STANDARDS/MANUALS/ GUIDELINES FOR SMALL HYDRO DEVELOPMENT

2.2 and 2.3
Civil Works–
Hydraulic and Structural Design

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Lead Organization:

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AHEC-IITR, "2.2 and 2.3 Hydraulic and Structural Design", standard/manual/guideline with support from Ministry of New and Renewable Energy, Roorkee, July 2013.

PREAMBLE

There are series of standards, guidelines and manuals available on electrical, electromechanical aspect of moving machines and hydro power related issues from Bureau of Indian Standards (BIS), Rural Electrification Corporation Ltd (REC), Central Electricity Authority (CEA), Central Board of Irrigation & Power (CBIP), International Electromechanical Commission (IEC), International Electrical and Electronics Engineers (IEEE), American Society of Mechanical Engineers (ASME) and others. But most of these are developed keeping in view the large water resources/ hydropower projects. Use of the standards/guidelines/manuals is voluntary at the moment. Small scale hydropower projects are to be developed in a cost effective manner with quality and reliability. Therefore a need to develop and make available the standards and guidelines specifically developed for small scale projects was felt.

Alternate Hydro Energy Centre, Indian Institute of Technology, Roorkee initiated the exercise of developing standards/guidelines/manuals specifically for small scale hydropower projects under the sponsorship of Ministry of New and Renewable Energy, Government of India, in 2006. The available relevant standards / guidelines / manuals were revisited to suitably adopt them for small scale hydro projects. These have been prepared by experts in their respective fields. Wide consultations were held with all stake holders covering government agencies, government and private developers, equipment manufacturers, consultants, financial institutions, regulators and others through web, post and meetings. After taking into consideration the comments received and discussions held with the lead experts the standards/guidelines/manuals are now prepared and presented in this publication.

The experts have drawn some text and figures from existing standards, manuals, publications and reports. Attempts have been made to give suitable reference and credit. However, the possibility of some omission due to oversight cannot be ruled out. These can be incorporated in our subsequent editions.

These standards / manuals / guidelines are the first edition. We request users of these to send their views / comments on the contents and utilization to enable us to review these after about one year of its publication.

Standards/ Manuals/Guidelines series for Small Hydropower Development

General	
1.1	Small hydropower definitions and glossary of terms, list and scope of different Indian and international standards/guidelines/manuals
1.2 Part I	Planning of the projects on existing dams, Barrages, Weirs
1.2 Part II	Planning of the Projects on Canal falls and Lock Structures.
1.2 Part III	Planning of the Run-of-River Projects
1.3	Project hydrology and installed capacity
1.4	Reports preparation: reconnaissance, pre-feasibility, feasibility, detailed project report, as built report
1.5	Project cost estimation
1.6	Economic & Financial Analysis and Tariff Determination
1.7	Model Contract for Execution and Supplies of Civil and E&M Works
1.8	Project Management of Small Hydroelectric Projects
1.9	Environment Impact Assessment
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1.11	Renovation, modernization and uprating
1.12	Site Investigations
Civil works	
2.1	Layouts of SHP projects
2.2	Hydraulic design
2.3	Structural design
2.4	Maintenance of civil works (including hydro-mechanical)
2.5	Technical specifications for Hydro Mechanical Works
Electro Mechanical works	
3.1	Selection of Turbine and Governing System
3.2	Selection of Generator
3.3	Selection of Switchyard
3.4	Monitoring, control, protection and automation
3.5	Design of Auxiliary Systems and Selection of Equipments
3.6	Technical Specifications for Procurement of Generating Equipment
3.7	Technical Specifications for Procurement of Auxiliaries
3.8	Technical Specifications for Procurement and Installation of Switchyard Equipment
3.9	Technical Specifications for monitoring, control and protection
3.10	Power Evacuation and Inter connection with Grid
3.11	Operation and maintenance of power plant
3.12	Erection Testing and Commissioning

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1.0 INTRODUCTION

SHP can be broadly categorized in two types in Indian conditions as

(i) Independent hydro projects in hilly regions on small streams. These are mostly of medium / high head utilizing small discharges and
(ii) Installations in plains which utilize water regulated for other purposes e.g. irrigation canals, small dams, etc. These stations are usually of low head utilizing larger discharges.

Civil structures required for SHP use construction material mostly RCC, PCC, Stone / Brick masonry or Steel. The relevant latest version of standards issued by Bureau of Indian standards are required to be used in each case. In all cases innovation is required on the part of the designer to make use, as far as possible, of the locally available materials and labour. In this context, the designer should invariably visit the project site, preferably with a geologist, to make himself familiar with the actual site conditions and have a feel of the project area.

2.0 REFERENCES

R1.	IS: 10137 (1982)	:	Guidelines for selection of spillways and energy dissipaters
R2.	IS: 11527 (1985)	:	Criteria for structural design of energy dissipaters for spillways
R3.	IS: 4997 (1968)	:	Criteria for design of hydraulic jump type stilling basins with horizontal and sloping apron
R4.	IS: 7365 (1985)	:	Criteria for hydraulic design of bucket type energy dissipaters.
R5.	IS:10635 (1993)	:	Freeboard Requirement in Embankment Dams – Guidelines
R6.	IS: 1498 (1970)	:	Classification and Identification of Soils for General Engineering Purposes
R7.	IS: 8414 (1977)	:	Guidelines for Design of Under-seepage Control Measures for Earth and Rockfill Dams
R8.	IS: 5050 (1992)	:	Code of practice for design, construction and maintenance of relief wells
R9.	IS: 7894 (1975)	:	Code of practice for stability analysis of earth dams
R10.	IS:823 (1964)	:	Code of practice for protection of slope for reservoir embankment
R11.	IS: 9761-1995	:	“Hydropower Intakes – Criteria for Hydraulic Design
R12.	IS:7112 (1973)	:	Criteria for design of cross section for unlined canals in Alluvial soils
R13.	IS:10430 (2000)	:	Criteria for design of lined canals and guidance for selection of type of lining
R14.	IS:10646 (1991)	:	Canal linings – cement concrete tiles – specifications
R15.	IS:11809 (1994)	:	Lining of canals by stone masonary – code of practice
R16.	IS:4558 (1995)	:	Under drainage of lined canals code of practice
R17.	IS:4880 (Part IV)	:	Design and Construction of Tunnel Plugs – Code of Practice”

R18. IS:11105 (1984) : "Code of Practice for Concrete Structures for the Storage of Liquids"
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R25. IS: 15119 (Part II -2002) : Measurement of Liquid Flow in Open Channels – Part 2: Determination of the Stage – Discharge Ralation (Superseding IS 2914:1964)
R26. IS: 10137(1982) : Guidelines for selection of spillways and energy dissipators,
R27. IS: 6934 (1998) : Hydraulic design of high Ogee overflow spillway
R28. IS: 11485 (1985) : Criteria for Hydraulic design of sluices in concrete and masonary dams
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3.0 CIVIL WORKS

3.1 The main civil works of a SHP are as follows:

(i) Diversion structure
(ii) Intake works

(iii) Water conductor system
(iv) Desilting tank
(v) Forebay / surge tank
(vi) Penstock
(vii) Power house building
(viii) Tailrace channel

4.0 DATA REQUIRED FOR DESIGN

4.1 Topographical Data

The following topographical data is required for the design of the various civil works:

(i) Contour plan of the project area from about 200 metres upstream of the proposed intake works or 200 metres upstream of submergence, whichever is more upto about 200 metres down stream of tailrace junction with the stream. In the lateral direction the contour plan should extend by about 10 metres above the proposed full supply level except in case of surge tank where it should be extended by about 10 metres above the upsurge level in an area of about 100 metres around the proposed surge tank location. The scale of this contour plan shall not be lesser than 1 in 1000 with atleast 2 metre contour interval.

(ii) Contour plan of the area around intake and diversion works from 200 metres upstream of intake to 300 metres downstream of the proposed dam / weir location and upto atleast 10 metres above the proposed full supply level. The scale of this contour plan shall not be lesser than 1 in 200 with a maximum of 1 metre contour interval.

(iii) Contour plan of the area around desilting tank from the river upto 5 metres above F.S.L. from 100 metres upstream to 100 metres downstream of the proposed location and in a scale not lesser than 1 in 200 with 1 metre contour interval.

(iv) Contour plan at the locations of cross-drainage works in an area of 200 m x 200 m with the proposed location of cross drainage work at the centre. The scale of these contour plans shall be at least 1:200 with 1 metre contour interval.

(v) Contour plan of the power house complex covering proposed locations of the surge tank / forebay, penstock, power house, switchyard, tailrace. The area covered should be atleast 100 metres beyond the proposed locations of the above works on either side. The scale of this contour plan shall be atleast 1:200 with 1 metre contour interval.

(vi) Longitudinal section of river water line on a particular day from 200 metres upstream of the proposed diversion / intake location or 200 metres upstream of submergence whichever is more upto 200 metres downstream of the propsoed junction of tailrace and the river. The proposed locations of various works and those of the junctions of rivulets crossing the water conductor system should be marked on the longitudinal section.

(vii) Cross-sections of the river at the locations of diversion weir and junction of river and tailrace and 50 metres upstream and downstream of the same. These sections shall also be marked on contour plans mentioned in above paras 3.1(i), 3.1(ii) and 3.1(vii) and 3.1 (viii).

(viii) Longitudinal sections of the rivulets from about 200 metres upstream to about 200 downstream of the proposed locations of the cross-drawing works.

(ix) Cross-section of the rivulets at the locations of cross-drainage works.

4.2 Geological and Geotechnical Data

The following geological data is required for the civil design of the various works.

Geological descriptions of the various locations of the civil works. These should be supported by the following:

(i) Geological logs of one to three drill holes at the location of proposed weir specially where the weir is to be designed on impermeable foundation.
(ii) Geological log of atleast one drill hole at the proposed location of intake.
(iii) Geological plan of the proposed weir & intake location with the drill holes locations marked on it.
(iv) Geological plan of the desilting tank location. In case of underground desilting tank, atleast one hole should be drilled at that site & geological log of the same supplied.
(v) Geological mapping along the headrace tunnel alignment on the basis of surface geology.
(vi) Geological log of atleast one hole drilled at the proposed location of surge tank.
(vii) Geological mapping along the proposed alignment of penstock. It should be supported by geological logs of holes drilled at a spacing of not more than 100 metres along penstock alignment.
(viii) Geological details of foundation strata below power house. Geological log of atleast on hole drilled at power house location in case the power house is proposed to be founded on firm rock.
(ix) Maximum level of the ground water table at the proposed power house & tailrace sites.
(x) C & ϕ values of the foundation material at power house & tailrace locations.
(xi) Soil grain size distribution curves at weir, intake and power house locations when these works are proposed to be founded on permeable foundations.
(xii) Maximum electrical resistivity of foundation strata.

4.3 Hydrological Data

The following hydrological data is required for the design of civil works

(i) Design flood discharges at locations of diversion structure and confluence of tailrace & river.
(ii) Design discharge of power house.
(iii) Gauge and discharge data at the locations of diversion structure and confluence of tailrace and river.

4.4 Other Data

(i) Number and capacity of machines.
(ii) The sediment particle size and concentration that is allowed to be passed though the turbine.
(iii) d50- 50% of finer size of boundary material of the river.

(iv) The following data is required for power house design from the supplier of electro-mechanical equipment:

(a) Layout of the power house appurtenant works such as cable tunnel, switchyard, by-pass water conductor system including tail race and access road.
(b) General plan of the power house showing the erection bay, control room, offices, workshop, utilities, etc, and other allied structures.
(c) Plan at generator floor level showing location of staircases, equipment and openings, etc.
(d) Plan at turbine floor level showing location of staircases, equipment and openings etc.
(e) Plan at centre line of spiral casing showing location of butterfly / spherical valve, expansion joint, etc.
(f) Plans showing features such as (i) galleries, (ii) draft tube deck, (iii) Transformers and fire walls, (iv) Transformer and gantry crane rails, (vi) Draft tube gates and gantry crane, (vii) Drainage and dewatering sump, (vii) Ducts, trenches and openings etc.
(g) Plan of the erection bay, control rooms and cable room.
(h) Transverse section (along the flow of water) through the centre line of machines, through the control and administration block and erection bay.
(i) Longitudinal section (at right angles to the direction of flow) through the centre line of machines.
(j) The shape and profile of the draft tube and its gated control along with hoisting arrangement and its liner.
(k) Shape and profile of spiral casing.
(l) Plan and elevations of generator foundations showing details of base plates anchor bolts, blockouts, etc.
(m) Drainage and dewatering arrangement comprising : (1) penstock, (2) draft tube
(n) Dimension, location and arrangement of inlet valve (if provided) supports.
(o) Dimensions and clearances of EOT cranes and drawings giving details of crane.
(p) Loads – The loads imposed by the equipment and accessories should be shown in plan and section such that the point of application of the load as well as its base area are clearly indicated.
    (1) Vertical load on stator sole plates
    (2) Tangential force in direction of rotation on stator sole plates
    (3) Tangential force in direction of rotation on stator sole plates
    (4) Vertical force on lower bracket sole plates.
    (5) Tangential force in direction of rotation on lower bracket sole plates
    (6) Force due to unbalanced magnetic pull (radial force)
    (7) Seismic force on stator sole plate;
    (8) Seismic force on lower bracket sole plate
    (9) Impact factor; and
    (10) Directions and points of application of above loads
(q) Load of the inlet valves including its auxiliaries
(r) Details of crane giving information on
    (1) The number and spacing of wheels of crane
    (2) Maximum wheel loads & impact factor
(s) Thrust due to servomotors

(t) Reaction of pelton turbine showing jet
(u) Vertical and horizontal loads of each switchyard equipment along with its supporting structures / towers, pulls of conductors and directions and points of application
(v) Horizontal thrust due to water to penstock indicating mode of transfer
(w) Water-hammer load due to sudden closure of the machine

5.0 STAGE – DISCHARGE CURVES

5.1 Stage – discharge curves are required to be developed at the location of diversion works as well as at the confluence of the tailrace and the river to carry out the hydraulic design of the diversion works, to fix top levels of the various works above high flood level and to carry out setting of the centre line of the turbines. Stage – discharge curves can be developed in accordance with the method described in IS: 15119: Part II. In case of SHP, the data required to develop a correct stage – discharge is generally not available. In such a case, a reasonable curve can be obtained by using Manning's equation at the cross-section of the river, where stage-discharge curve is required. The Manning's equation is as follows

$$Q = \frac{1}{n} A R^{2/3} S^{1/2},$$

Where,

Q = discharge in cumecs,
A = area of stream in sq.m,
R = Hydraulic mean depth in metres as ratio of area (A) with the wetted perimeter (P),
S = Longitudinal slope of the stream, and
n = Manning Rugosity coefficient of the channel bed & sides.

For determination of cross section of the stream, cross sections at the location where stage discharge curve is required to be evaluated and those at 50 metres upstream and 50 metres downstream may be plotted overlapping each other and an average section of the three sections may be considered the cross section of the river as indicated in Fig. 1 below.

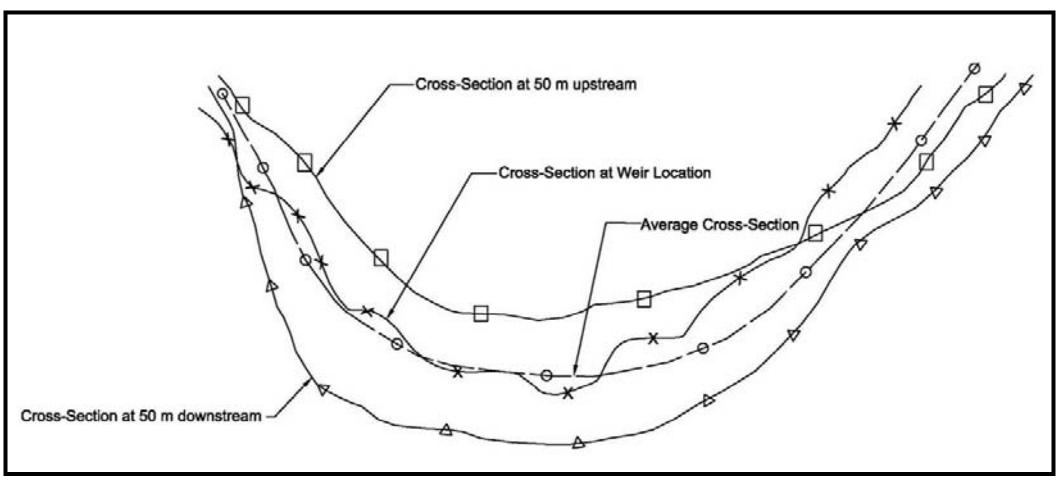


Fig. 1: Determination of Average River Section

With the use of average cross section of the stream, areas of flow 'A' and wetted perimeters and the hydraulic mean depth 'R' can be determined corresponding different water levels. Longitudinal slope 'S' can be taken as the average water slope of the stream from 200 metres upstream to about 200 metres downstream of the stream. The value of rugosity coefficient of the stream can be estimated from one or more of the empirical equations given in Table 1.

Table 1: Empirical Equations for Estimation Manning's Rugosity Coefficient for Boulder Streams

S. No.	Investigator(s)	Equation	Remarks
1.	Golubtsov (1969)	$n=0.222\,S_0^{0.33}$	data from central Asian rivers; S_0 = 0.4 – 20%
2.	Anderson et.al. (1970)	$n=0.0395(d_{50})^{1/6}$	data from flumes and natural rivers; $S_0<0.01$, d_{50} in ft
3.	Bray (1979)	$n=0.104\,S_0^{0.177}$	data from 67 gravel-bed rivers in Alberta: S_0 = 0.02 – 1%
4.	Jarret (1984)	$n=0.39R^{-0.16}\,S_0^{0.38}$	for high gradient boulder streams
5.	Bathrust (1985)	$n=5.62\log(y/d84)+4$	y = depth of flow; d_{84} = size of boulders; British mountainous rivers; S_0 = 0.004 – 0.04
6.	Abt et.al. (1988)	$n=0.456(d_{50}\,S_0)^{0.159}$	d_{50} in inch; and d_{50}=26-157 mm; S_0 = 0.01 – 0.20
7.	Rice et.al. (1998)	$n=0.029(d_{50}S_0)^{0.147}$	d_{50} in mm; S_0 = 0.1 – 0.4
8.	Strickler	$n=(d_{50})^{1/6}\,/\,24$	d_{50} is in metres

d50: 50% of finer size of boundary material river

6.0 DIVERSION AVERAGE STRUCTURE

There are a number of types of diversion structures which are normally constructed. The type of structure depends upon many factors which can be as follows:

(i) Importance and size of SHP
(ii) Availability of funds
(iii) Type of foundation strata
(iv) Longitudinal slope of the river
(v) Availability of head

The design criteria for diversion structures normally employed in SHP projects are described below:

6.1 Partial Weir

This is a temporary structure made of mud and stones, which is constructed just downstream of the intake to divert water into the intake mouth. This type of structure does not extend all the way across the stream. The structure helps stream flow to head upto an extent near the intake so that a portion of the stream discharge gets passed into the power channel. The total height of such structure is limited to about one metre or so. This being a temporary structure works only during lean flows and gets washed away during the floods, when the intake can draw water due to the high stage of the stream only. A typical arrangement of partial dam is shown in Fig. 2 from which its design is self explanatory. The mud used for the dam structure should be well puddled clay mixed with about 10% by volume of rough wheat husk. The stones used should be of 150 to 300 mm size and the space between the stones should be thoroughly filled with mud. Before laying the stones the foundation area should be cleaned of all loose material upto a depth of atleast 0.3 metre or so. As indicated in Fig. 2, a portion of the nose of the partial dam is required to be protected with bamboo / balli crates. The bamboos / ballies used to fabricate the crate around the nose should be atleast 50 mm in diameter and should be driven inside the river bed upto atleast 1.5 times the scour depth corresponding maximum non-monsoon discharge. The spacing of the bamboos – ballies should be around 300 mm in both horizontal and vertical directions.

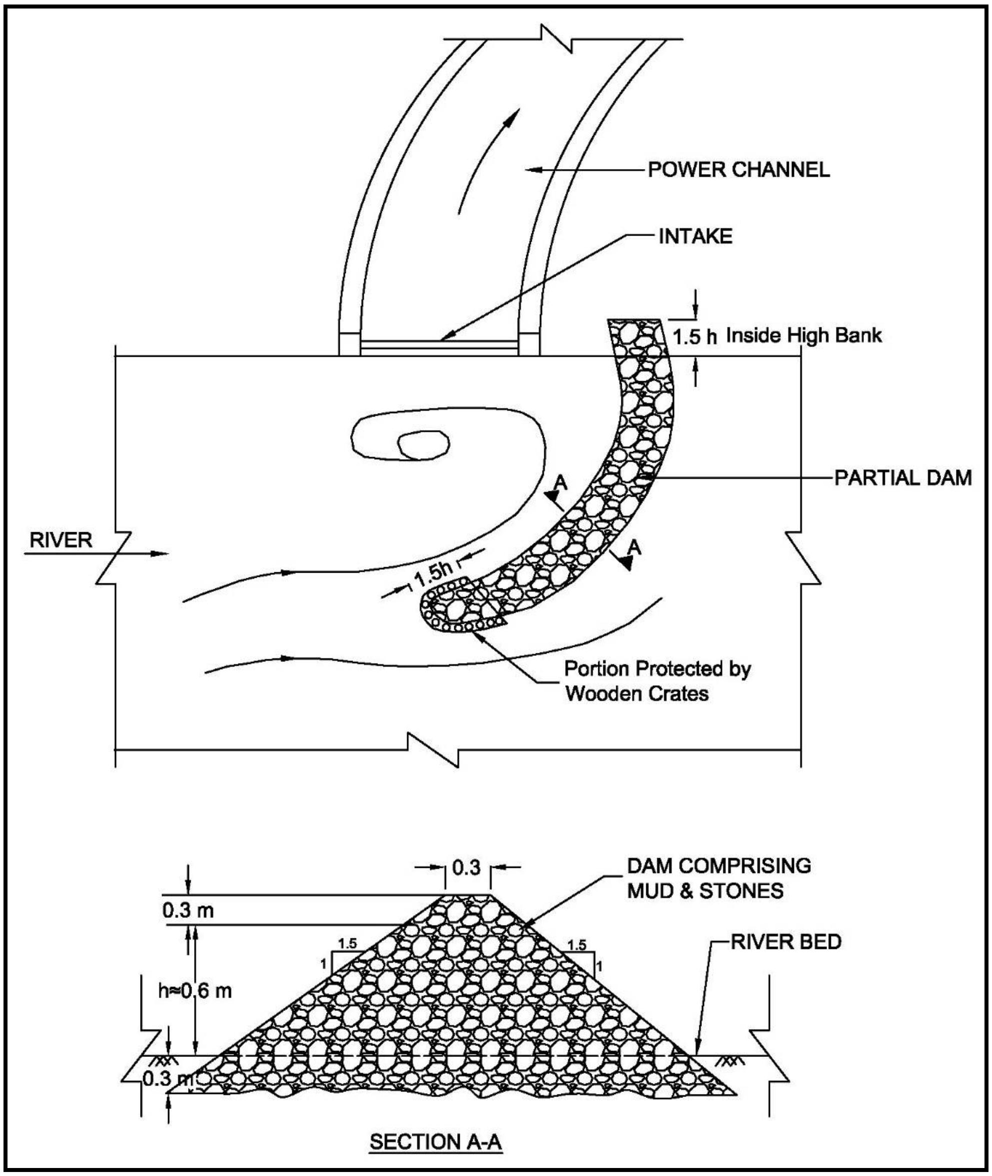


Fig. 2: General Arrangement of Partial Weir

6.2 Gabion Weir

This type of weir comprises solid boulder structure formed by encasing boulders in G.I. wire crates. This type is suitable for installations on small streams, where reduction in cost of diversion structure is desired. This type of structure is exposed to damage by heavy rolling boulders during monsoon months and hence requires maintenance specially after floods. This option should only be adopted for the following conditions:

(i) When the drop height does not exceed 4 metres
(ii) When the specific flow i.e. maximum discharge per unit width does not exceed 3 cumec/metre width.

