \frac{W_h \cdot V}{6.12\,\eta_m} \quad KW$$

Where,

W_h = hoisting load, tons.
V = hoisting speed of wire rope: unit m/min.
η_m = mechanical efficiency of the system.
= 0.35 (In this particular case).

The calculated value may be fractional but next higher available standard capacity motor is selected. Which should be called rated capacity. The initial torque shall be 250% more than the rated torque of the motor.

11.36 Design of Worm Gear (For motor operated hoisting system).

(I) General

Worm and worm gear drive can be employed for large speed ratios ranging from 10 : 1 to 100: 1 and ratios as high as 500: 1. These drives are used in most cases to transfer motion from input shaft to the follower shaft at an angle of 90. Worm and worm gear drive is mostly used as speed reducer with worm as the driver and worm gear or wheel as the driven member.

(II) No. of Worm Starts or threads

The no. of worm starts usually range from 1 to 4. It may be upto 6 or 8 with various speed ratios and the gear teeth should be greater than 29.

VELOCITY RATIO	NO. OF STARDS OR THREADS
36 AND OVER	ONE
12 TO 36	DOUBLE
8 TO 12	TRIPLE
6 TO 12	QUADRUPLE
4 TO 10	SEXTUPLE

(III) Lead angle of Worm

Lead angle Λ may vary from 9° to 45°.

It is usually taken as 6° per worm teeth. Experience indicates that angle less than 9° results in rapid wear. A safe value for Λ is given as $\Lambda \geq 12\frac{1}{2}°$.

For a compact design, angle Λ may be selected approximately from the relation

$$\tan \Lambda = (N_g/N_w)^{1/3}$$

where,

N_g = No. of teeth in worm wheel
N_w = No. of struds in worm.

If $\Lambda < 5°$, then the system shall be self locking; but the efficiency would be decreased.

(VI) Face width of the Worm Gear (f)

It's value may be taken from experiment as :

$$f = 2.38\ P_c + 0.65 \text{ cm, with worm start 1 to 2}$$
$$f = 2.15\ P_c + 0.50 \text{ cm, with worm start 3 to 4}$$

Also the face width of the worm gear should be equal to the length of the tangent drawn to the worm pitch circle between it's points of intersection with t he addendum circle.

11.37 Checking of wear strength.

The limiting load F_w, for wear may be determined from the relation,

$$F_w = D.f.K \quad \text{Kg}$$

where,

D = Pitch dia of the gear, cm
f = Face width, cm
K = Load stress factor, Kg/cm²

The factor K is taken from the table for different material combination of worm and worm gear, which is given below:

SL NO.	WORM MATERIAL	WORM GEAR MATERIAL	K, KG/CM²
1.	250- BHN STEEL	PHOSPHOR BRONZE	4.2
2.	HARDENED STEEL	PHOSPHOR BRONZE	5.6
3.	HARDENED STEEL	CHILLED PHOSPHOR BRONZE	8.4
4.	HARDENED STEEL	ANTIMONIC BRONZE	8.4
5.	HARDENED STEEL	CAST IRON	3.5
6.	CAST IRON	PHOSPHOR BRONZE	10.5

The values given in the above table are suitable for lead angle upto 10°. For angle between 10°and 25°, increase K by 25%. For lead angle greater than 25°, increase K by 50%.

11.38 Final specifications of worm wheel set.

(i) Worm

PCD = D_w, Module = m,
No. of teeth = N_w = No. of start.
Lead angle = Λ
Pressure angle ϕ = 20° (generally taken)

Addendum = a = 0.3683 P_a

where,

P_a	=	Axial Pitch = $\pi\ D_g/N_g = \pi .\ m$
Dedendum(d)	=	1.2 a
Clearance	=	0.2 a
Lead L	=	$P_a .\ Nw = \pi\ m\ N_w$
D_{wo}	=	$D_w + 2a$

(ii) Worm wheel

PCD = D_g = N_g m;
Helix angle = ϵ
Pressure angle = ϕ = 20°
Addendum = a
Dedendum = d

Having full and final specifications, the worm gear box is designed adequately to maintain an allowable temperature difference of 40°F approximately so that no serious abrasion and scoring occur to the teeth faces of the gear set. Also the gear box should be filled with a specified oil which can absorb and carry away the heat generated during continuous operation of the gate.