Normally the downstream face of the weir should be vertical as weirs with stepped / battered downstream face are not fitted to natural streams with a significant gravel / boulder transportation. In fact, continuous dropping & sliding of solids on downstream steps / sloping face provokes tear of the gabion nets. The weir upstream face should always be stepped in order to facilitate bonding between the gabions and earthfill. The earthfill on the upstream face not only makes the weir impervious, the earthfill's weight on the steps adds stability to the structure. Typical sections of gabion weir are shown in Fig. 3(a) & 3(b).

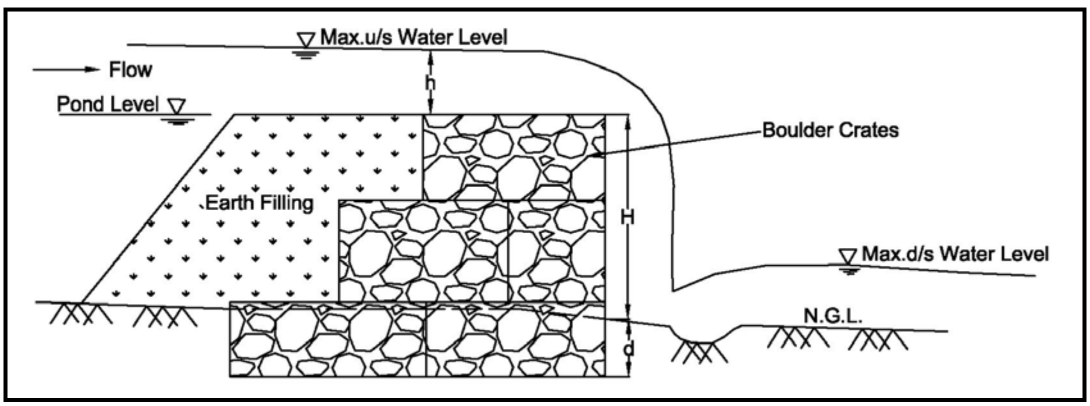


Fig. 3 (a): Typical Section of Gabion Weir

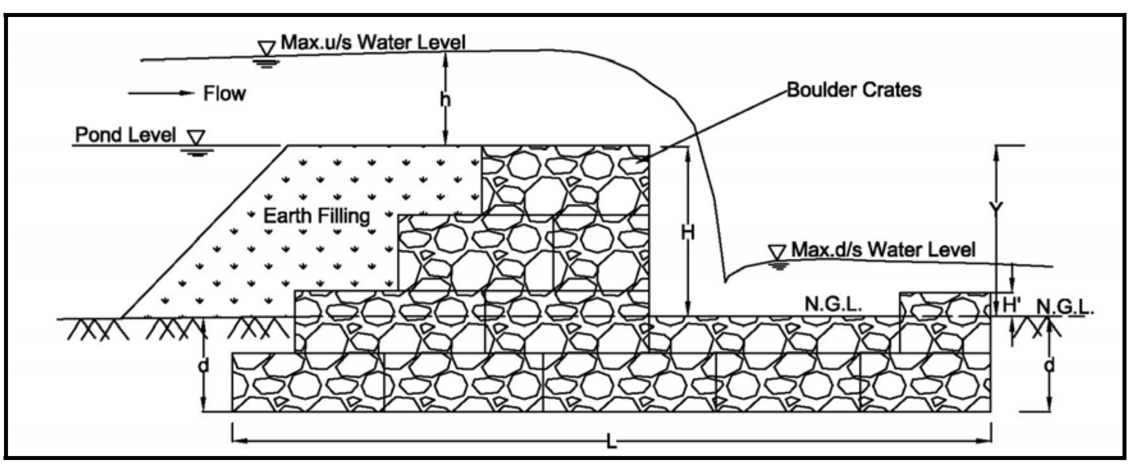


Fig. 3 (b): Typical Section of Gabion Weir with Downstream Protection Works

6.2.1 Design of gabion weir

(i) Hydraulic Design

The length of the weir across the flow is required to be sufficient enough to pass design flood. This is given by the following relation:

$$Q = c \times b \times h^{3/2},$$

Where,

Q	= Design flood discharge in cumecs
c	= Coefficient of discharge which varies from 1.71 for smooth top surface to 1.33 for rough surface,
b	= Length of weir across the flow in metres, and
h	= Water depth atop the weir in metres

For a gabion weir the maximum water depth atop the weir should be limited to 3 metres.

(ii) Depth of Foundation

The type of weir shown in Fig. 3(a) does not have provision of energy dissipation device in the downstream side. This type of weir is suitable only when the stream bed material is resistant to erosion. In such a case the foundation of the weir should be kept at impervious unerodable strata. The type of weir shown in Fig. 3(b) is applicable for pervious erodable strata. In this case the foundation of the weir should be kept at atleast 1.50 times the scour depth corresponding design flood discharge. The scour depth is given by the following relation

$$R = 1.35\left(\frac{q^2}{f}\right)^{1/3} \text{ metres,}$$

Where,

R	= scour depth in metres below design flood level,
q	= design flood discharge/ unit length of weir in cumecs/ metre, and
f	= silt factor

Silt factor 'f' can however be obtained from the following relation,

$$f = 1.76\sqrt{d_{50}},$$

(iii) Height 'H' of Counter Weir

The height 'H' of the counter weir may vary from H/4 to H/6 depending upon the site condition. Firstly a counter weir of height H/6 may be constructed. If the site conditions so require, this height can be raised to H/4 by putting additional layer of gabions.

(iv) Checking Against Piping

In order to keep the weir, safe against piping, the seepage length 'L' be checked from the following Bligh's relation:

$$L > C \times Y$$

Where,

L = seepage length in metres

= $d + l + d_1$ (refer Fig. 3(b))

C = a coefficient depending upon soil characteristics as shown in Table 2

Y = Maximum water level difference between upstream and downstream of weir in metres

Table 2: Bligh's Coefficient 'C' Values

Type of Soil	Sizes of Particles (mm)	C
Fine silt & mud	0.01 to 0.05	20
Coarse silt & fine sand	0.05 to 0.10	18
Fine sand	0.12 to 0.25	15
Medium sand	0.30 to 0.50	12
Coarse sand	0.60 to 1.00	10
Gravel	>2.00	9-4
Hard clay	0.005	6-3

(v) Stability Analysis

The main section of weir should be safe against the following stability tests:

(a) Against excessive pressure on foundation soil
(b) Against sliding
(c) Against uplift

The stability analysis shall be done for the loads and forces shown in Fig. 4.

The loads and forces in Fig. 4 are as below

Wg = Weight of gabions
H_u = Earth and Water pressure from upstream side
W_w = Weight of earth and water on gabions
Hu = Water pressure from downstream side
U = Uplift pressure corresponding upstream and downstream water levels

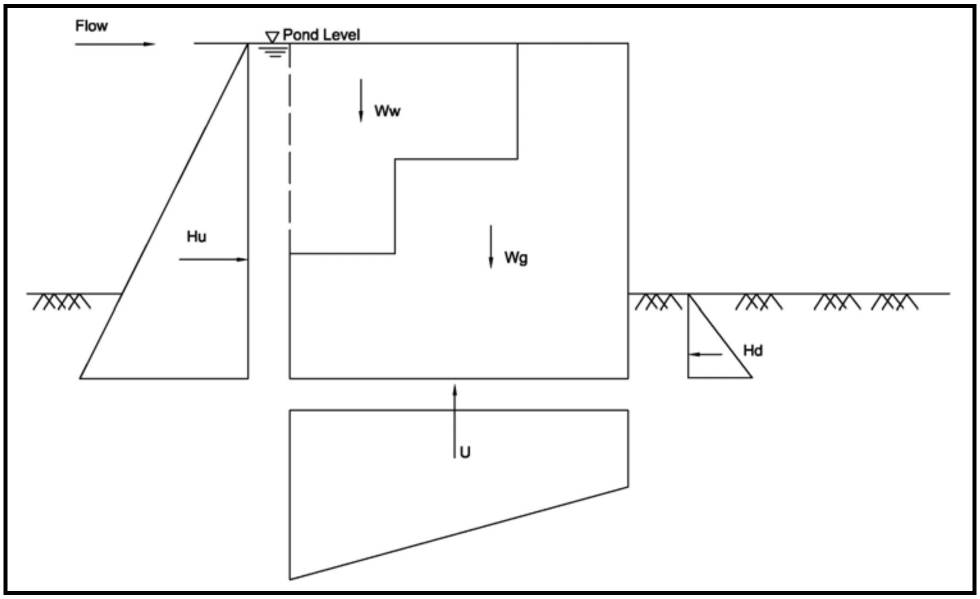


Fig. 4: Loads for Stability Analysis

The factors of safety against sliding and uplift shall be atleast 1.50 and 1.25 respectively.

6.3 Trench Weir

Trench weir (Tyroleam, bottom intake) is eminently suitable for small hydro schemes on steep hilly streams (>1 in 20) because of the following advantages:

(i) It is a simple structure and can be safely constructed across steep streams with permeable erodible strata.
(ii) The structure does not materially affect the regime of the stream.
(iii) The structure has capability to draw even the minimum flow of the stream very effectively and efficiently.
(iv) It can normally be adopted for discharges as small as 0.5 cumecs and upto 25 cumecs.
(v) The structure does not cause any obstruction for aquatic life movement.

The trench weir consists of a rectangular or trapezoidal trench constructed across the flow of the stream. The top of the trench is kept normally at river bed level. A trash rack is provided on top of the trench opening sloped in the direction of flow so that stones and pebbles do not settle but roll away with stream flow. The trench is provided with a bed slope of 1 in 12.5 less or so in the flow direction of the diverted water so that sufficient velocity is generated to carry along the silt that may find entry into the trench through the trash rack openings. The trench is so designed that it may carry atleast 1.2 times of the diverted discharge. The intake structure which is mainly a vertical rectangular well is located at the end of the trench weir well inside the river bank. It has three openings, the first at the exit end of the main trench, the other matching the location of the power channel and the third one at the bottom of the chamber on the downstream side for silt flushing. Sometimes a skimming slab is also provided with its top matching with the bed level of the power channel. It is provided so that the bed load that finds its way into the intake chamber is flushed out and does not enter into the power channel. The flushing discharge of the intake chamber is kept about 15 to 20% of that of the power channel. The above mentioned three opening of the intake well are controlled by vertical gates provided at the top of the intake chamber. The power channel originating from the intake chamber should be continued as a covered duct till a level is reached which is above HFL of the stream to prevent the flows from the stream entering into the water conductor system. Protection works are needed on the upstream and downstream of the trench. These works may comprise of suitable size boulder crates with a concrete slab of about 100 mm thickness on the top to protect the wire mesh from tearing or cement concrete blocks. These protections are provided both upstream and downstream of the trench. Concrete toe walls, one on the upstream and two on the downstream of the trench are provided to serve as a safe guard against erosion. Typical details of a trench weir are shown in Fig. 5 (a) and (b).

6.3.1 Design of trench weir

(a) Gauge Site Discharge Curve

A gauge discharge curve should first be developed at the weir site in order to arrive at the top levels of the side walls and the intake chamber which should be atleast one metre above the design HFL.

(b) Sizing of Trench

The top width of the trench in metres is given by:

$$B = \frac{Q}{E_1\, E_2\, C_d\, L \sqrt{2gE}},$$

Where,

Q = diverted discharge in cumecs
(This sould be atleast 1.2 times the sum of the following:
(i) Design discharge of the power house
(ii) Leakage, seepage or evaporation losses in the water conductor system
(iii) Flushing discharge needed at the desilting system i.e. flushing discharge at the intake of trench weir and flushing discharge at the desilting chamber, if any

L = Width of stream in metres

E_1 = $\dfrac{\text{Clear area of trash rack opening}}{\text{Total area of trash rack surface over trench}}$

E_2 = Ratio of trash rack openings likely to be clogged (It may be taken to be 0.5)

Cd = Coefficient of discharge through trash rack openings (It may be taken to be 0.46)

E = Specific energy at any section of the trench weir = $\left(\dfrac{Q}{CL}\right)^{2/3}$

Where,

C = Coefficient of discharge of broad crested weir (It may be taken as 1.7)

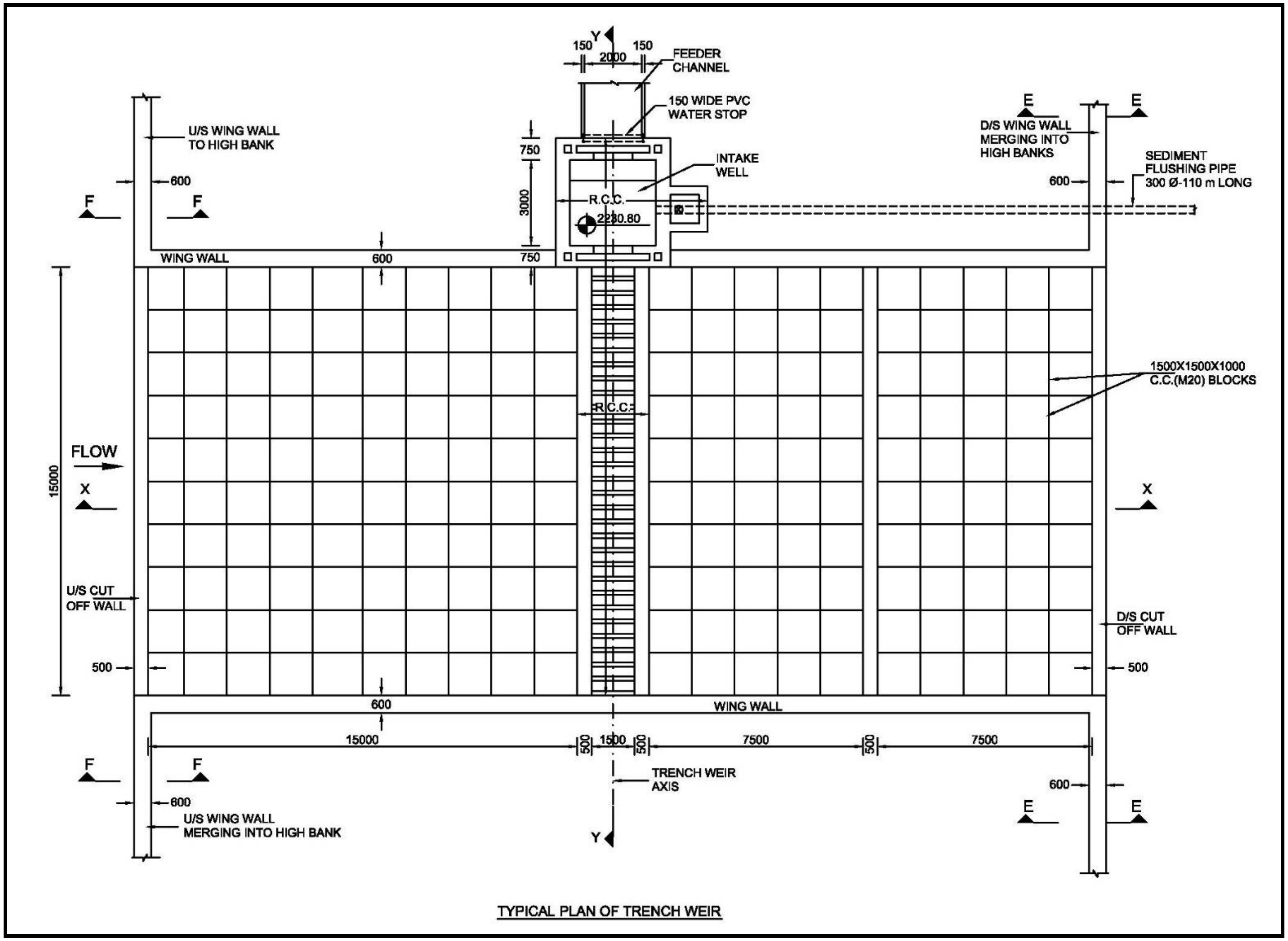


Fig. 5 (a): Typical Details of Trench Weir (Plan)

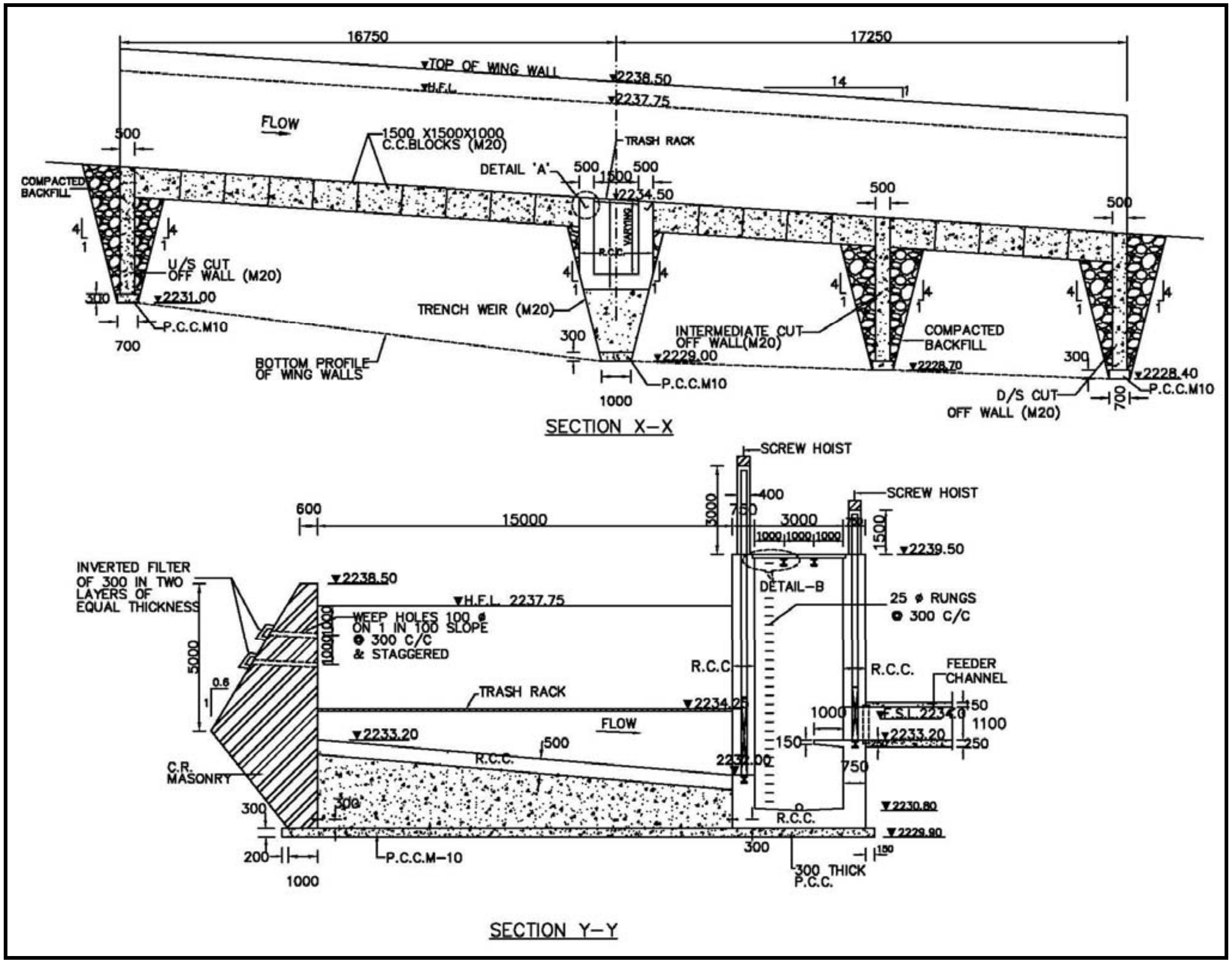


Fig. 5 (b): Typical Details of Trench Weir (Sections)

The total area of the top of the trench should be such that velocity through the total trash rack area even with 50% clogging should not be more than 0.4 metres / sec i.e.

$$B \geq \frac{2Q}{0.4 \times L}$$

If 'd' be water depth in metres along the section of the trench, Area 'A' = B×d, Velocity head 'h_σ' at the end of the trench is given by

$$h_\sigma = \frac{h}{h+1} \times \frac{A}{2B} = \frac{0.5}{1.5} \times \frac{Bd}{2B} = d/6,$$

where h is a constant depending upon the bottom profile of the trench and may be taken as 0.5.

The corresponding discharge is given by

$$\begin{aligned} Q &= A\sqrt{2gh_\sigma} \\ &= Bd\sqrt{2g \times d/6} \end{aligned}$$

The water depth evaluated above should be provided at the upstream end of the trench i.e. the end on the opposite bank of intake chamber. There should be a clearance of atleast 0.15 to 0.2 metres between the water level & the bottom of the racks.

The water depth at the intake end may be determined by providing a bed slope of atleast 1 in 12.5 in the trench. As such the water depth 'D' in metres at the intake end of the trench is given by

$$D \geq d + \frac{L}{12.5}$$

Average depth of the trench = $\frac{d+D}{2}$

& average area of trench in sq. metres = $\frac{B(d+D)}{2} m^2$

In case bottom half of the trench gets filled up with silt,

net average depth = $d_n \frac{d+D}{4}$ &

net area of the trench = $A_n = \frac{B(d+D)}{4}$

Thus half clogged section of the trench should be sufficient enough to pass the total discharge 'Q'.

This can be checked by the following Manning's equation:

$$Q = \frac{An.R^{2/3}.S^{1/2}}{n}$$

Where,

R = hydraulic mean depth

$= \frac{A_n}{B + 2d_n}$

S = Slope of the trench = $\frac{D - d}{L}$

n = Rugosity coefficient (This can be assumed to be 0.018)

In case the value of Q as calculated above happens to be lesser than required the area / slope of the trench will have to be increased.

(c) Depth of Cut off Walls

The depth of the upstream and downstream cut off walls below respective design flood levels should be atleast 1.5 and 1.75 times the scour depth respectively. Scour depth can be calculated from the following relation scour depth = $1.35\left(\frac{q^2}{f}\right)^{1/3}$ metres,

The bottom of the trench weir should also be kept atleast 1.5 times the scour depth below the HFL. Cutoff walls should also be provided below the side walls in such a manner that the bottom of the cutoff on the upstream matches with that of the upstream cutoff wall and the bottom on the downstream cutoff matches with that of the downstream cutoff wall.

(d) Silt Flushing Pipe / Channel

Silt collected at the bottom of the intake chamber is required to be flushed out regularly into the river. In order to do it, the bottom of the intake chamber is kept sufficiently below the bottom of the trench weir at that end. A flushing channel or a pipe of sufficient size (not lesser than 0.3 m diameter) is provided at the bottom of the intake chamber and is taken to the river at such a point where the HFL is atleast 0.5 metre below the invert of the flushing channel / pipe.

The flushing channel / pipe should have sufficient slope to generate a flushing velocity of 2.5 to 3.5 m/sec. In case a flushing pipe is provided the alignment of the pipe should be straight and the pipe sections should have flanged ends so that the pipe could be easily dismantled and cleaned in case it gets choked. The exit end of the flushing pipe/ channel should be well protected against erosion by river.

(e) Additional Safety Measures

The trench weir option is generally adopted on small stream having steep gradient. Generally such streams carry large amount of bed load comprising boulders upto 1 metre size or so. In such case the following safety measures are worth taken into account:

(i) Trash racks section should be quite enough so as to withstand pounding of the boulders.
(ii) A portion of trash rack should be easily removable so that the trench could be cleaned of the silt deposited into it (after flood).
(iii) The top portion of the trench should be steel lined with atleast 6 mm thick plate.
(iv) In very steep stream carrying large amount of bed load, old rails at about 200 mm centre to centre can be embedded in the upstream and downstream protection works. The heads of these raips shall protrude above the concrete surface as shown in Fig. 6. The rails shall be laid along the flow of the stream.

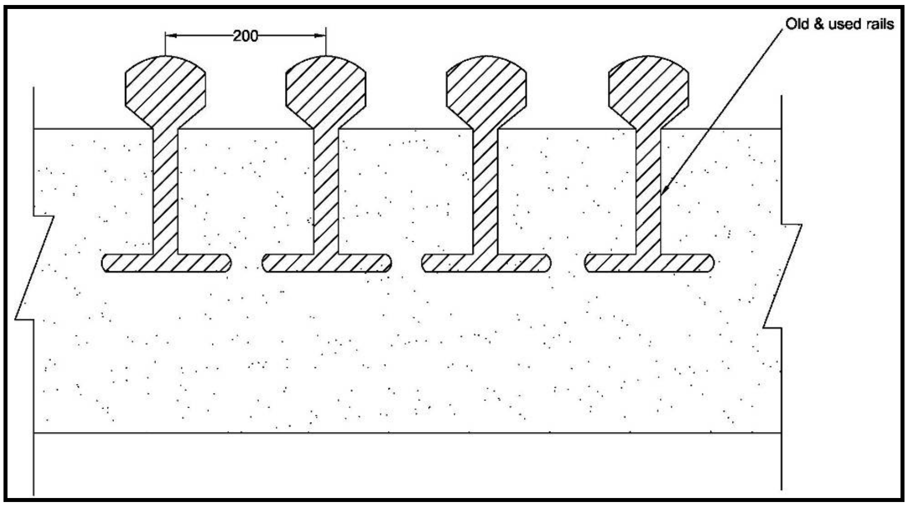


Fig. 6: Typical Arrangement of Protection of Trench Weir with Steel Rails

6.4 Raised Weir (barrage)

Raised weir (barrage) is generally constructed under following conditions:

(i) The discharge required to be diverted for power generation is large.
(ii) The stream has gentle slope (flatter than 1 in 100) and some additional head is required to be gained by raising the water level at the weir.
(iii) In case some storage is required for peaking purposes.

The weir can have erodable or or non-erodable foundation. The design for the two cases will have to be different.

6.4.1 Gauge discharge curve

Gauge discharge curve should first be developed at the proposed weir/ barrage site in order to arrive at the safe levels of side walls and design of energy dissipation devices.

6.4.2 Pond level

Pond level upstream of the barrage shall normally be obtained with the following considerations.

(i) Area allowed to be submerged on the upstream of the weir / barrage.

(ii) Head needed for required power generation
(iii) Geology of the weir / barrage site

6.4.3 Retrogression

Progressive retrogression of the stream on the downstream side as a result of construction of weir / barrage causes lowering of the downstream river stages which has to be suitably considered in the design of downstream energy dissipation works. While a retrogression of 1.25 to 2.25 m may be adopted for alluvial rivers, a retrogression of 0.3 to 0.5 m may be considered for hilly streams with well defined section.

6.4.4 Waterway

In deep and confined streams with stable banks, the overall waterway between abutments including thicknesses of piers should be approximately equal to the actual width of stream at the design flood. However for alluvial rivers, the following looseness factor may be applied to Lacey's waterway as per Table 3.

Table 3: Silt Factor and Looseness Factor

Silt Factor	Looseness Factor
Less than 1	1.2 to 1
1 to 1.5	1 to 0.6

Lacey's waterway is given by the following formula:

$$P = 4.83\, Q^{1/2}$$

Where,
P = waterway in metres
Q = design flood discharge in cumecs

The following additional considerations may be taken into account while deciding the final waterway:

(a) Cost of cutoffs and protection works – more the water way lesser will be depth of cutoffs & cost of protection works.
(b) Afflux constraints (It would be optimum if in case of barrages the upstream HFL and pond level are kept the same).

6.4.5 Levels of crest and upstream floor

The levels of the crest in the pocket upstream of the under sluices and upstream floor should normally be kept at the normal low bed level of the stream. For weirs, the crest level shall be kept at the required pond level. For barrages, the head required to pass the design flood at the desired maximum upstream level determines the crest level of the bays other than under sluice bays.

6.4.6 Shape of barrage / weir profile

Where the upstream floor level and the crest level are not the same, the crest may be provided flat with a width of about 2 m. The upstream and downstream slopes joining the respective floors incase of erodoble foundations may be 1 (H) :1 (V) and 3 (H) : 1 (V) respectively. On nonerodable rocky foundation the upstream & downstream slopes of the weirs may be 0.25 (H) : 1 (V) and 0.7 (H) : 1 (V) to 1:1 respectively.

6.4.7 Energy dissipation

For barrages / weirs resting on erodable foundation, hydraulic jump type stilling basin designed in accordance with IS:4997 would normally be required for energy dissipation. While designing such basin a provision of 20% concentration of flow may be provided. For barrages / weirs resting on unerodable foundations, the energy dissipation devises may be either of the following

(i) Hydraulic jump type stilling basin designed in accordance with IS:4997
(ii) Skijump or roller bucket designed in accordance with IS:11527

6.4.8 Under sluices

A weir / barrage requires a deep pocket infront of the intake so that bed load of silt may be got flushed downstream through the lower crest of the undersluice bays and the intake may be provided with comparatively clearer water. A divide wall is provided between the undersluice bays and the weir/ other bays of the barrage to produce favorable flow conditions and to eliminate mixing of the silt deposited upstream of the weir / other bays of the barrage. The width of undersuluice portion shall be determined on the basis of following considerations

(i) It should be capable of passing atleast double the intake discharge.
(ii) In should be capable of passing 10 to 20% of design flood discharge.
(iii) It should be wide enough to keep the approach velocities sufficiently lower than critical velocities to ensure maximum settling of suspended silt load
(iv) In case of weirs, it should be capable of passing fair weather floods.

6.4.9 Divide wall

The following guidelines shall be adopted in fixing the length of divide wall between under sluice bays and weir / other bays of the barrage:

(i) It shall normally cover atleast two – third to full length of the intake.
(ii) The upstream length shall not be less than two times the level difference between the crest levels of under sluice and weir / other bays of barrage.

6.4.10 Cut offs

Cut offs are generally required on the upstream and downstream of the floors in case when the structure in founded on erodable foundation. The upstream and downstream cutoffs should generally be provided upto a depth of IR and 1.25 R respectively below high flood level, where R is the depth of scour. The concentration factor shall be taken into account in determination of scour depth. The cutoffs should be either extended into both banks upto atleast twice their depth from the top of floors or the whole of the structure should be boxed by providing cutoffs in the side walls also with the bottom of these side cutoffs varying from the bottom of the upstream cutoff on the upstream side to bottom of the downstream cutoff on the downstream side. In addition to above, the depth of the downstream cutoff alongwith the total length of impervious floor should be sufficient to reduce the exit gradient to within safe limits. The cutoffs may be of masonary, concrete or sheet piles depending upon the type of foundation soil and depth of cutoff and to suit construction convenience.

The exit gradient may be determined with the help of Khola's curve. The factor of safety for exit gradient for different types of foundation strata shall be as per Table 4:

Table 4: Safe Exit Gradient for Different Sediment Strata

Sediment Size	Safe Exit Gradient
Shingle	4 to 5
Coarse sand	5 to 6
Fine sand	6 to 7

No cutoffs are required for weirs founded on impermeable foundation.

6.4.11 Protection works

Protections as mentioned below are needed to be provided for weir founded on erodible foundations only.

(i) Upstream Block Protection

Just beyond the upstream end of the impervious floor, protection works comprising cement concrete blocks of adequate size laid over a base of hand packed stones shall be provided. The cement concrete blocks shall be of adequate size so as not to get dislodged and shall generally be of size not less than 1500 x 1500 x 900 mm. The length of upstream block protection shall be equal to depth of scour below floor level. In case of very steep stream, the concrete blocks may be interconnected.

(ii) Downstream Block Protection

Downstream block protection shall comprise cement concrete blocks of adequate size laid over a suitably designed inverted filter. The cement concrete blocks shall generally be of size not less than 1500 x 1500 x 900 mm laid with gaps of 75 mm width packed with gravel. The length of this protection shall be equal to 1.5 times the depth of scour below flood level. A toe wall of masonary or concrete extending upto about 500 mm below the bottom of filter shall be provided at the downstream end of the inverted filter to prevent it from getting disturbed. The graded inverted filter should generally conform to the following design criteria:

(a) $$\frac{D_{15}\ of\ filter}{D_{15}\ of\ foundation} \geq 4 \leq \frac{D_{15}\ of\ filter}{D_{15}\ of\ foundation}$$

(D_{15} denote grain size such that 15% of solid grains are smaller this size).

(b) The filter may be provided in two or three layers. The grain size curves of the filter layers and the base material should be roughly parallel.

(iii) Loose stone protection

Beyond the block protection on upstream and downstream of a weir/ barrage located on erodable foundation, launching aprons comprising loose boulders shall be provided. The boulders used shall not weight less than 40 kg. These may be laid in a length of 1 to 1.5 times the scour depth and their thickness shall be from 1 to 1.5 metres.

6.4.12 Discharge

The discharge shall be obtained from the following formula:

$$Q = CLH^{3/2}$$

Where,

Q = discharge in cumecs,
C = coefficient of discharge (in free flow condition)
L = Clear waterway of the barrage or weir in metres, and
H = Head over the crest in metres

The value of 'C' depends upon many factors such a drowning ratio, shape and width and surface roughness of the sill and can, therefore, be assessed correctly only by model studies. In the absence of model studies, the value of 'C' can be determined from the curve shown in Fig. 7.

6.4.13 Total floor length and exit gradient

In case of weirs founded on erodible foundations a minimum length of the impervious floor has to be determined in order to keep the exit gradient within limits so as to eliminate the possibility of piping phenomenon.

Exit gradient 'G_E' is given by the following formula:

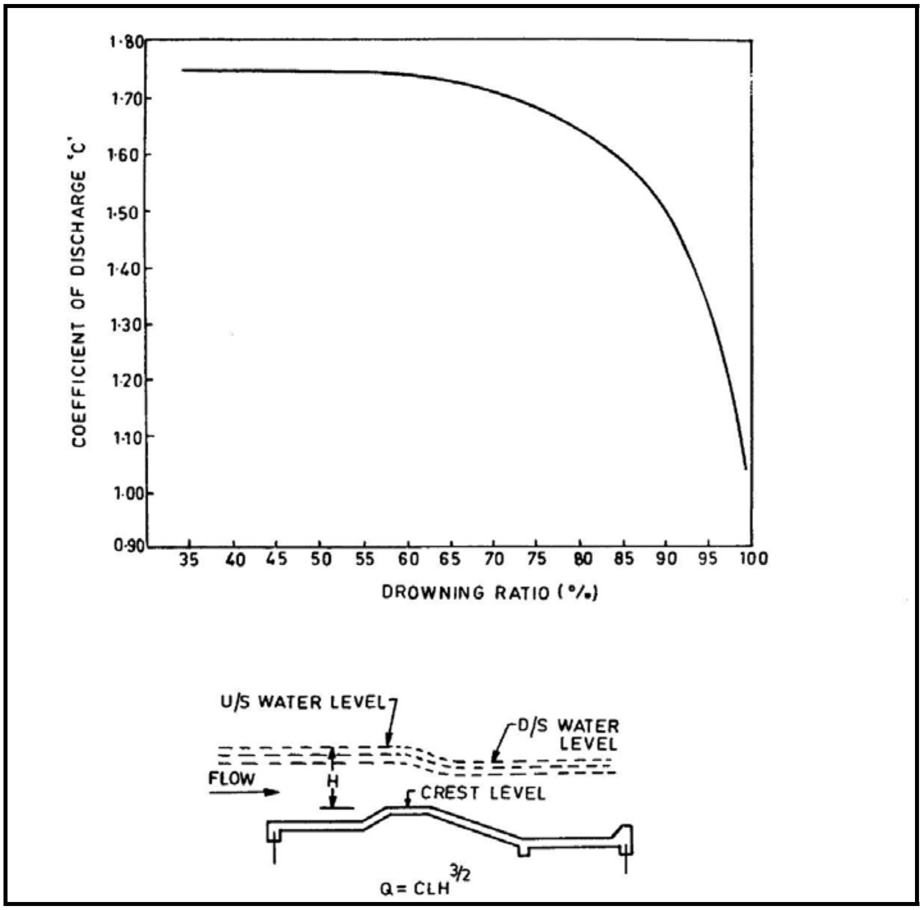
$$G_E = \frac{H}{d}\frac{1}{\pi\sqrt{\lambda}},$$

Where,

H = Maximum static head in metres,
d = Depth of downstream cut off in metres, and
$\frac{1}{\pi\sqrt{\lambda}}$ = a factor obtained from Khosla's curve against $\frac{b}{d}$, in which b is the total length of impervious floor in metres

After fixing the value of G_E on the basis of foundation strata & 'd' on the basis of scour depth, factor $\frac{1}{\pi\sqrt{\lambda}}$ can be determined and corresponding value of $\frac{b}{d}$, can be read on the Khosla's curve.

As the value of'd' is known, 'b' the total length of impervious floor can be determined. In case the length of the floor works out to be too large, it can be reduced by increasing the depth of cut off within practical limits. The minimum length of downstream floor is determined by energy dissipation consideration as per para 6.4.7. The length of the floor in excess of the downstream requirement is to be given on the upstream of the gate.



Where

Q = Discharge in cumecs
C = Coefficient of Discharge in Free Conditions
L = Clear Water Way in Meter
H = Head of Water Over Crest in Meter

$$Drowning\ Ratio = \frac{Downstream\,Water\ Level - Crest\ Level}{Upstream\ Water\ Level\ - Crest\ Level}$$

Fig. 7: Drowning Ratio Vs Coefficient of Discharge Curve (in MKS Units) (Based on Malikpur Curve) Applicable in Broad Crested Barrages

6.4.14 Floor thickness

After fixing the profile of the weir section from upstream to downstream end, uplift pressures at key points shall be determined from Khosla Theory for different flow conditions i.e.

(i) Static condition with gates closed, upstream water at full reservoir level and no water in the downstream side,

(ii) High flood condition

The uplift pressures are then plotted along the longitudinal section of the weir for the two conditions mentioned above. The floor thickness at any point should not be less than the maximum ordinate between the sub soil pressure line and the water surface or floor surface, if there is no flow, divided by (r-1), where r is the specific gravity of concrete which may be taken as 2.25.

For weirs founded on unerodable rocks, the uplift forces can either be balanced by providing floor thicknesses as mentioned above or they can be balanced by providing anchor bolts. For design of anchor bolts, the vertical component of the uplift load, less the dead load of concrete, is first determined. The anchorage value of the bars is determined on the basis of pull – out tests. In case pull out values are not available, the anchorage value can be obtained by the following relation:

$$P = \frac{2\pi d^2}{3} \times s$$

Where,

P	= ultimate tensile strength of the anchor bar in kg,
d	= Minimum depth of embedment of the anchor bar in metres,
s	= Allowable shear stress of the rock, which can vary from 4000 kg/m^2 to 5,500 kg/m^2 depending upon the quality of rock.

Normally the spacing of the anchor bolts is kept half the depth of embedment.

Typical details of a barrage founded on permeable foundation is shown in Fig. 8(a) and 8(b) weir founded on impermeable foundation is shown in Fig. 9.

6.5 Concrete / Stone Masonary Gravity Dam

Concrete / stone masonary gravity dams are generally adopted under following conditions:

(i) When the proposed diversion site is situated in a narrow valley and sound rock is available in the river bed and banks at a reasonal depth.
(ii) Where additional head is required to be gained without submergence of much of useful land.
(iii) In case some storage is required for peaking purposes.
(iv) Where construction materials i.e. aggregate, cement etc. are readily available within reasonable distance

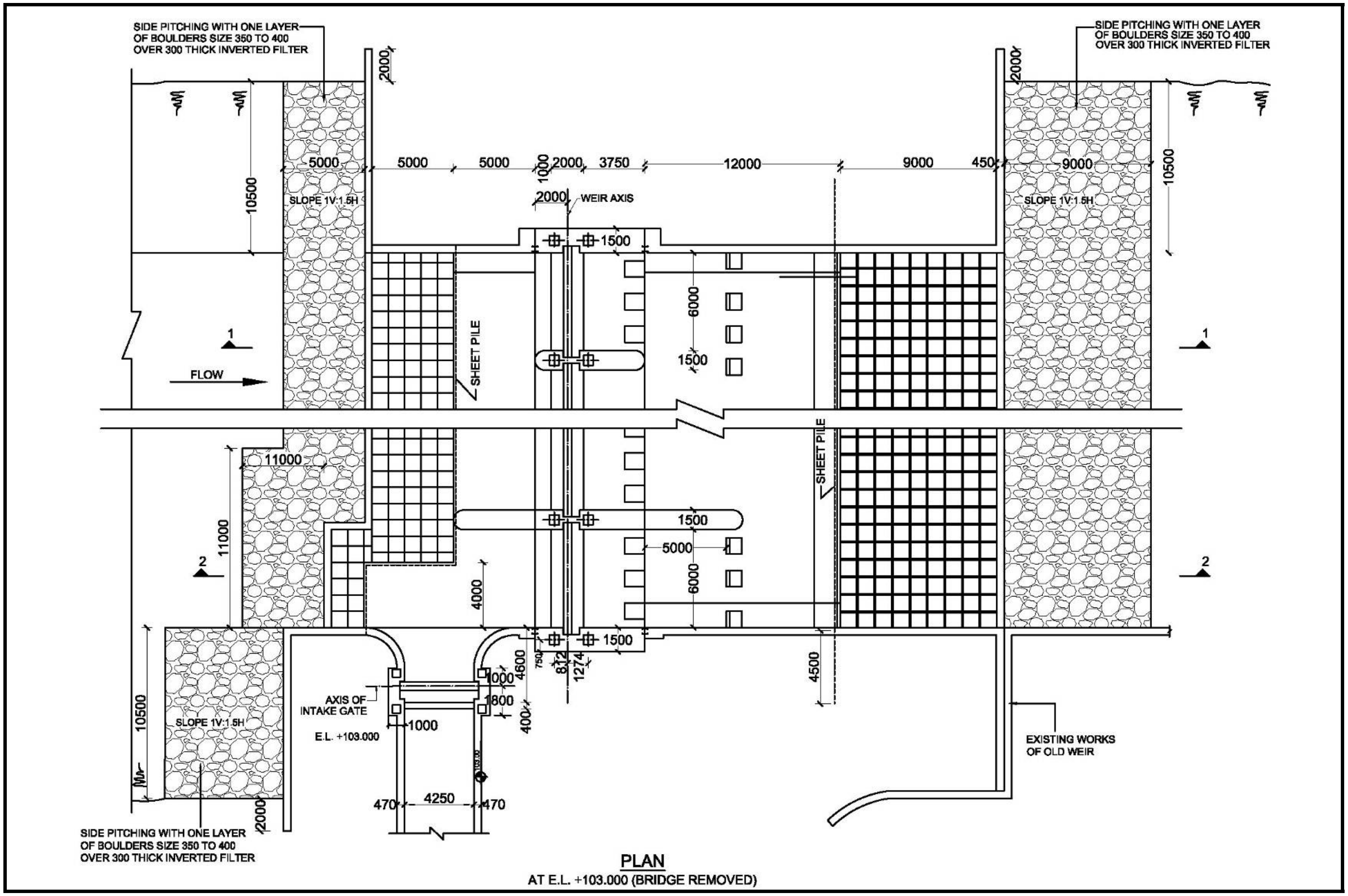


Fig. 8(a): Typical Details of a Raised Weir on Erodable Foundation (Plan)

Concrete / masonary dams have following advantages:

(i) They are simple in construction.
(ii) They have less amount of volume as compared to earthfull dams
(iii) They can facilitate overflowing in floods even during construction as well as during operation

The dams pertaining to SHP are normally of low height say upto about 30 m, as such stone masonary dams should be preferred because of the following advantages:

(i) They have less rigid requirements during construction.
(ii) They use less cement and make use of local materials.
(iii) They are labour intensive and can be constructed using local labour.
(iv) The heat of hydration is lesser.