11.39 Design of shafts.

Shafts shall be designed for bending, shear and torsion. In the case of bushing, The bearing stresses shall also influence the choice of the diameter. The angular twist in load transmitting shafts shall be less than 0.3° per meter. Linear deflections shall be less than 1mm per meter span.
Shafts shall be designed by applying the combined torsion and bending moment formula. The diameter of the shaft is calculated by

$$D^3 = \frac{16}{\pi f_s}\sqrt{(K_b . M_{max})^2 + (K_t . T)^2}$$

Where,

F_s	=	Design stress of the shaft material, Kg/cm²
K_b	=	Bending factor for gradually or steady load =1.5
K_t	=	Torsion factor for gradually or steady load=1.00
M_{max}	=	Maximum moment Kg-cm. (It is calculated from shear force and bending moment diagram of the shaft).
T	=	transmitted torque on the shaft.

Design stress of shaft:

Design stress, F_s = 0.3 x elastic limit in tension.
= 0.18 x ultimate tensile strength.
(whichever is smaller).

and design stress, f_t = 0.6 x elastic limit in tension
= 0.36 x ultimate tensile strength.

For commercial shafting,

Design stress, $f_s = 560\ Kg/cm^2$
and Design stress, $f_t = 1120\ Kg/cm^2$

The above values are for shafts without keyways.

For shafts with Keyways:-

Design stress = 0.75 x Design stress without keyway.

Reference.

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Annexure A-1

ABBREVIATIONS

Abbreviations used :

kg	:	kilogram
km	:	kilometer
kN	:	kilo Newton
m	:	meter
mm	:	millimeter
N	:	Newton
cum	:	cubic meter
KHW	:	kilowatt hour
ltr	:	liter
Nos	:	numbers
Km^2	:	square kilometer
m^2	:	square meter
u/s	:	upstream
d/s	:	downstream
cm^2	:	square centimeter
psi	:	pound per square inch
N/mm^2	:	Newton per square millimeter
Kn/m^2	:	kilonewton per square meter
pcf	:	pound per cubic foot
mm/day	:	millimeter per day
psf	:	pound per square foot
t/ft^2	:	ton per square foot
Kn/m^3	:	kilonewton per cubic meter
Kn/m	:	Kilomewton per meter
mm^2	:	square millimeter
m^3/sec	:	cubic meter per second
N/m^2	:	Newton per square meter
N/m	:	Newton per meter
l/s	:	litre per second
m/s	:	meter per second
Kg/cm^2	:	Kilogram per square centimeter
Kg/m^3	:	Kilogram per cubic meter
t/m^2	:	ton per square meter
t/m	:	ton per meter
t/cm^2	:	ton per square centimeter
t-cm	:	ton-centimeter
cm3	:	cubic centimeter
lb/ft^2	:	pound per square foot
Hz	:	Hertz
Kg/mm^2	:	kilogram per square millimeter
KW	:	Kilowatt
Cwt	:	hundred weight
ft	:	foot
H.P.	:	horse power
in	:	inch

lbs	:	pounds
yd	:	yard
P/km	:	per kilometer
sqm	:	square meter
N-m	:	Newton-meter
km^2	:	square kilometer
m^2	:	square meter
hectares/cumec	:	hectares per cubic meter per second
cumec/million sqm	:	cubic meter per second per million square meter
$m^3/sec/m$	:	cubic meter per second per meter

Annexure A-2

Unit Weights of Basic Materials

Materials	Unit Weight (KN/m^3)
Aluminium	27.0
Asphalt	21.2
Brass	83.6
Bronze	87.7
Brick	18.9
Cement	14.7
Coal, loose	8.8
Concrete, Structural	23.6
Concrete, Plain	22.6
Copper	86.4
Cork, normal	1.7
Cork, compressed	3.7
Glass, window(soda lime)	25.5
Granite, Basalt	26.4
Iron	
- cast	70.7
- wrought	75.4
Lead	111.0
Lime stone	24.5
Marble	26.4
Sand, dry	15.7
Sandstone	22.6
Slate	28.3
Soil, dry	15.7
Soil, moist	17.3
Soil, saturated	18.9
Steel	77.0
Timber	5.9-11.0
Water, Sea	10.1
water, Fresh	9.8
Zinc	70.0

Annexure A-3

CONVERSION TABLE

Basic Conversion Factors :

The following equivalents of SI units are given in imperial and where applicable, Metric Technical Units.

1 mm = 0.03937 in 1 m = 3.281 ft = 1.094 yd 1 km = 0.6214 mile	1 in = 25.4 mm 1 ft = 0.3048 m 1 yd = 0.9144 m 1 mile = 1.609 km
1 mm^2 = 0.00155 in^2 1 m^2 = 10.76 ft^2 1 m^2 = 1.196 yd^2 1 hectare = 2.471 acres = 1000 m^2	1 in^2 = 645.2 mm^2 1 ft^2 = 0.0929 m^2 1 yd^2 = 0.8351 m^2 1 acre = 0.4047 hectares
1 mm^3 = 0.00006102 in^3 1 m^3 = 35.31 ft^3 = 1.308 yd^3	1 in^3 = 16390 mm^3 1 ft^3 = 0.02332 m^3 1 yd^3 = 0.7646 m^3

Force :

1 N = 0.2248 lb = 0.102 kg 1 kg = 2.205 lb 1 KN = 0.1004 Ton = 102.0 kg = 0.102 Tonne	1 lb = 0.4536 kg 1 Ton = 9.964 KN = 1016 kg = 1.016 Tonne
1 Tonne = 1000 kg = 0.9842 Ton = 9.807 KN	1 Cwt = 112 lbs = 50.80 kg = 0.508 Quintal
1 Quintal = 1.9684 Cwt	

Force Per Unit Length :

1 N/m = 0.06852 lb/ft = 0.1020 kg/m 1 lb/ft = 14.59 N/m = 1.488 kg/m	1 KN/m = 0.0306 Ton/ft = 0.1020 Tonne/m 1 Ton/ft = 32.69 KN/m = 3.333 Tonne/m

Force Per Unit Area :

1 N/mm^2 = 145.0 lb/in^2 = 10.20 kg/cm^2 1 lb/in^2 = 0.006895 N/mm^2 = 0.0703 kg/cm^2 1 N/m^2 = 0.02089 lb/ft^2 = 0.102 kg/m^2	1 Ton/in^2 = 15.44 N/mm^2 = 157.5 kg/cm^2 1 N/mm^2 = 9.324 Ton/ft^2

Force Per Unit Volume :

1 N/m^3 = 0.006366 lb/ft^3 1 lb/ft^3 = 157.1 N/m^3 = 16.02 kg/m^3 1 KN/m^3 = 0.003684 lb/in^3 = 0.102 $Tonne/m^3$	1 lb/in^3 = 271.4 KN/m^3 = 27.68 $Tonne/m^3$ 1 KN/m^3 = 6.366 lb/ft^3 1 Ton/ft^3 = 351.9 KN/m^3 = 35.88 $Tonne/m^3$

Fluid Capacity :

1 litre = 0.22 Imperial gallons = 0.2642 US gallons
1 Imperial gallon = 4.546 litres = 1.201 US gallons

Power :

1 H.P. = 0.7457 Kilowatts	1 Kilowatt = 1.341 H.P. 1 Kilowatt-hr = 1.341 H.P.-hr