6.5.1 Gauge discharge curve

Gauge discharge curve should first be developed at the proposed dam site in order to assess water levels on the downstream side of the dam. The G-D curve will also be helpful in designing the most efficient energy dissipation device on the downstream side of the spillway section.

6.5.2 Pond level

Full reservoir level / pond level upstream of the dam shall normally be obtained from the following considerations:

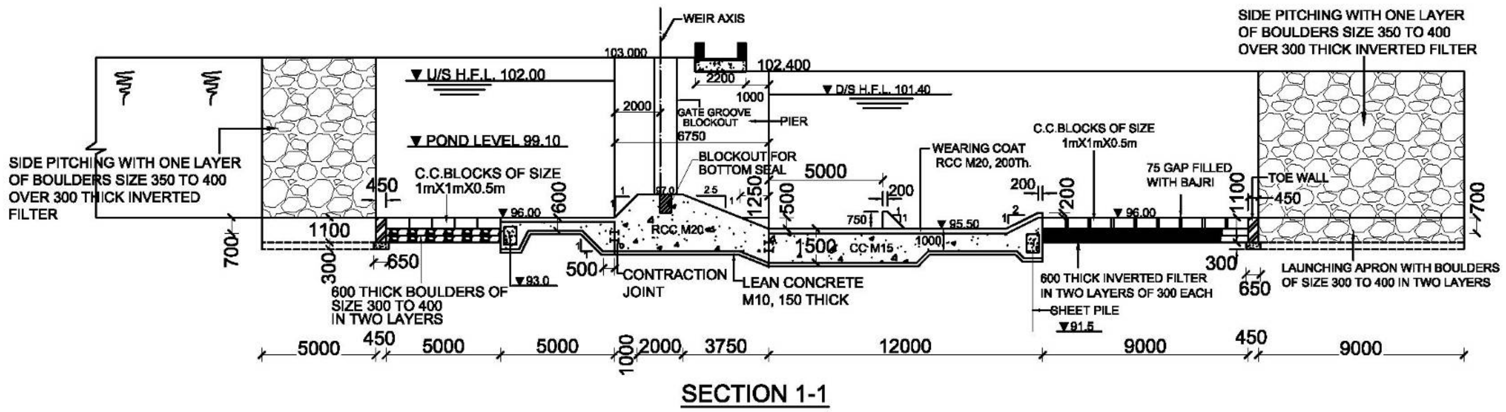
(i) Area allowed to be submerged on the upstream side of the dam at pond level / high flood level whichever is higher. (It would be better if the spillway is so designed that the pond level & high flood levels remain the same).
(ii) Head needed for power generation on the basis of techno-economic studies.
(iii) Geological conditions of the dam site.

6.5.3 Design of dam

Structural design of dam can be done on the criteria mentioned in IS:6512 – Criteria for design of solid gravity dams.

6.5.4 Design of spillway

The spillway is required to be designed to pass design flood at a pre-determined maximum allowable upstream water level. Spillway best suited to the prevailing site conditions can be designed in accordance with IS: 10137 – Guidelines for selection of spillways and energy dissipators, IS: 6934 – Hydraulic design of high Ogee overflow spillway and / or IS: 11485 – Hydraulic design of sluices in concrete and masonary dams and / or IS: 5186 – Design of chute and side channel spillways.



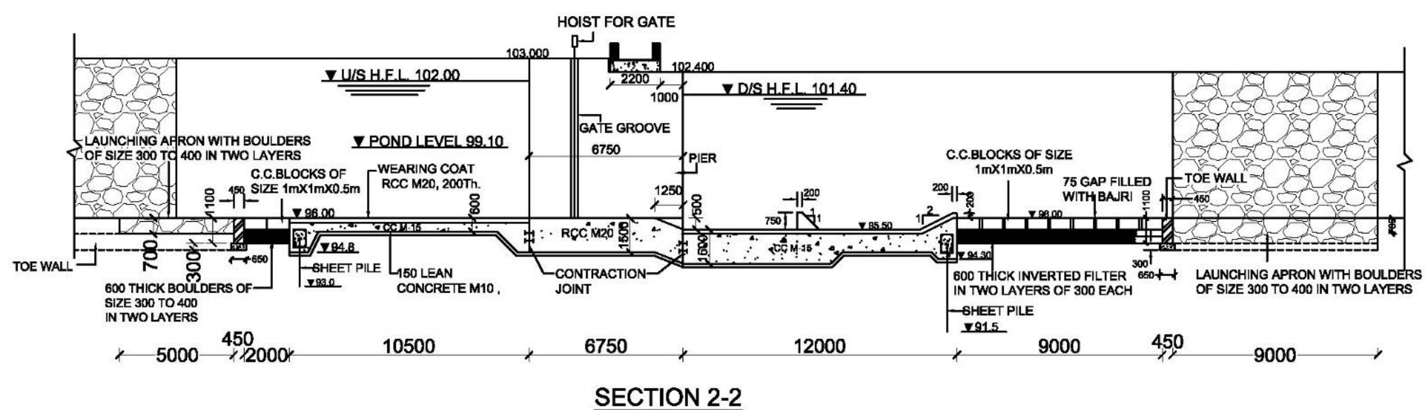


Fig. 8(b): Typical Details of a Raised Weir on Erodable Foundation (Section)

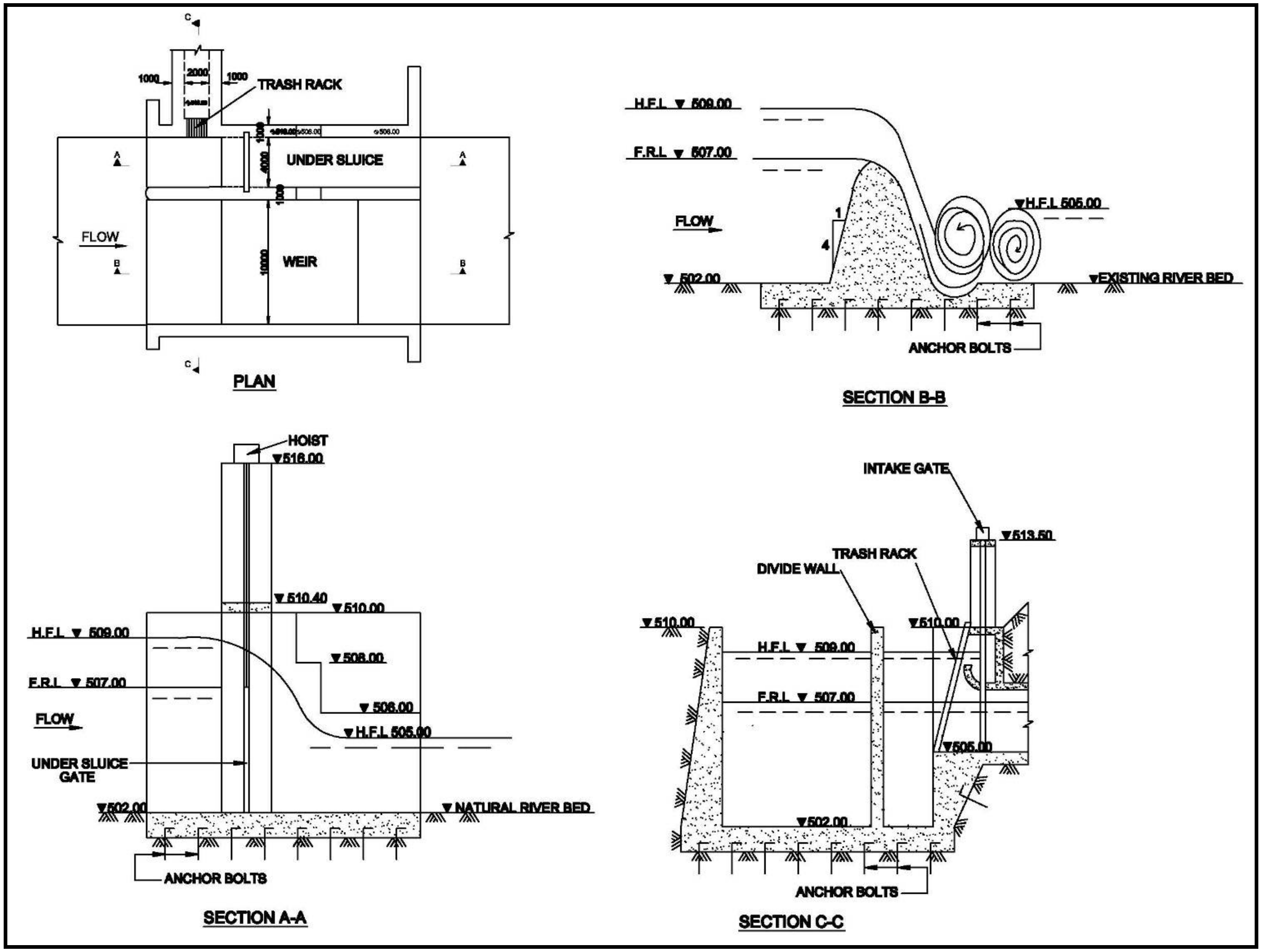


Fig. 9: Typical Details of a Weir on Impermeable Foundation

The design of proper energy dissipaters on the downstream side of the spillways occupies a vital place in the design of dams. The problem of designing energy dissipaters is essentially of reducing high velocity flow to a velocity low enough to minimize erosion of natural river bed. The reduction in velocity can be accomplished by any, or a combination of the following, depending upon the head, discharge intensity, tail water conditions and the quality of bed rock / material.

(a) Hydraulic jump type spilling basins
    (i) Horizontal apron type; and
    (ii) Sloping apron type

(b) Jet diffusion and free jet stilling basins
    (i) Jet diffusion basins,
    (ii) Free jet stilling basins,
    (iii) Hump stilling basins; and
    (iv) Impact stilling basins

(c) Bucket type dissipaters
    (i) Solid and slotted roller buckets; and
    (ii) Ski jump or flip buckets

(d) Interacting jets and other special type of stilling basins

The selection and design of energy dissipation devices best suited for a particular location can be done in accordance with the following IS: codes

IS: 10137	- Guidelines for selection of spillways and energy dissipaters
IS: 11527	- Criteria for structural design of energy dissipaters for spillways
IS: 4997	- Criteria for design of hydraulic jump type stilling basins with horizontal and sloping apron
IS: 7365	- Criteria for hydraulic design of bucket type energy dissipaters.

6.6 Roller Compacted Concrete Dam

Roller compacted concrete dam (RCC Dam) is structurally similar to a concrete gravity dam and can be deigned on similar lines. RCC dam construction is characterized by high-flyash cement in the heating zone of the dam. Similar to an earth dam the concreting of such type of dam is carried out by dumpers and vibratory rollers are used to for compacting the concrete. No contraction joints are provided in the dam body during initial course of construction. Transverse contraction joints are, however, sawed at regular intervals after compaction operation. Compared with conventional concrete gravity dam, the cement consumption per cubic meter of RCC dam as well as the overall cost of dam gets reduced by about 45% and 15% respectively and the construction period also gets reduced to a large extent. Fig. 10 which is based on RCC dam projects in USA in 1986. Unfortunately no roller compacted concrete dam has been constructed in India so far. It would, therefore, be worth while to consider such dams for small hydro projects.

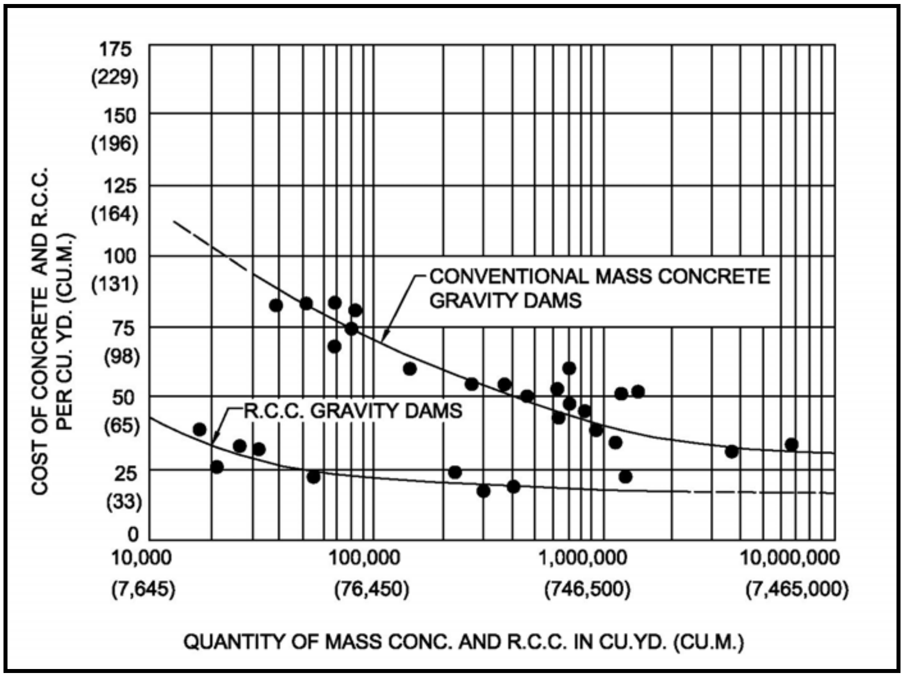


Fig. 10: Comparison of RCC and Mass Concrete Costs (1986 Dollars) (R-37)

6.6.1 Design considerations

Placing RCC in layers that are compacted by vibratory rollers does not change the basic design concepts for dams or other massive structures. However it does affect construction procedure. Design and layout of appurtenant structures, construction planning and treatment of joints should take into consideration the advantages and disadvantages of the rapid construction that is possible with RCC. Most placements of sufficient length and width to accommodate rollers and spreading equipment can economically benefit from RCC.

Three distinctly different concepts of design and construction of RCC dams have independently emerged in USA, Japan and the UK. They have been classified as follows:

(i) Lean RCC Dam

This concept was developed in USA. It is characterized by low paste content concrete to reduce the cost and the potential for heat generation. Total cementatious contents of about 120 kg/m^3 or less is used with flyash content ranging from 0-40% and aggregates have a relatively high proportion of natural fines.

(ii) Rolled Concrete Dam (RCD)

This concept was developed in Japan. The RCD mix contains cement content of 120-140 kg/m^3 with 20-30% flyash and is spread in three to four lifts compacted in one layer of 60-70 cm thickness.

(iii) High Paste Content RCC Dam

This concept of RCC was developed in UK and also accepted in USA. To achieve a density near to theoretical air-free density, a relatively high paste content is required. It has been possible to obtain both a low cement content and high paste content using pozzolana. A high paste mortar ratio also leads to improve joint properties and good bonding between the layers with a time gap upto three days between lifts. In the mix the cement plus flyash content is greater than 200 kg/m^3 with a high percentage of flyash.

Mix Design

The world's RCC dams constructed, under construction or planned are given in Table 5, which may be helpful to some extent in mix design. However, mix of RCC to be used for a specific project, will have to be designed through laboratory tests. The object of mix design is to produce a mix with maximum density and lowest cementatious material content compatible with even distribution of the paste through the mix and sufficient enough to ensure an adequate bond between layers. The following procedure of RCC mix design as outlined in ACI committee, 207, "Report on Roller compacted Concrete" can be used for mix design:

(1) Determine fly-ash / cement ratio (F/C), and Water / cementatious ratio (W/C+F) from the trial mix strength, using ACI curve, shown in Fig. 11.

(2) Determine the volume of coarse aggregate, (Vca) by dividing the loose bulk density of coarse aggregate to its specific gravity, in order to obtain an initial value for Vca.

(3) Calculate the volume of air-free mortar per cu. m (Vm) assuming 2 percent entrapped air, from:

$V_m = C_v\,(0.98 - V_{ca})$,

where C_v = unit volume of concrete.

(4) Determine of air free volume of paste content 'p'. A minimum paste / mortar ratio, 'P' is required for bond between layers and to yield a high relative density, from p = Px Vm

(5) Determine fine aggregate volume 'V_{fa}' from the relation

$V_{fa} = V_m - p$

(6) From F/c and W/ (C+F) ratio equations, and the known volume of paste fraction, p = C+F+W, the cement, fly-ash and trial water content can be calculated. The proportion by weight can be calculated by multiplying volumetric fraction to each material by specific gravity of the material.

(7) Check the consistency by varying water content of the mix to obtain the optimum water content which achieves maximum compacted density.

A typical section of RCC dam is shown in Fig. 12.

Table 5: RCC dams constructed, under construction or planned

S. No.	Name of Project	Country	River	Height (m)	Length	Dam Facing			Placement		Cementatious Content	
						Upstream Slope	Downstream Slope	Spillway Slope	Layers (mm)	Lifts (mm)	OPC (kg/m³)	Pozzolan (d)(kg/m³)
1.	Shmajigawa	Japan	Shimaji	89	240	V/0.30:1	0.8:1	0.8:1	150-200	500-700	91 84	39 36
2.	Holbeam Wood	UK	Lemon	12	80	V	0.8:1	0.8:1	250	250	105	0
3.	Willow Creek	USA	Willo	52	543	V	0.8:1	0.8:1	300	300	47	19
4.	Copperfield River	Australia	Copperfield	40	340	V	V/0.9:1	0.8:1	300	300	80	30
5.	Corraoll. E. (Ecton) Galesville	USA	Cow	51	291	V	0.8:1	0.8:1	300	300	53	51
6.	Castilblanco de los Arrayou	Spain	Cala	25	123	V	V/0.75:1	0.75:1	400	400	102 88	86 94
7.	Kengkou	China	Pingshan	57	123	V	0.75:1	0.75:1	300	300	70	50
8.	Craigbourne	Australia	Coal	25	247	V	1.0:1	1.0:1	300	300	70	60
9.	De Mist Kraal	S. Africa	Little Fish	30	300	V	0.6:1	0.6:1	250	250	58	58
10.	Arabic (now Mokgoma Matlala)	S. Africa	Olifants	36	455	V	V/0.75/0.5	0.75:1	300	300	36	74
11.	Bucca Weir	Australia	Kolan	12	128	V	N.A	0.5:1	300	300		90
12.	Zaaihoek	S. Africa	Slang	47	527	V	0.62:1	0.62:1	250	250	36	84
13.	Lower Chase Creek	USA	Lower Chase	20	122	V	0.7:1	0.7:1	300	300	64	40
14.	Upper Stillwater	USA	Rock Creek	90	825	V	0.32:1/0.6:1	0.6:1	300	300	79	173
15.	Los Morales	Spain	Morales	28	200	V	0.75:1	0.75:1	300-400	300 -400	81 73 69	140 127 153
16.	Les Olivettes	France	La Peyne	36	255	V	0.75:1	0.75:1	300	300	0	130
17.	Tamagawa	Japan	Tama	100	432	V/0.60:1	0.8:1	0.8:1	150-200	750	91	39
18.	Pirika	Japan	Shiribeshit-oshibetsu	40	910	V/0.80:1	0.8:1	0.8:1	150-200	500	84	36
19.	Elk Creek	USA	Elk Creek	51	365 786	V	0.8:1	0.8:1	150	600	70	33
20.	Santa Eugenia	Spain	Xallas	83	310	0.05:1	0.75:1	0.75:1	250	250	72 88	143 152
21.	Mano	Japan	Mano	69	239	V	0.8:1	0.8:1	150	500	96	24

S. No.	Name of Project	Country	River	Height (m)	Length	Dam Facing			Placement		Cementatious Content	
						Upstream Slope	Downstream Slope	Spillway Slope	Layers (mm)	Lifts (mm)	OPC (kg/m^3)	Pozzolan (d)(kg/m^3)
									200			
22.	Tashkumir	USSR	Narin	75	320	V	0.75:1	0.75:1	400	400	90	30
23.	Stagecoach	USA	Yampa	46	115	V	0.8:1	0.8:1	300	300	71	77
24.	Longmentan	China	Chang Creek	58	57	V/0.3:1	0.75:1	0.75:1	300	300	54	86
25.	Knellpoortt*	S. Africa	Rietspruit	50	200	V	0.5:1	0.5:1	250	250	61	142
26.	Tianshenqiao No2	China	Nanpanjaing	59	499	V	0.75:1	0.75:1	300	300	55	85
27.	Xitou*	China	Jiyang Creek	47	119	V	0.325:1	0.325:1	250	250	80	120
28.	Asahiogawa	Japan	Ogawa	84	260	V/0.90:1	0.8:1	0.8:1	150-200	500	n/a	n/a
29.	Cuesta Blanca	Argentina	San Antonio	83	793	V	0.8:1	0.8:1	200	600	75	15
30.	Wolwedans*	S. Africa	Great Brak	70	268	V	0.5:1	0.5:1	250	250	58	136
31.	Shangban	China	Wenchuan	50	122	V	0.75:1	0.75:1	n/a	n/a	n/a	n/a
32.	Wriggleswade	S. Africa	Kubusie	34	780	V	n/a:1	n/a:1	250	250	n/a	n/a
33.	La Pucbla de Cazalla	Spain	n/a	70	n/a	V	0.75:1	0.75:1	300-400	300-400	n/a	n/a
34.	Taynangba	China	n/a	55	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
35.	D’Aoulouz	Morocco	n/a	79	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
36.	Kennedy’s Vale	S. Africa	Groot Dwors	50	250	V	0.7:1	0.7:1	200	200	40 40	110 60
37.	Los Canchales	Spain	n/a	18	200	n/a	n/a	n/a	n/a	n/a	n/a	n/a
38.	Shiromizugawa	Japan	Shiromizu	55	367	V	0.8:1	0.8:1	150-2000	500	n/a	n/a
39.	Plalanovryssi	Greece	Nestos	95	270	n/a	n/a	n/a	n/a	n/a	n/a	n/a

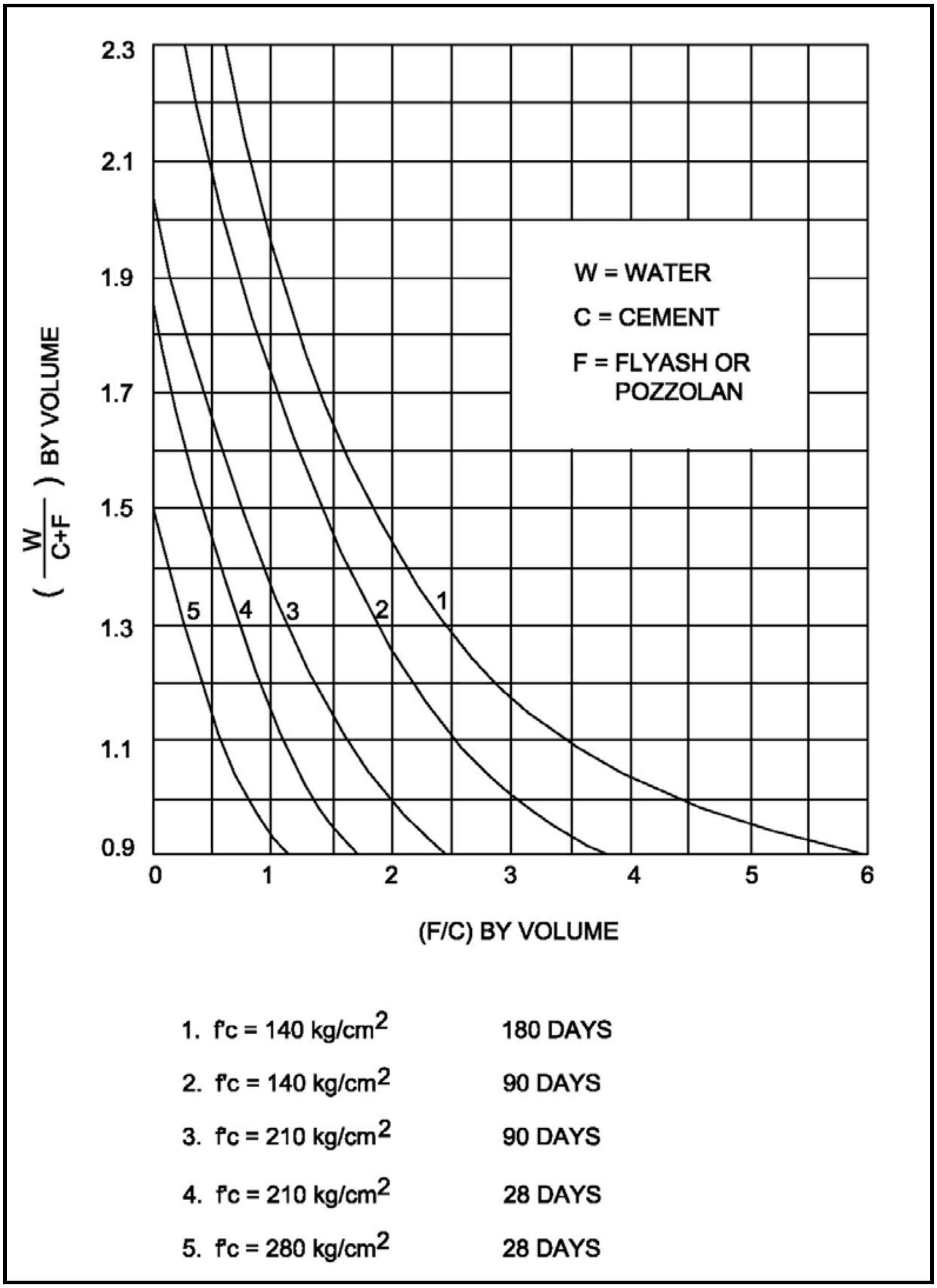


Fig. 11: Proportioning Curves for Equal Strength Concrete (R-37)

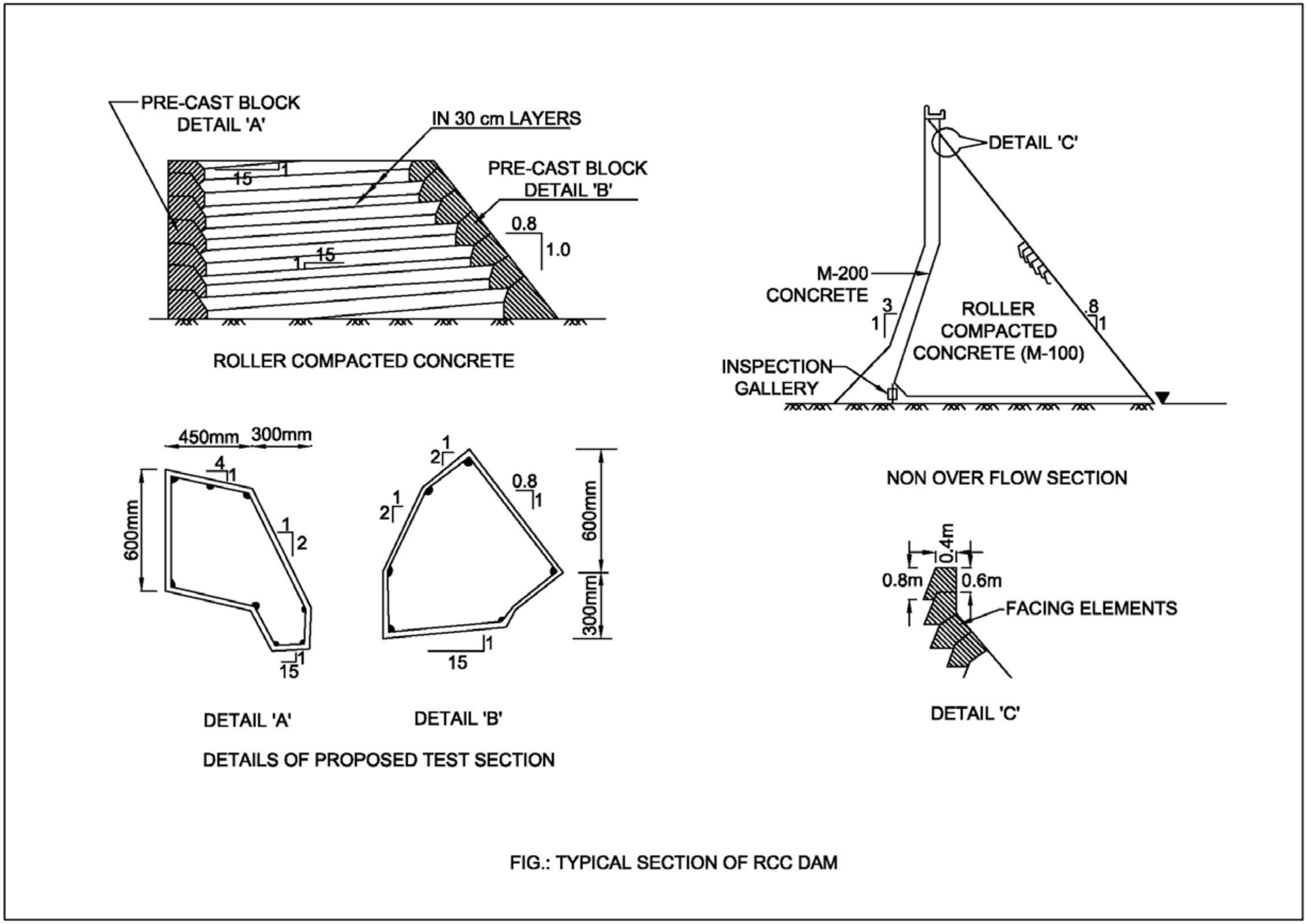


Fig. 12: Typical Section of RCC Dam

6.7 Concrete / Stone Masonary Arch Dam

These types of dams have not been constructed in India so far but a number of such dams have been constructed in China and it would be worthwhile to start constructing such dams in India also. These dams are suitable when the diversion dam is proposed at a narrow gorge of a river with very good geological conditions not only in the river bed but in the abutments also. In such dams almost all of the water and silt pressure exerted on the upstream side of the dam is transferred to the abutments via the dam body. As such the geological soundness of the abutment rocks is very essential. In this case uplift pressures on the base of the dam are not very important as the weight of the dam is not the primary force for resisting upstream silt and water pressures. These types of dams require less quantity of concrete / masonary and are, therefore, relatively cheaper than solid gravity type dams. In addition to this the arch dam can not slide. The compressive forces which act in three dimensions in the body of the dam tend to increase the impermeability of the dam.

Broadly the suitable range of adopting an arch dam can be taken to be as below

$$\frac{L}{H} \approx 1.5 \, to \, 5,$$

Where,

L = Length of crest of the dam, and

H = Height of the dam

Typical plan and section of an arch dam are shown in Fig. 13.

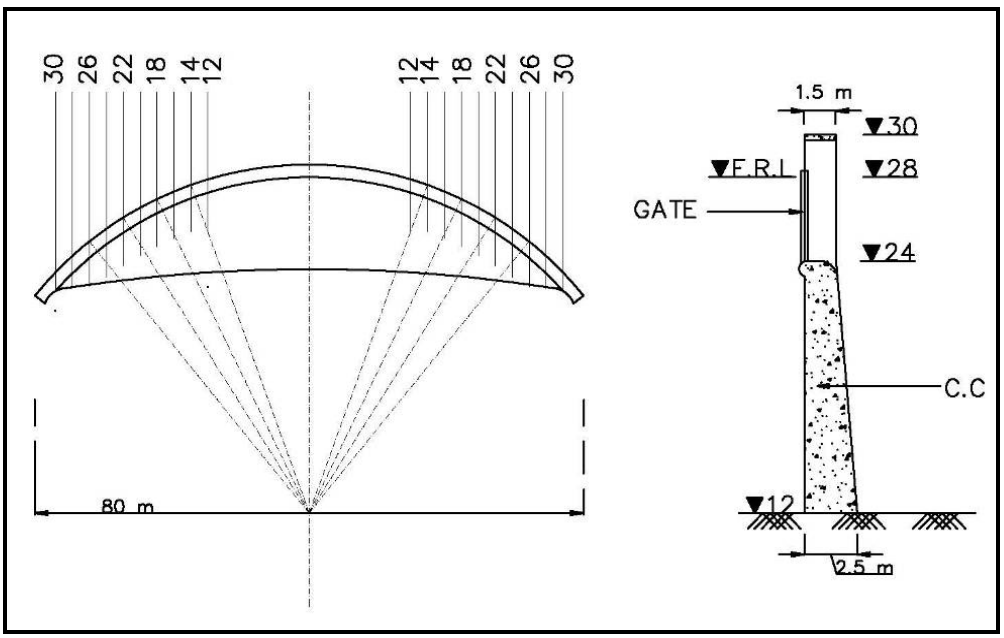


Fig. 13: Typical Plan and Section of Arch Dam

6.8 Inflatable Rubber Dam

Inflatable rubber dam is actually a rubber gate its action being, just like falling shutters. It consists of a rubber bladder anchored to a concrete foundation across the stream, which is proposed to be dammed. The wall of the rubber dam consists of a convas of polyamide or nilon fiber with protective layers of polychloropene rubber or neoprene hot pressed with synthetic rubber. The rubber sheet is available in rolls and can be installed without a mid structure upto 150 m width of the stream. A typical cross-section of the rubber dam is shown in Fig. 14. The rubber dam can be constructed for 0.5 m to 5.5 m height, though dams more than 4 m height are very rare.

The concrete foundation, downstream energy dissipation devices and upstream and downstream protection works are similar to those of raised weir. The top of the concrete works where rubber dam is installed is normally be kept at river bed level, so that when deflated, the work does not materially change the regime of the stream.

The rubber dams can be fixed on to the foundation by either single line anchoring or double line anchoring (Fig. 14). While single line anchoring system is suitable for low overflow and minimal tailwater conditions, double line anchoring system increases the stability and reduces the vibration of the dam body allowing higher head water of overflow on top of the dam. The rubber dams can be installed with any side slope angle upto 90^o. The dam can either be air filled or water filled. While water filled dam is almost free from pulsations or vibrations, it is not very much suitable for cold climates as the freezing of water inside the rubber body may rupture the dam.

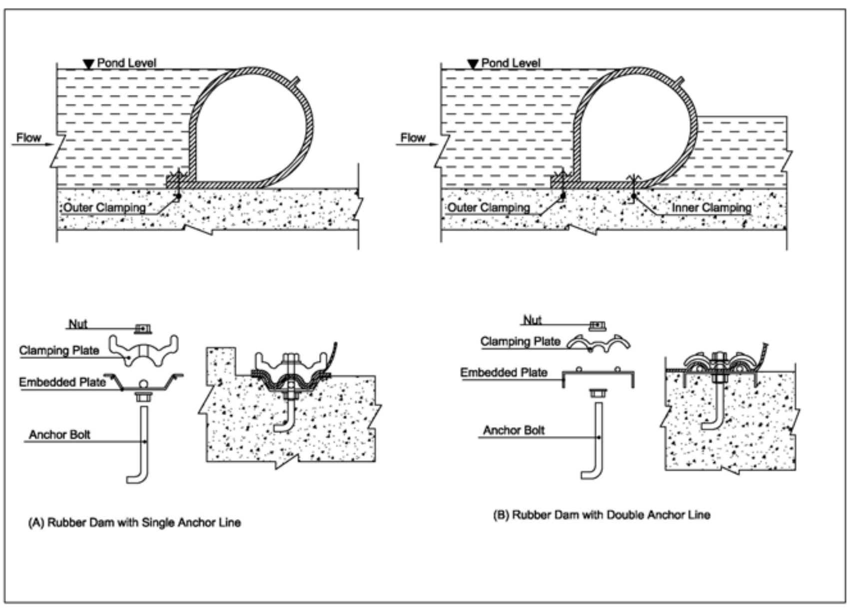


Fig. 14: Typical Cross Section of Inflatable Rubber Dam

6.8.1 Operation

The rubber dams are operated from a control room with fully automatic, semiautomatic or manual supply of air / water upto the specified or desired pressure and height. An ultrasonic sensor can be used for detecting the rise and fall of the water level upstream of the dam. When the air / water is fully exhausted, the rubber dam lies flat to the bottom thereby completely opening the channel for free flow of water. The control system can be designed to operate unsupervised under all adverse conditions.

6.8.2 Merits and demerits of a rubber dam

The rubber dam has following merits

(i) Rubber dam are quite cost effective for spans over 40 m and height less than 3 m.
(ii) With rubber dams the use of steel and other construction material is greatly reduced.
(iii) The dam can be installed in a week or so as all its components are factory made.
(iv) There is far less obstruction for passing the floods.
(v) The rubber dam has good resistance to earthquakes.

The rubber dam has following demerits

(i) The service life of a rubber dam is 20 to 30 years only.
(ii) It needs more maintenance specially in silt carrying rivers.
(iii) It is difficult to repair under water.
(iv) It is highly sensitive to typhoons

6.8.3 Anchoring of rubber dam

Anchorage of the rubber dam is an important feature in the design of this type of dam as any accident of the anchorage will result in failure of the dam.

6.8.3.1 Anchorages at bottom

The anchorage pattern can be classified into (i) single line anchorage and (ii) double line anchorage as shown in Fig. 14. Steel plates and anchor-bolts are generally used to fix the dam bag to the base slab.

6.8.3.2 Anchorage at the sides

Anchorages can be carried out in side walls, which may be either inclined or vertical, as indicated in Fig. 15. In such a case when the dam bag is inflated, there are folding gaps formed above the foot of the slopes as shown in this figure. Also when the dam is deflated, there are folding or wrinkles formed near the foot of the slopes.

The problem of the folds & wrinkles shown above can be solved by two methods called breast-wall arrangement and pillow type arrangement.

(a) Breast wall arrangement

In this arrangement the dam bag is anchored in the base slab only and the two ends are of hemi-spherical shape and are stretched through the breast wall as shown in Fig. 16. In this case the configuration of the inflated dam should match with the underlying curve of the breast wall giving some pressure on the curved surface and making it water tight. Thus the folding – gaps and wrinkles are eliminated but the cost of breast wall will be additional.

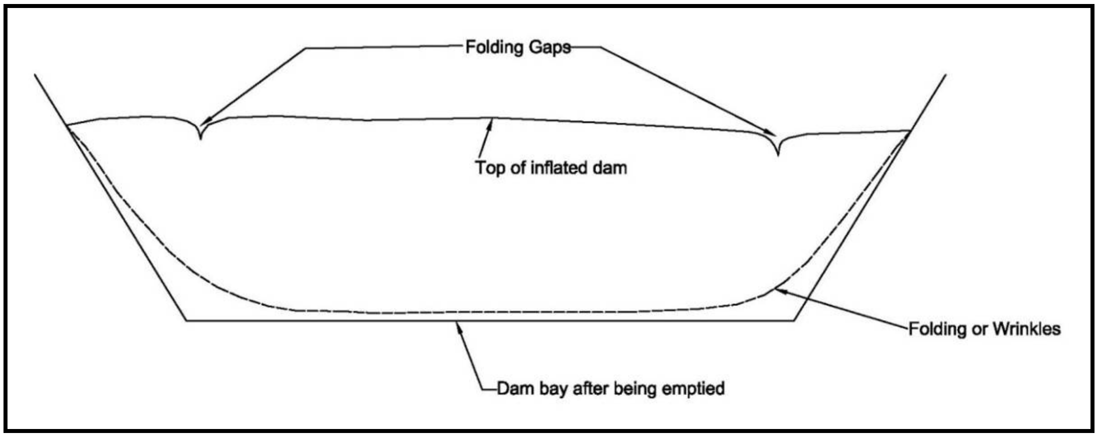


Fig. 15: Folding gaps or Wrinkles of Conventional Rubber Dam

(b) Pillow type arrangement

In this case, both ends of the rubber dam are evenly closed by rubberized canvas and are in direct contact with the vertical abutment or intermediate pier as shown in Fig. 17. The ends of the inflated dam body come in tight contact with the vertical wall, thus eliminating the leakage through both ends and wrinkles & folds.

6.8.4 Protection against bed load

In streams carrying heavy silt load, once the silt gets deposited upto the top of the rubber dam, it would be advisable to deflate the rubber dam and let the silt pass over the deflated body. The silt while passing over the diflated dam body scratches the rubber body which gets damaged with times. Some times sponge layers are installed inside the dam body or directly on the foundation. The sponge layer gives some cusioning effect to rolling bed load and results in reduction to the damage of the dam body (Fig. 18).

6.9 Earthen Dam

Earthen dams have the following advantages:

(i) It is generally possible to design an earthen dam to be safe on any type of foundation provided that the properties of foundation materials are thoroughly explored.
(ii) They can be designed using any type of locally available material.

6.9.1 Criteria for safe design of earth dams

The following criteria are required to be satisfied for safe design of an earth dam:

(i) There should be no possibility of the dam getting overtopped by flood waters.
(ii) The seepage line should be well within the downstream face.
(iii) The upstream and downstream slopes should be stable under worst conditions.
(iv) The foundation shear stresses should be within safe limits.
(v) There should be no opportunity for free flow of water from upstream to downstream face.

(vi) The dam foundations should be safe against piping.

(vii) The upstream face should be protected against wave action and the downstream face against rain cuts.

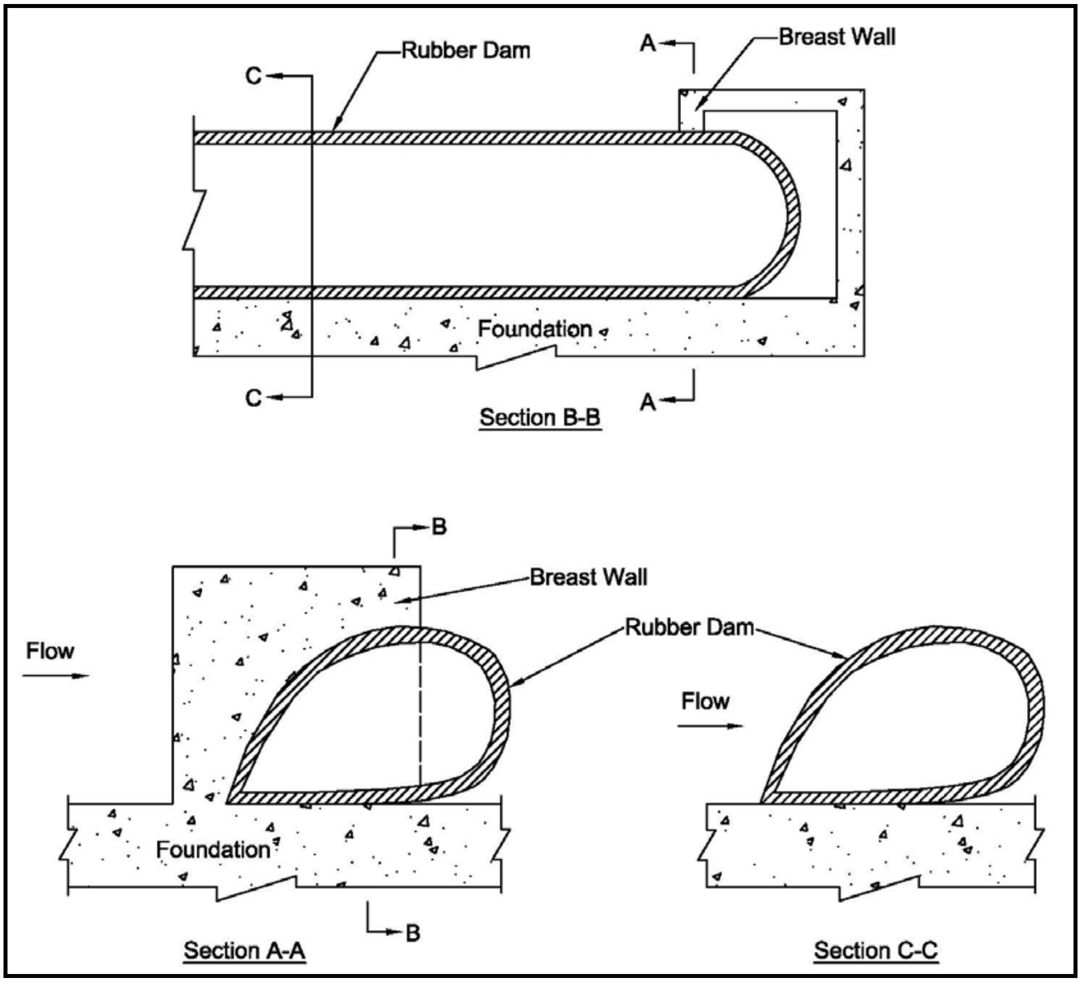


Fig.16: Breast Wall Arrangement

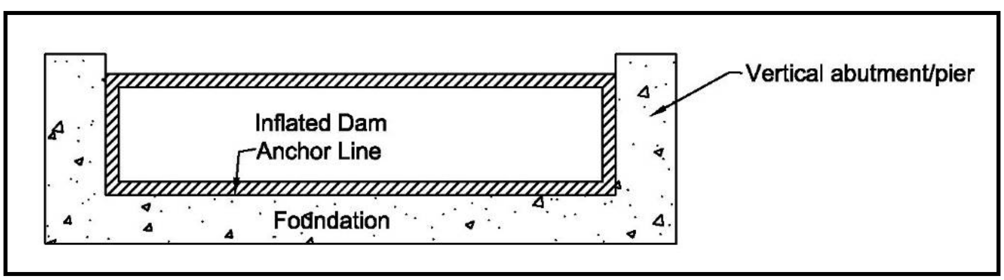


Fig. 17: Pillow Type Arrangement

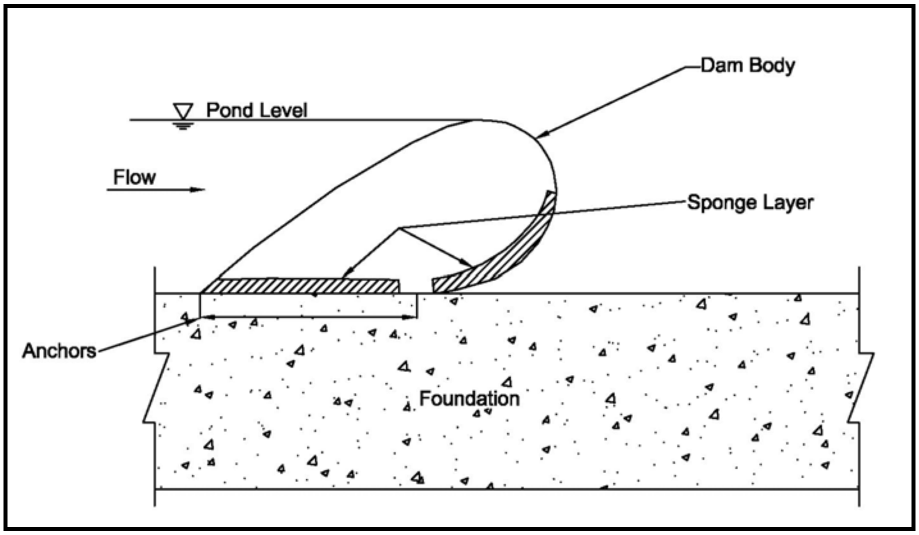


Fig. 18: Typical Arrangement of Sponge Layers inside Dam Body

6.9.2 Safety against overtopping

The following precautions are required to be taken against overtopping:

(i) Providing ample spillway capacity – As the overtopping of an earthen dam would result in failure creating catastrophic condition in the downstream side, the spillway flood hydrograph should not be underestimated and spillway capacity should be on the conservative side.

(ii) Provision of sufficient freeboard – The freeboard should be sufficient to prevent overtopping by waves and should take into account the settlement of embankment and foundation. Freeboard for wave run up on slope shall be provided in accordance with the provisions contained in IS:10635 – "Freeboard Requirement in Embankment Dams – Guidelines". Analysis should also be made for computing the settlement of the embankment and of the foundations in order to determine extra freeboard as settlement allowance. For unyielding foundation, the amount of settlement for embankment should be restricted to 1% of the dam height. But for compressible foundations, the settlement should be computed on the basis of laboratory tests. Longitudinal camber varying from zero height at the abutments to maximum at the location of the maximum height of the dam should be provided at the top of the dam along the dam axis to take into account the settlement.

6.9.3 Control of seepage line

It is necessary to ensure that the seepage line in the proposed dam section does not cut the downstream face of the dam and thereby produce softening or sloughing of the toe. In order to keep the seepage line well within the dam section, internal drainage system comprising the following needs to be provided.

(i) Inclined or vertical filter – Inclined or vertical filter abutting the downstream slope of the impervious core is provided mainly to collect seepage emerging out of the core and thereby keeping downstream section relatively dry (Fig. 19). The filter can be designed on the basis of criteria mentioned in clause 5.4.12, (ii). For three layered inclined or vertical filter, the maximum total horizontal width of 3.0 to 4.5 m is necessary.

(ii) Horizontal filter – Horizontal filter collects the seepage from the inclined or vertical filter or from body of the dam and carries the seepage to the toe drain. (Fig. 19). For horizontal filters minimum thickness shall be 15 cms for sand and 30 cms for gravel.

(iii) Rock Toe – The main functions of the rock toe are to facilitate drainage of the seepage water and to protect the lower part of the downstream slope from tail water erosion. For dams of low to moderate heights the rock toe should be ¼th to 1/3rd of dam height depending upon the availability of rock.

(iv) Toe Drain – The toe drain is provided at the toe of the earth dam to collect seepage from horizontal filter and rain water falling on the downstream slope and to discharge the same away from the dam (Fig. 19).

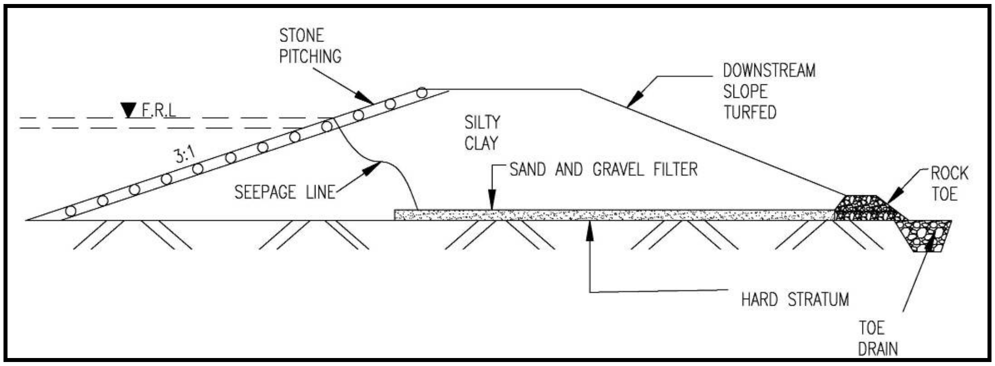


Fig. 19: Typical Section with Silty Clay

6.9.4 Control of seepage

The seepage through an earth dam is controlled by providing an effective cut-off. It has the following two functions:

(a) To reduce loss of stored water through foundation and abutments, and
(b) To prevent subsurface erosion by piping.

6.9.4.1 Curtailment of seepage through embankment

(a) By impervious membrane – The impervious membrane is placed at the upstream face of a rock fill dam. It has the following advantages as compared to an earth core:

(i). The dam with upstream membrane has a greater margin of safety against shear failure because of low pressures in the rockfill, larger rock mass to resist water pressure and water pressure having larger downward component,
(ii). The pervious rock embankment develops no uplift as it permits free percolation of water upward from the foundation,
(iii). The upstream membrane is exposed for inspection and repairs,
(iv). The dam can be raised by dumping rock on the downstream side and then extending the membrane upwards on the upstream sloping face,
(v). The dam has comparatively lesser volume of fill.

A dry rubber layer between main rockfill and upstream impervious membrane may be provided to act as a cushion for equalizing settlement and providing an even surface for laying the membrane. The upstream membrane may be of reinforced concrete or asphaltic concrete.

(i) Reinforced concrete membrane – The reinforced concrete membrane on the upstream face of a rockfill dam may consist of size RCC slabs of thickness 1% of head of water with a minimum of 300 mm. The membrane have vertical contraction joints at a spacing of 12 to 18 metres with closer spacing near the perimeter. The horizontal joints are provided near the perimeter only. Water stops are required to be provided to make joints water-tight. The concrete slabs shall be provided with reinforcement in both directions and equal to about 0.5% of the concrete volume. For 300 mm thick slabs the reinforcement may be provided in one layer but for thicker slabs it should be provided in two layers. (Fig. 20).

(ii) Asphalic concrete membrane – The asphalic concrete membrane is cheepaer than concrete, it is more flexible, it can be constructed rapidly and the protion of the membrane above water can be easily repaired. Being soft, it is more easily damaged than concrete. The asphalic concrete should consist of well graded aggregates with about 10 percent filler material passing through 75 micron IS sieve. Pure asphalt binder of about 8 to 10 percent by mass of aggregate should be used. The material should be mixed and compacted hot. An air content of 2 to 3 percent is optimum. A purely asphaltic concrete should have a thickness of about 1 percent of the head of water with a minimum of 300 mm. It should be placed in about 50 to 75 thick layers and rolled.

A cut off wall, which is also called plinth, is provided near upstream toe of the membrane of a rock fill dam to provide water tight connection between membrane and foundation (Fig 21). The dimensions of cut-off wall / plinth are based on following requirements:

(i) The contact length on foundation should be adequate to provide a minimum seepage path equivalent to one – twentieth to one – tenth of head of water depending upon rock quality but not less than 3m,
(ii) It should serve as a cap for consolidation / curtain grouting,
(iii) It should provide surface to facilitate putting of membrane form work, and
(iv) It should provide atleast 1 m rock fill under membrane to permit the membrane to deflect normal to its face.

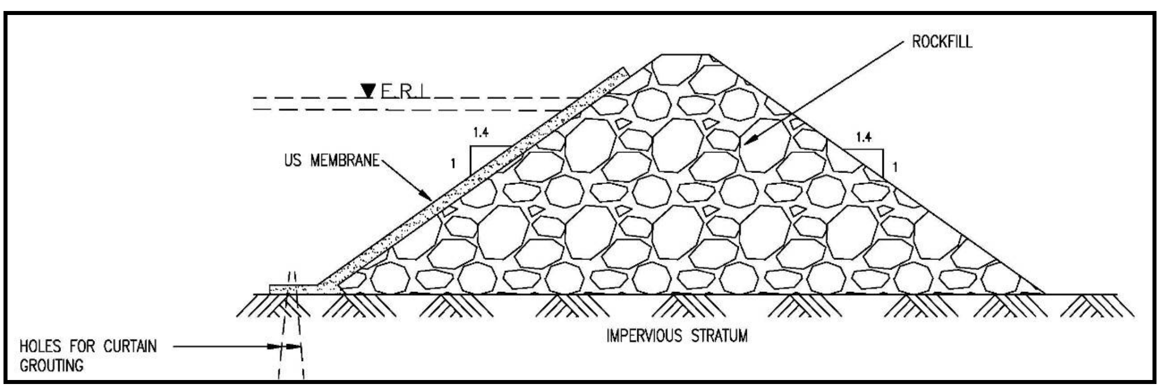


Fig. 20: Typical Section through Rockfill Dam

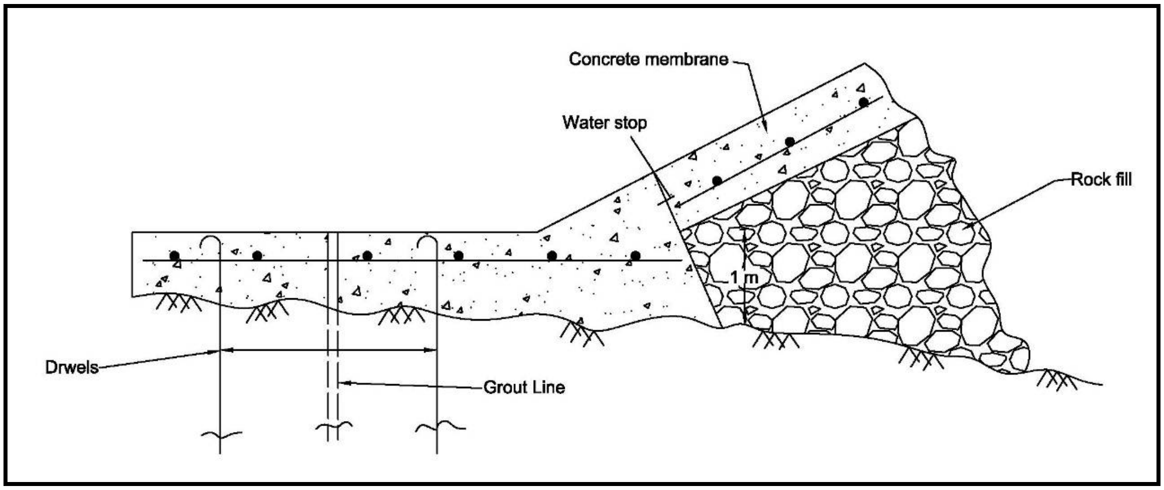


Fig. 21: Typical Details of a Cut off / Plinth

(b) Impervious core – An impervious core may be used in earth dams as well as rock-fill dam. Impervious soils are generally suitable for the core. IS 1498 may be referred for suitability of soils for core. The core may be located centrally or inclined upstream. The main advantage of the central core is that it provides higher pressures at the contact between the core and the foundation reducing the possibility of leakage and piping (Figs. 22, 23, 24 and 25). The inclined core, on the other hand, reduces pore pressures at the downstream portion of the dam and thereby increasing its safety. The inclined core also permits construction of downstream casing ahead of the core and allows the use of relatively large volume of random material on the downstream. The core thickness of 1/3 to ½ of water head with minimum top width of 3.0 m is generally satisfactory. In highly seismic zones the core thickness should be kept on higher side. The top level of the core should be kept atleast 1.0 metre above the maximum water level (Figs. 22, 23, 24 and 25).

(c) Curtailment of seepage through the foundation – This can best be obtained by a complete vertical cut off through pervious strata upto impervious ledge below. A vertical cut off can be provided by one of the following methods.

(i) Cut off trench – A trench is excavated upto the impervious ledge for a depth 1 to 2 m inside it. The bottom width of trench should be sufficient enough to provide working space for compaction equipment & to provide safety against piping. A minimum width of 4 m is recommended. In order to satisfy piping requirements, the width should be 10 to 30 percent of hydraulic head. The cut off trench should be backfilled with the same material as that of the impervious core. The cut off in the flanks should normally extend upto the top of the impervious core. In case the rock ledge into which the cutoff is terminated happens to be weathered or has cracks & crevices, necessity of providing grouting curtain may arise. Rocks having lugeon value more than 5 should be grouted. The use of cut off trenches is very reliable but it becomes too expensive beyond depths of about 20 m (Figs. 23, 24 and 25).

(ii) Concrete cut off walls – Concrete cut off walls of 1.5 to 2.0 m thickness can be provided by pouring concrete in a sheeted and braced trench.

(iii) Slurry trenching and concreting – A trench is excavated by a trenching machine and is kept filled with bentonite slurry which prevents caving of the sides. The trench is then backfilled with concrete or impervious soil (Fig. 26).

(iv) Impervious blanket – A horizontal upstream impervious blanket in combination of the impervious core is provided to increase the length of seepage path when full cut off is not practicable. In this case the foundation is subjected to seepage pressure and as such it becomes necessary to collect seepage water of the dam section into a gravel filter and that of the foundation through relief wells (Fig. 27). Reference may be made to IS: 1498 for suitability of soils for the blanket.

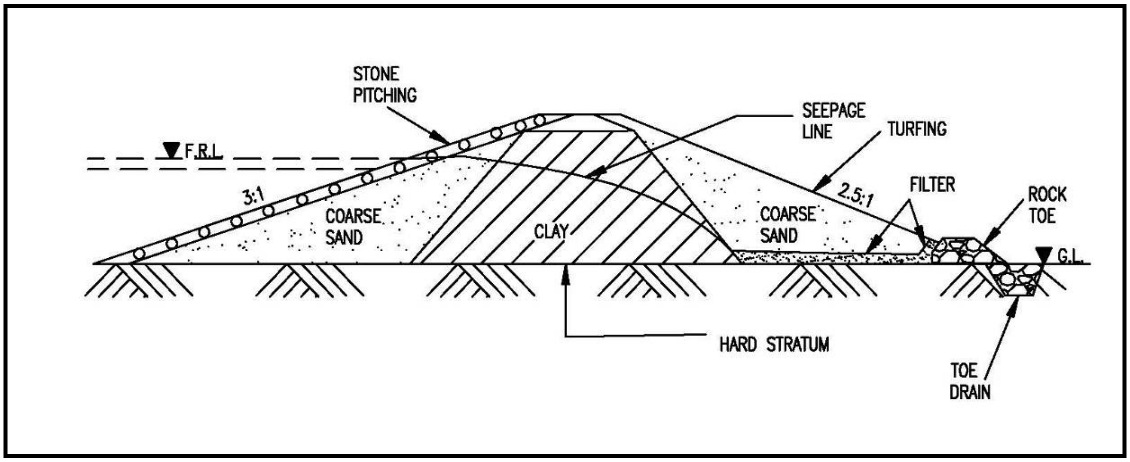


Fig. 22: Typical Earth Dam Section with Coarse Sand Material

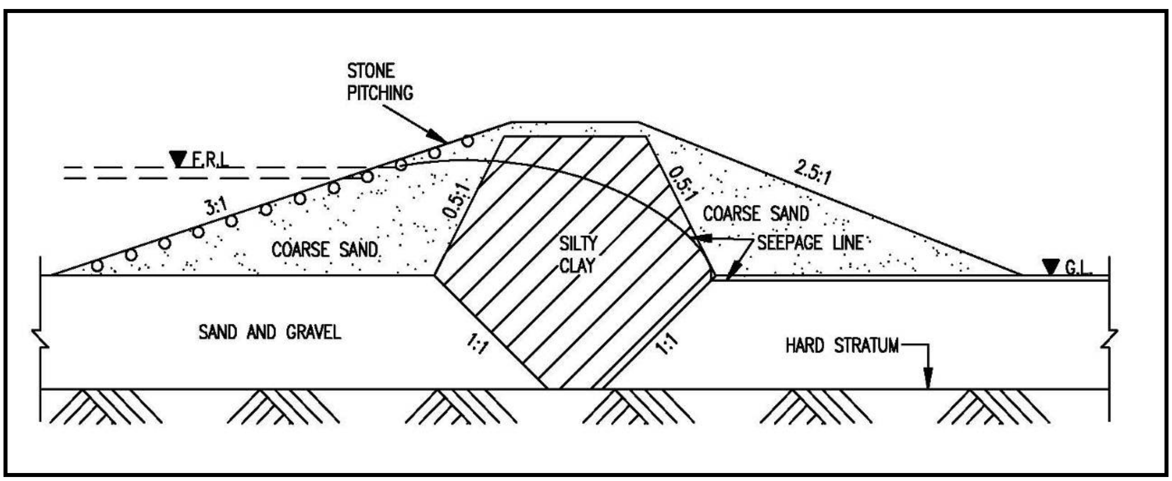


Fig. 23: Typical Earth Dam Section with Coarse Sand

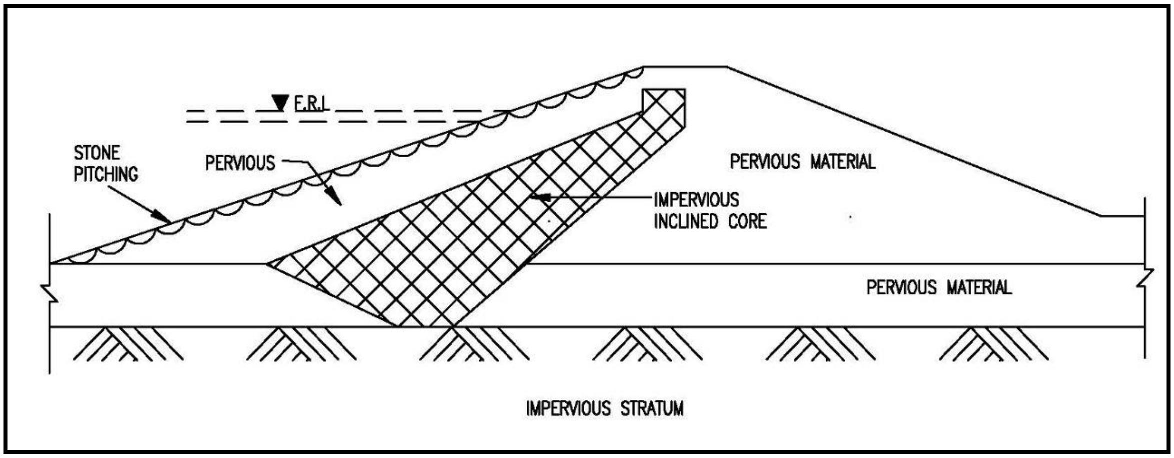


Fig. 24: Typical Earth Dam Section with Pervious Foundation upto Small Depth

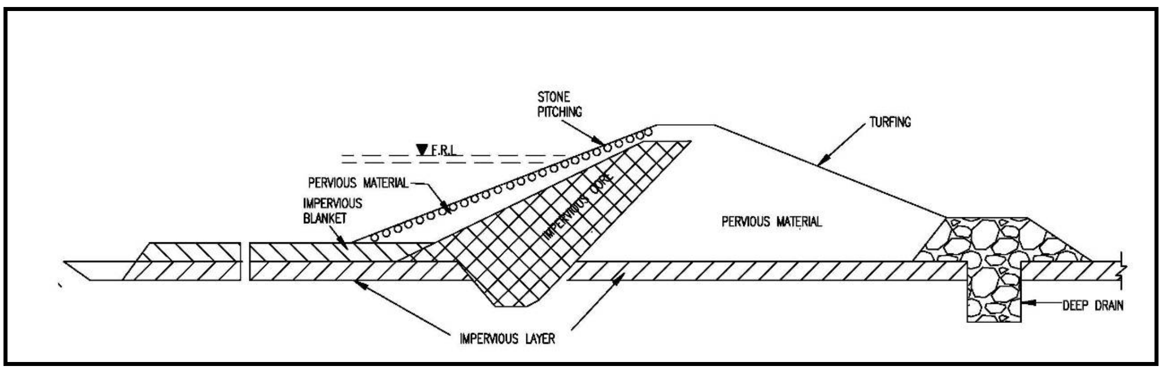


Fig: 25: Typical Earth Dam Section with Foundation Pervious to Large Depth

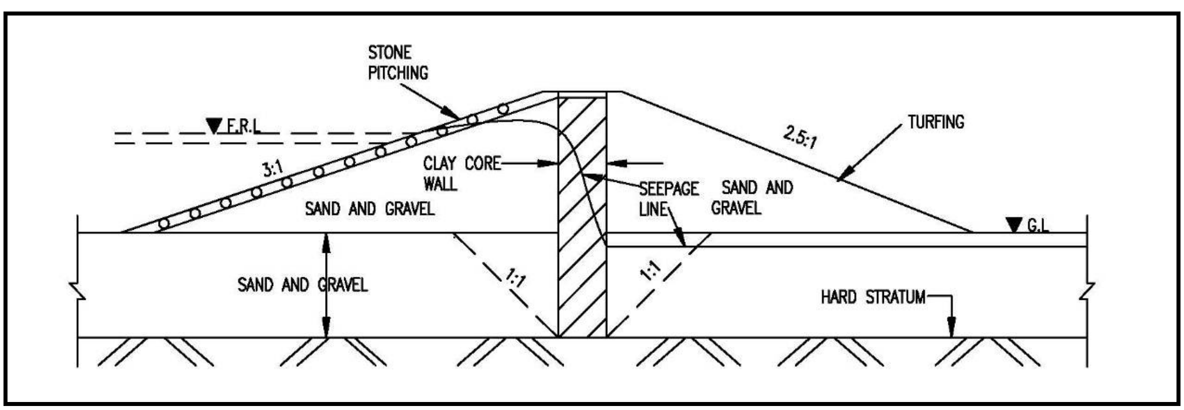


Fig. 26: Typical Earth Dam Section with Foundation Pervious to Large Depth

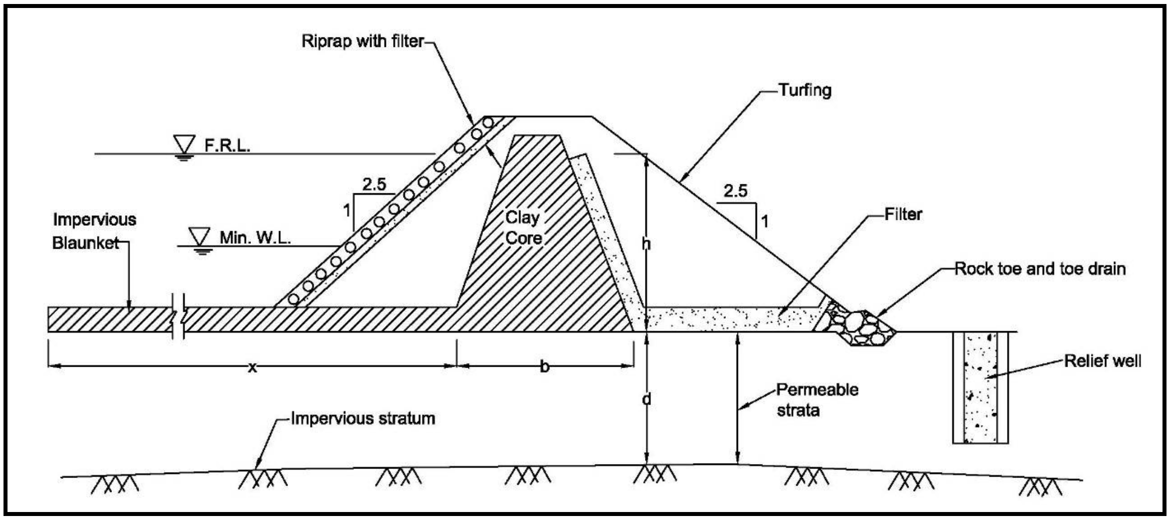


Fig. 27: Section of Earth Dam with Impervious Blanket

A 30 cm thick layer of random material may be put over the blanket to prevent its cracking due to exposure to atmosphere. The design of impervious banket can be done as per IS: 8414.

Assuming the permeability of the blanket material to be negligible and rate of loss of head to be uniform, an approximate estimate of the effectiveness of the blanket can be obtained as follows:

With reference to Fig. 27,

Let b = base width of the impervious core,
h = seepage head,
k = permeability of permeable strata
d = depth of permeable strata upto pervious strata,
and x = length of blanket

Seepage discharge, q, per unit length of dam without blanket is approximately given by,

$$q = k \frac{h}{b} d$$

Similarly seepage discharge, 'q_b', with blanket is given by

$$q_b = k \frac{h}{b + x} d$$

$$\therefore \quad \frac{q_b}{q} = \frac{b}{b + x}$$

Usually the blanket is 1.2 to 3.0 m thick and is made about 8 to 10 times the head in length.

(v) Relief wells – Relief wells are an important adjunct for seepage control. They are used not only with upstream impervious blanket but in other cases also to provide additional assurance that hydrostatic pressures do not develop in the downstream portion of the dam, which could lead to piping. They should extend deep enough into the foundation so that the effects of minor geological details on performance are minimized (Fig. 27). The relief wells can be designed & maintained in good working condition as per IS: 5050 – Code of practice for design, construction and maintenance of relief wells.

6.9.5 Upstream and downstream slopes of the dam

The upstream and downstream slopes of an earthfill / rockfill dam are mainly dependent upon stability against sliding of the construction materials. A rigorous analysis of the stability of the dam can be done in accordance with IS: 7894 – Code of practice for stability analysis of earth dams. Generally the upstream & downstream slopes in case of earthfill dams vary from 2H:IV to 3.5 H:IV. But in case of rockfill dams, there is an adequate margin of safety against sliding on an inclined face of the fill because of high interlock strength in the material and absence of pore pressures. As such the slopes of a rock fill dam can be as steep as 1.5 H:IV to 1.2 H:IV. Berms are provided on the downstream as well as upstream slopes for the following purposes:

(i) To break continuity of slopes thereby reducing surface erosion,
(ii) To provide level surfaces for construction and maintenance operations, and
(iii) To prevent undermining of riprap in case of upstream slopes

The berm should slope towards the inner edge to prevent rainwater from flowing down the slope of dam. This slope can be of the order of 1:50. A berm may be of 3 to 6 m width. The berms should be provided at a vertical spacing of 10 to 15 m. A berm is desirable at the top of the rock toe also.

6.9.6 Top width

The top width of the dam should be provided according to working space requirements. It can be from 2 to 6 m.

6.9.7 Foundation shear stresses

The safety of the foundation material against shear may be checked as below. With reference to Fig. 28, the total horizontal shear under a slope of the dam is given by

$$S = r\frac{h_1^2 - h_2^2}{2}\tan^2\left(45^o - \frac{\phi_1}{2}\right),$$

where,

r = effective average unit weight of the soil,
h_1 & h_2 = heights above hard stratum from top and toe of the dam, and
ϕ_1 = equivalent angle of internal friction, determined by the equation

$$rh_1 \tan \phi_1 = C + rh_1 \tan \phi,$$

where C and ϕ are the actual soil properties

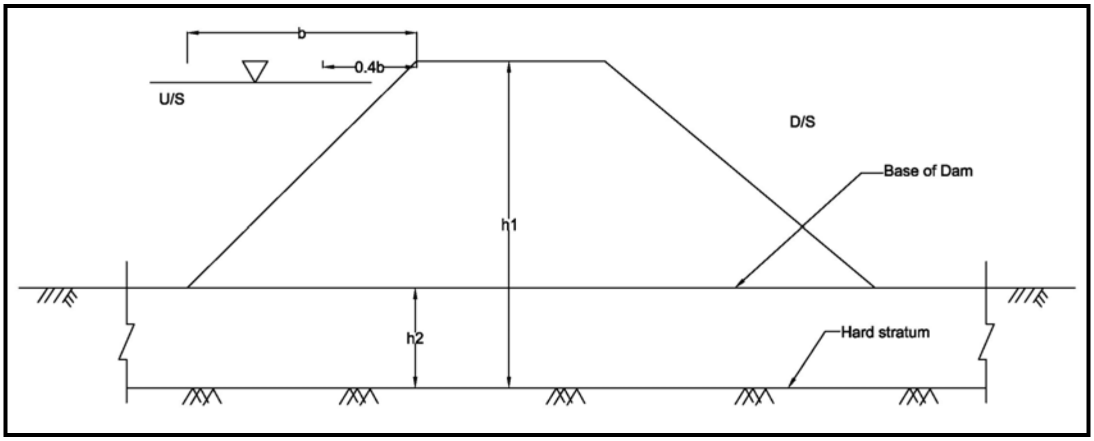


Fig. 28: Upstream and Downstream Slopes of the Dam

The average shear stress S_a is equal to S/b. It has been found that the maximum intensity of shear stress occurs at a distance of 0.4 b from top shoulder and is equal to about 1.4 times the average. Knowing the maximum intensity of shear stress and its location, the ratio of shear strength to shear stress at that location determines the factor of safety against foundation shear which should be at least 1.5.

6.9.8 Safety against free flow of water from upstream to downstream

Free flow may develop along a joint between earth and masonary of sluices or conduits or penstocks passing through the dam unless proper bond is ensured. It is, therefore, preferable to have no pressure conduits passing through an earthen section as any crack resulting escape of water would have serious consequences. The necessary outlets and penstocks may better be taken through tunnels in the hill side on either side of the dam. A continuous passage for free flow may be made by large burrowing animals. As such large burrowing animals like crocodiles should not be allowed to develop in the reservoir.

6.9.9 Safety against piping

Safety against piping is obtained by surrounding the drainage toe or horizontal or vertical drainage on all sides by graded filter so that while water can get through freely, no soil particles of the dam or foundation can enter the filter layer.

6.9.10 Upstream and downstream slope protection

The upstream and downstream slopes of an embankment dam shall be protected in accordance with IS:823 – Code of practice for protection of slope for reservoir embankment.

7.0 INTAKE WORKS

An intake should be able to perform following functions:

(i) It should be able to draw required amount of water at minimum reservoir level,
(ii) It should draw minimum amount of sediment
(iii) It should check entry of trash and debris
(iv) It should have hydraulically smooth passage so as to have minimum head loss

(v) In case the intake is directly connected to an under pressure water conductor system, the inlet should be so located as to eliminate vortex formation.

In order to attain the above functions, the following design details are necessary:

(i) The centre line of the intake and the size of inlet should be such that the intake could draw the required amount of water at minimum reservoir level without vortex formation and at a sufficiently low velocity.
(ii) The sill of the intake should be kept at the highest possible level in order to reduce silt entry upto minimum extent.
(iii) Looking from the river towards the bank, the intake, as far as possible should be located on the concave side so as to reduce silt content.
(iv) The inlet opening should be of bell mouth shape and transitions should be hydraulically streamlined.
(v) A trash rack structure should be provided at the entrance and the velocity through the trash racks should be limited to 0.75 m / sec., where manual cleaning of racks is provided and 1.5 m/sec, where mechanical cleaning is adopted.

7.1 Types of Intake

There may be many types of intakes depending upon the type of scheme, location, etc. The intakes can be broadly classified as Run-of-river type intakes, and Reservoir type intakes (Refer IS: 976):

7.2 Run-of-river Type Intakes

7.2.1 Intake just upstream of a raised weir / barrage

In such a case an under-sluice with a low crest is provided adjacent to the bank, where the intake is proposed. A divide wall upto the end of intake is provided by extending the pier of the river side end of the under-sluice, thus creating a deep pool. The intake is aligned at 90^o to 110^o to the weir / barrage axis with its sill as much above the crest of the under-sluice bays as possible. A typical arrangement of the intake is given in Fig. 29.

7.2.2 Intakes for canal power house or barrage power houses

An intake which is an integral part of the power house is located across large canals to utilize the head of a canal fall or across a river to utilize the head created by the barrage. In such power houses Kaplan turbines with concrete spiral casing or tubuler / bulb turbines are used for power generation. Typical layouts are shown in Figs. 30 & 31.

7.2.3 Forebay Intake

In some canal power projects, where the power house is planned at some distance from the intake location and head is appreciable, the free flow of the canal is terminated in a basin known as forebay and the intake for the penstocks is provided at the downstream end of the forebay. The forebay is a broadened portion of the canal which is utilized to absorb the surges created by the load variations of the power house. A typical layout of forebay intake is shown in Fig. 32.

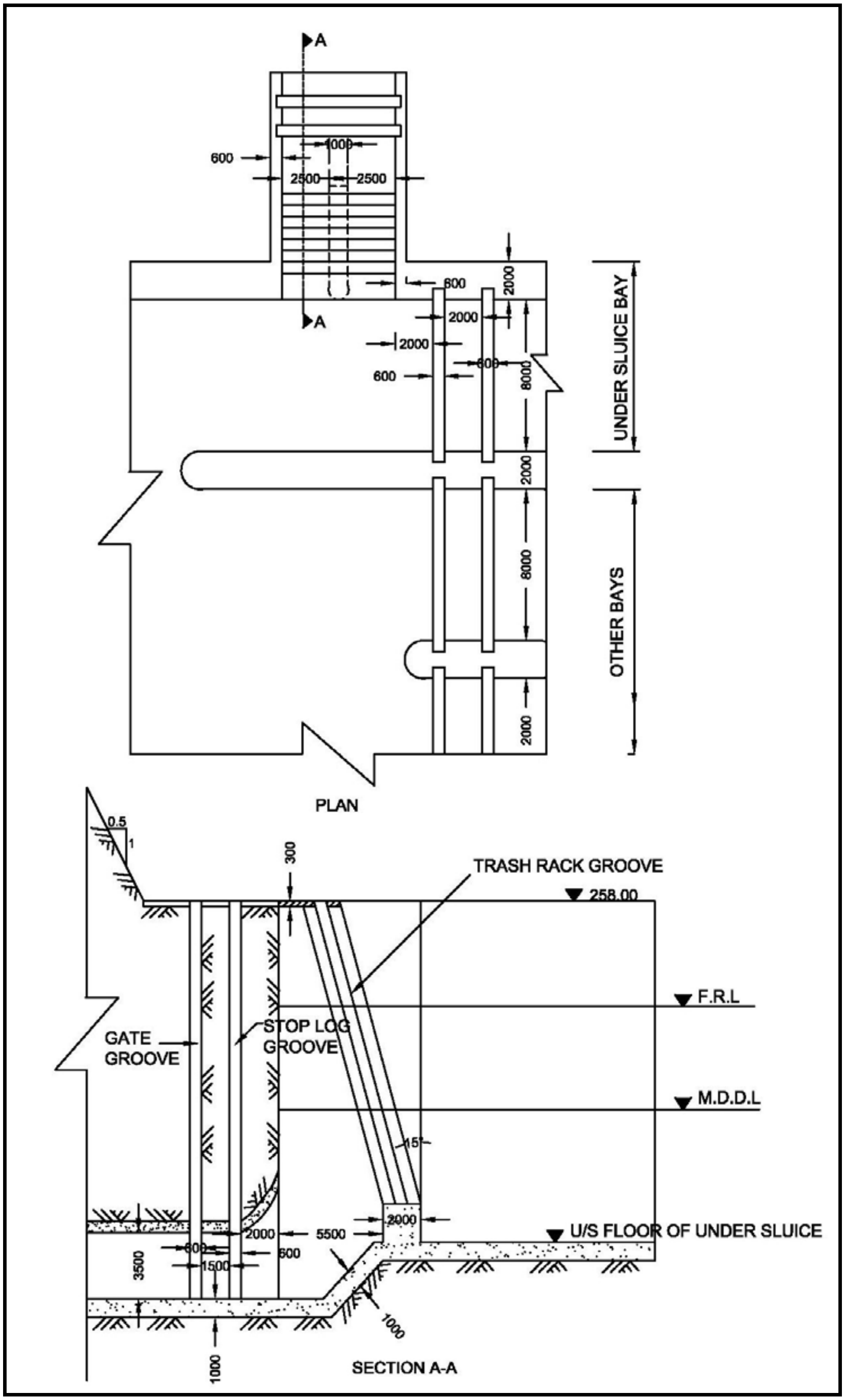


Fig. 29: Typical Intake Details Upstream of Raised Weir / Barrage

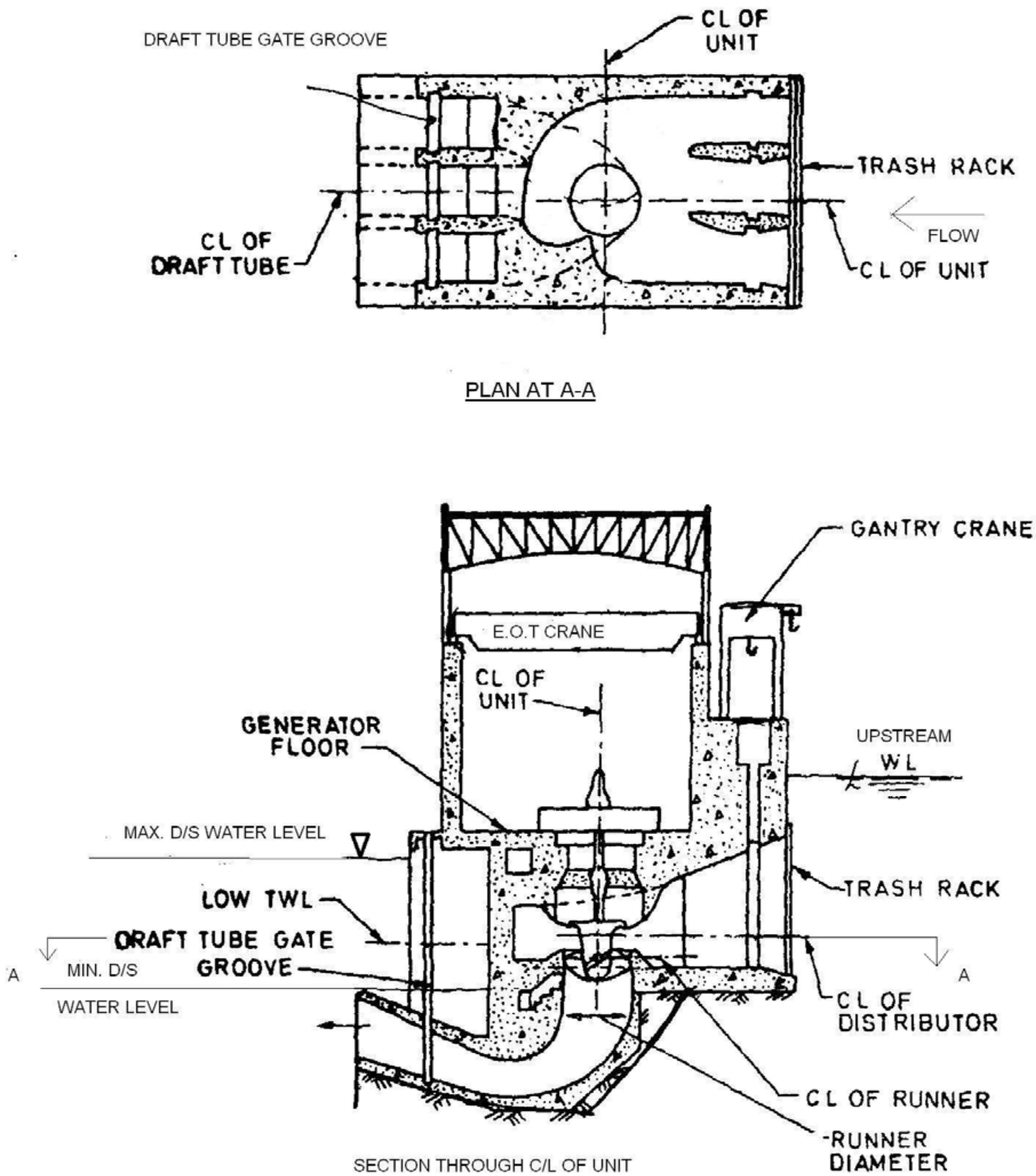


Fig. 30: Canal / River Power House Intakes (Kaplan Turbines)

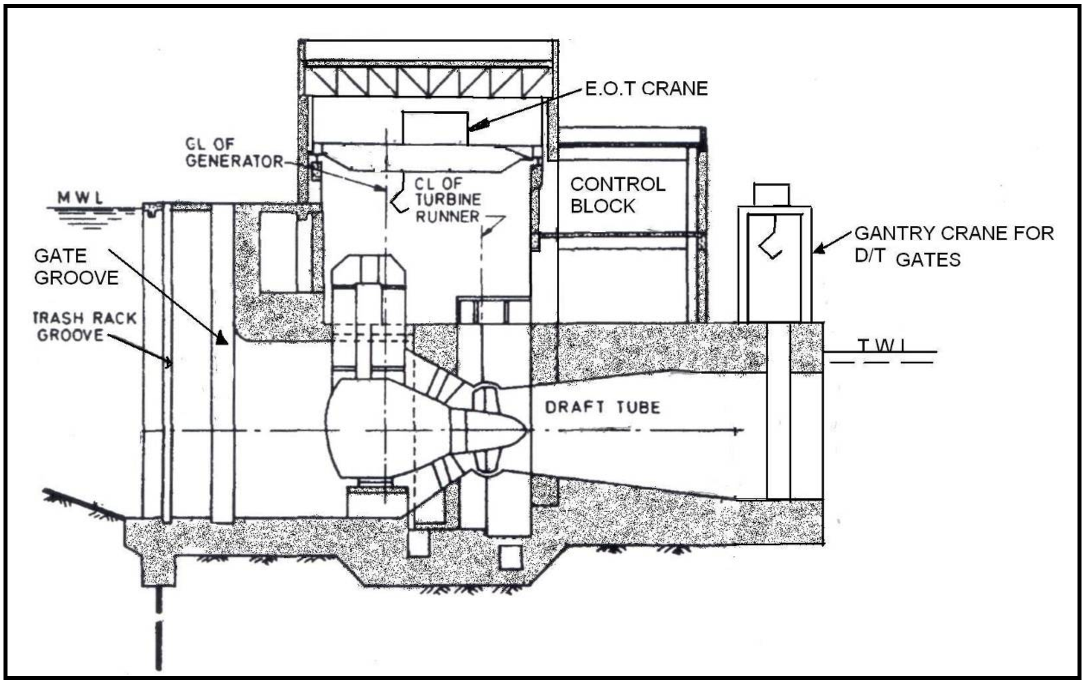


Fig. 31: Canal / River Power House Intakes (Bulb Turbines)

7.3 Reservoir Type Intakes

7.3.1 Intake in concrete or masonary dams

When the power house is located at the toe of a concrete or masonary dam and the penstock is embedded in the dam body, a semi-circular cage type intake is provided at upstream face of the dam. The layout of such intake is shown in Fig. 33.

7.3.2 Intake for earthen dam

In case of an earthen dam, normally the water conduit is not taken through the dam. In this case the pressure conduit takes off from a suitable place in the reservoir. A typical layout of such intake is shown in Fig. 34.

7.4 Hydraulic Design of Intake Components

7.4.1 Bell mouth opening and transitions

The area of the intake should be as follows:

$$\text{Area at inlet} = \frac{Area\ penstock}{C_c \times Cos\,\phi}$$

where,

C_c = Coefficient of contraction (C_c = 0.6 for high and medium heads & C_c = 0.7 for low heads)

ϕ = Angle of inclination of the centre line of penstock to horizontal

The shape of the intake entrance is shown in Fig. 35.

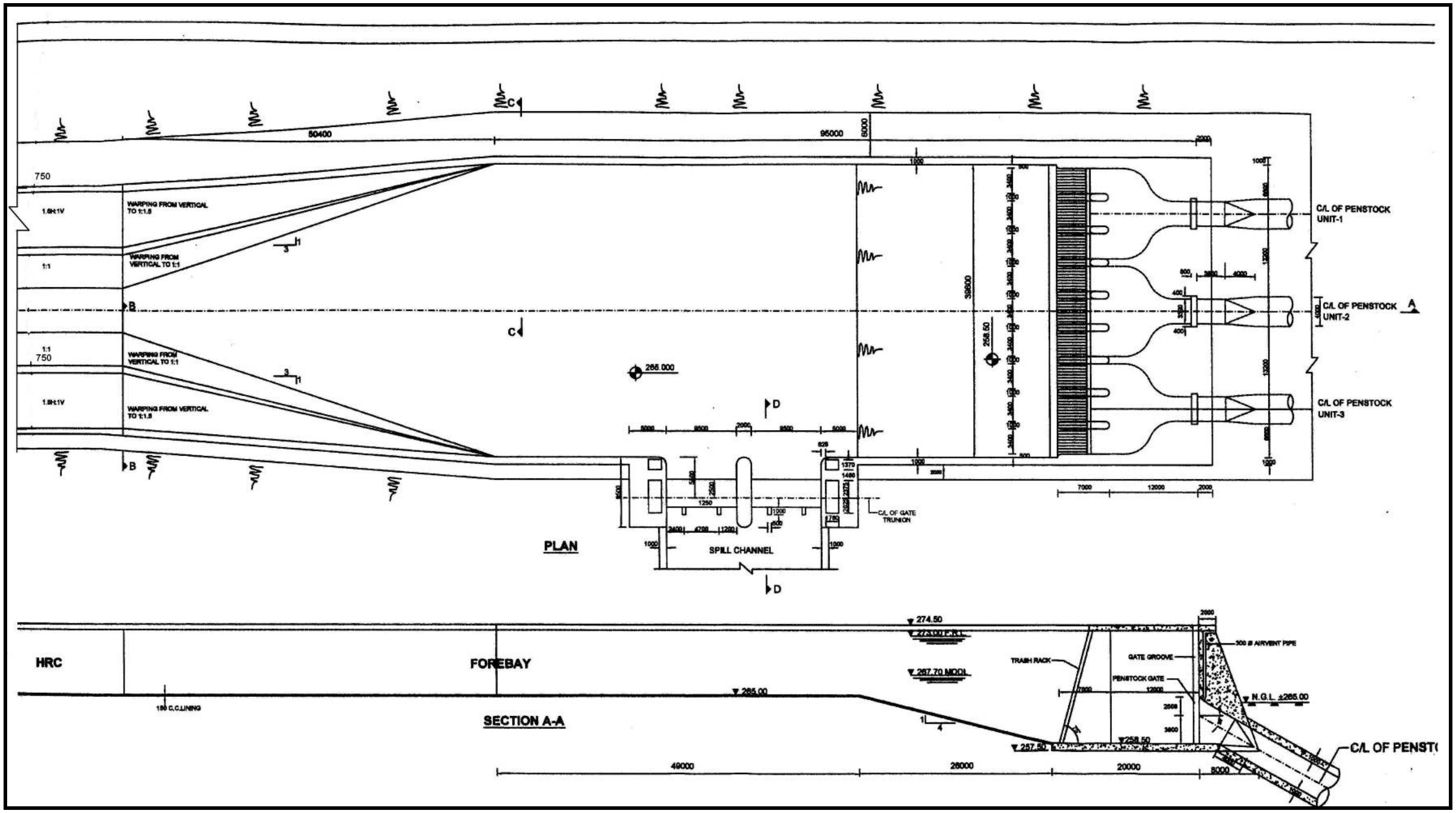


Fig. 32: Forebay Intake

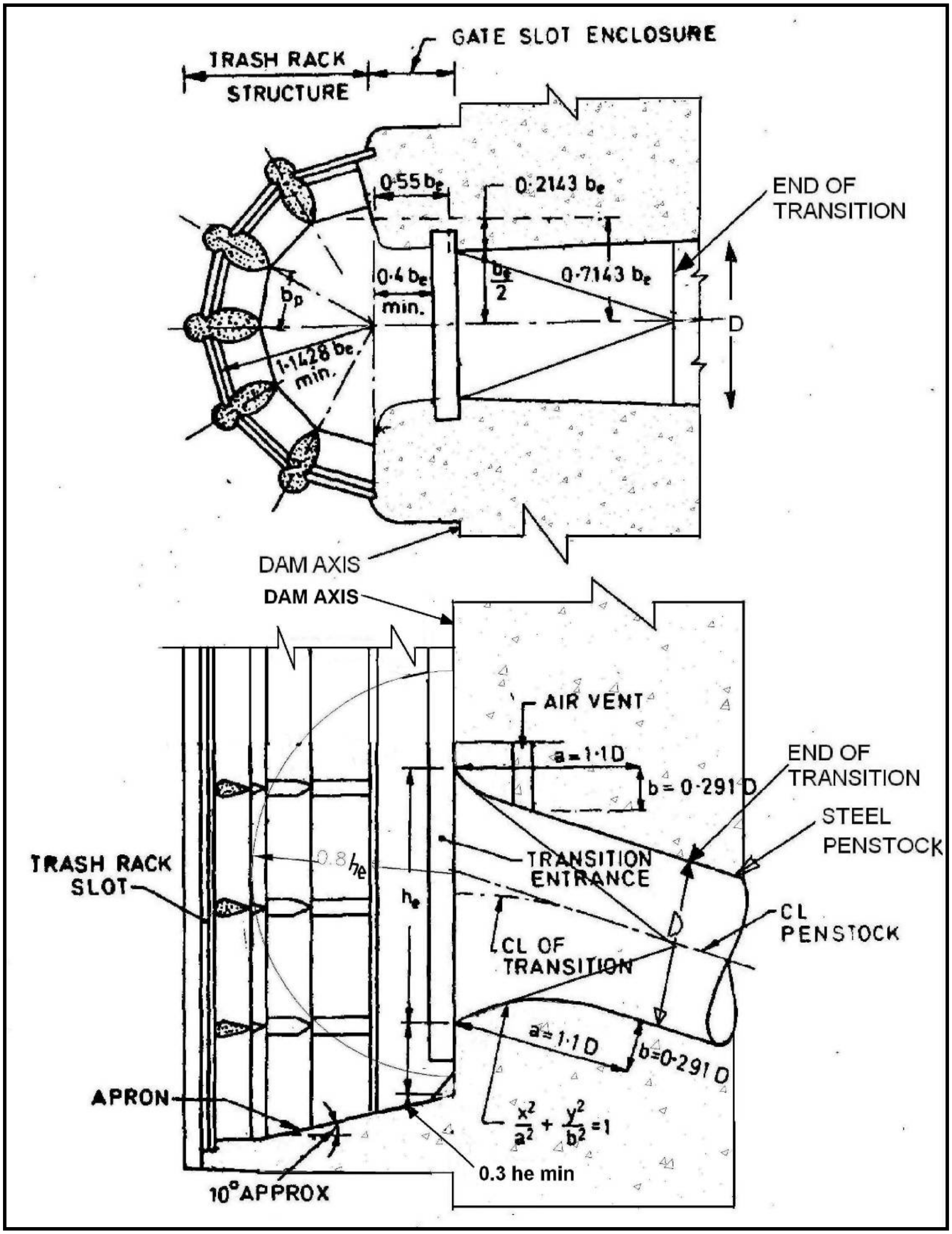


Fig. 33: Semi Circular Type Intake Structure

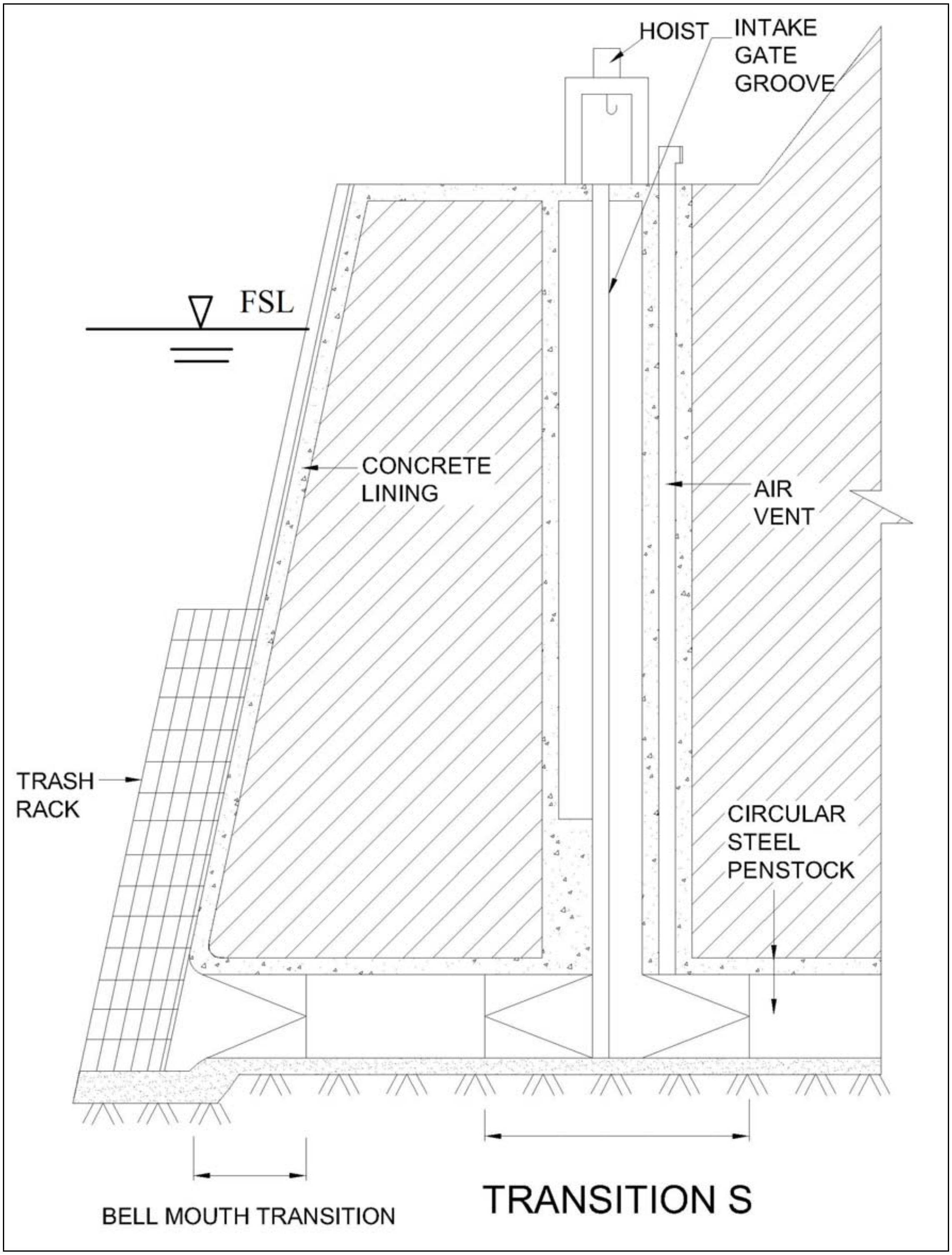


Fig. 34: Intake in Reservoir Independent of Dams

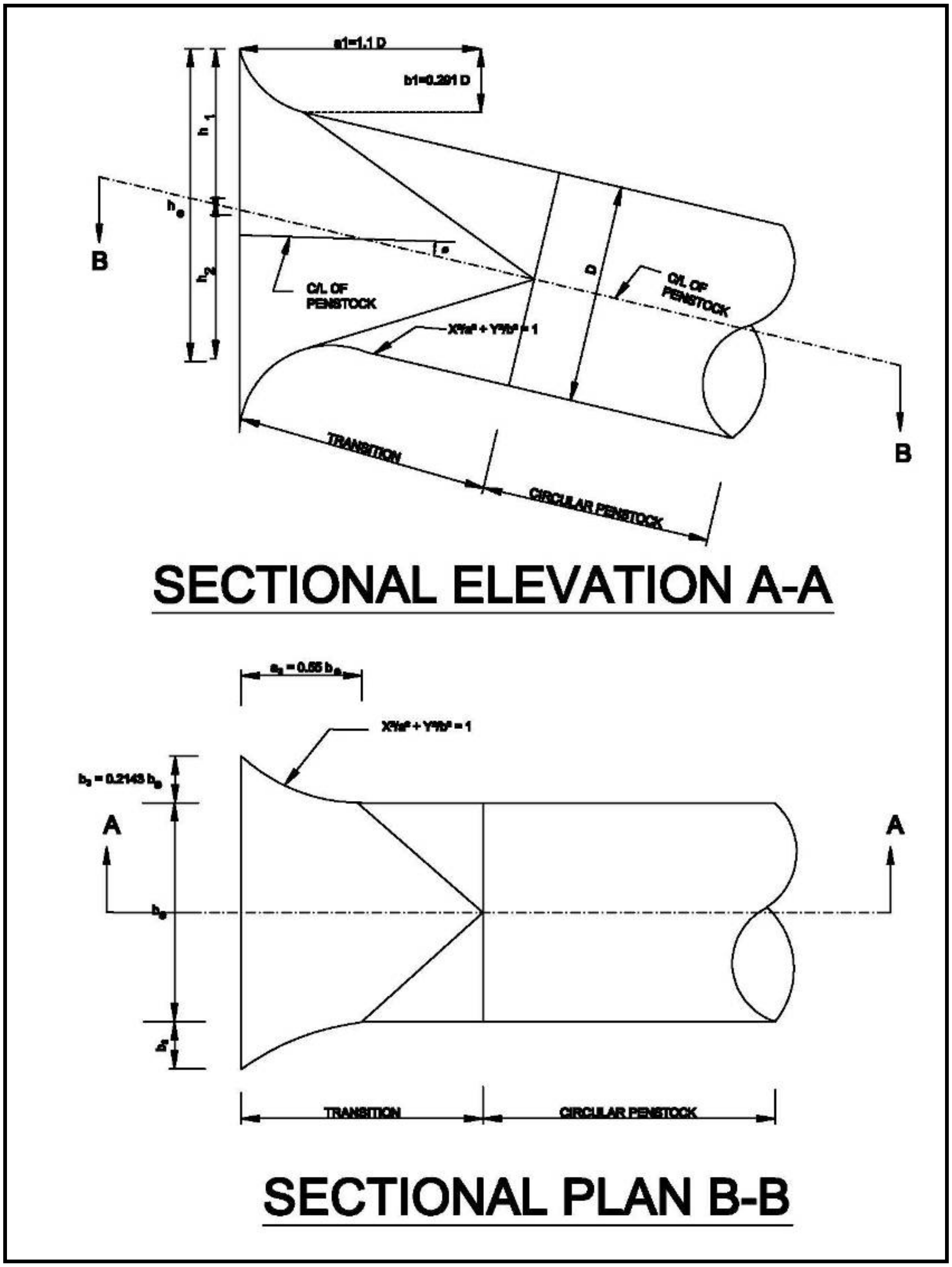


Fig. 35: Typical shape of intake opening

With reference to Fig. 35,

$$H_1 = D\left[(1.2\tan^2\phi + 0.0847)^{1/2} + \frac{1}{2\,Cos\,\phi} - 1.1\tan\phi\right],$$

$$h_2 = D\left[\frac{0.791}{Cos\,\phi} + 0.077\tan\phi\right],$$

$$h_e = h_1 + h_2,$$

$$b_e = \frac{Area\ of\ inlet\ opening}{h_e}$$

In the above relations, D = diameter of penstock.

The transition from rectangular opening to circular section should be designed according to following guidelines:

(i) Transitions should be made about the centre line of mass flow and should be gradual,
(ii) Side walls should expand at a maximum of 5^o from the centre line of mass flow,
(iii) All gate slots etc. should normally be beyond the transition zone.

7.5 Centre Line of Intake

Vorticity can appear in pressurized power intakes and should be avoided as it interferes with good performance of turbines. Lack of sufficient sub-mergence and asymmetrical approach are the most common causes of vortex formation. The minimum depth of submergence ‘h’ is defined as shown in Fig. 36.

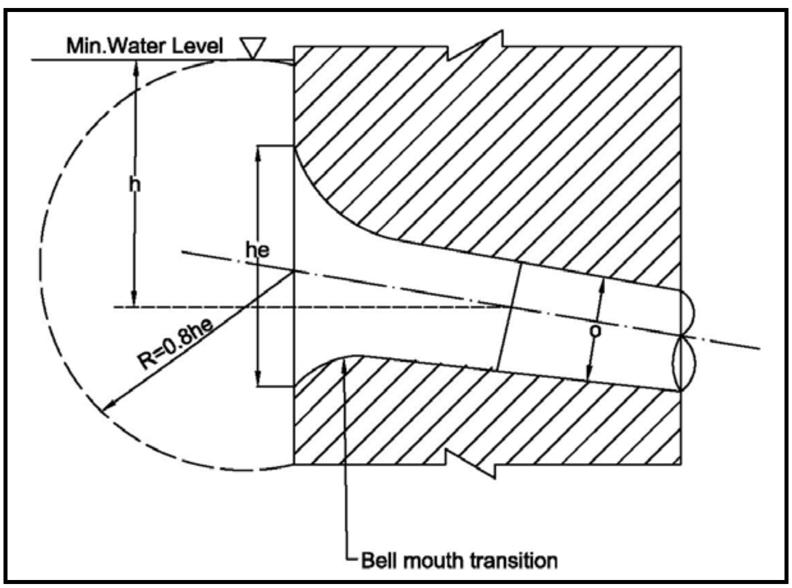


Fig. 36: Sub-mergence Requirement of Pressure Conduits

Minimum value of h should be as below:

$$h \geq \frac{D}{2} + C.V.\sqrt{D},$$

where,

D = diameter of conduit beyond transitions,
V = velocity corresponding 'D',
C = 0.7245 for asymmetric approach conditions and
= 0.5434 for symmetric approach conditions

In addition to above, the level diffence between minimum water level and the centre of inlet opening should not be less than 0.8 h_e, where h_e is the height of inlet opening. Besides a minimum submergence, anti-vortex measures like parallel fins at the top of intake opening, floating grating, perforated breast wall above intake opening may help to protect from vortex formation. (Refer to clause 5.2.3 of IS: 9761-1995 – "Hydropower Intakes – Criteria for Hydraulic Design.)

8.0 WATER CONDUCTOR SYSTEM

The water conductor system from intake upto surge tank / forbeay may comprise the following type of works:

(i) Earthen channel
(ii) Lined channel
(iii) RCC channel
(iv) Stone masonary channel
(v) Tunnel-concrete lined or unlined
(vi) Pipes – steel, RCC, HDPE, GRP
(vii) Cross-drainage works

Before deciding the type of conveyance system, the layout of the water conductor system should be marked on the contour plan of 1:1000 scale. This layout will include the locations of intake works, desilting device, if any, and forebay / surge tank. The choice for adopting any type of conveyance system shall depend upon the following:

1. Availability of construction material
2. Ease of construction
3. Economical considerations – Apart from total cost of the system, consideration should also be kept for the construction time.
4. Geology and topography of the terrain along the proposed alignment.

Depending upon the above considerations, the water conductor system may comprise one or more types of above mentioned works.

8.1 Earthen Channel

In case of small hydropower projects, earthen channels are normally used only on canal related projects where power projects are constructed to utilize canal falls for power generation. In such cases the diversion system of the canal, where the power house is constructed should be designed on the same lines as the existing canal. In this case IS:7112 – Criteria for design of cross section for unlined canals in Alluvial soils can be refereed to.

In hilly terrains the structures of water conductor system are generally located on hill slope and slope instability due to construction of canal is a major problem in small hydro projects. Generally the main causes of slope failure are:

(i) Water leakage from canals or forebay or desilting tank (Fig. 37),
(ii) Infiltration of surface run off
(iii) Erosion due to surface torrents
(iv) Where the bedding planes of rocks happen to be parallel to surface
(v) Surfacial jointing and weathering of rock slopes, etc.

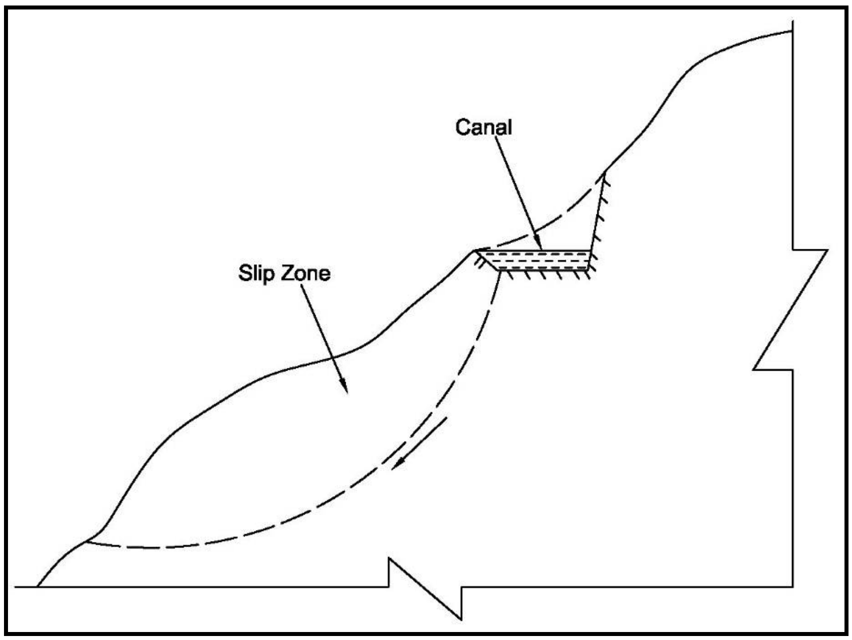


Fig. 37: Typical slip failure due to SHP construction

Generally most slope failures take place during or after rainstorms or prolonged rains.

Small hydro projects can not afford detailed geological investigation and as such great attention is required to be paid to field inspections and either the design of the work should be modified or the location of the works shifted from sites which appear to be unstable. Certain engineering measures as mentioned below can be taken to improve upon potential unsafe locations:

(a) Checking and diverting surface run off from entering potential sliding area
(b) Sealing or blanketing the infiltration area in the potential sliding zone by clay blanket.
(c) Lowering the ground water table in slide prone areas by a network of drains as shown in Fig. 38 or drainage wells as shown in Fig. 39.
(d) Removal of upper part of potential slide to reduce sliding force
(e) Putting retaining wall at a safe distance from slope foot to avoid foot cutting.

(f) Slope protection by stone pitching
(g) Rock bolting of unstable rock slopes

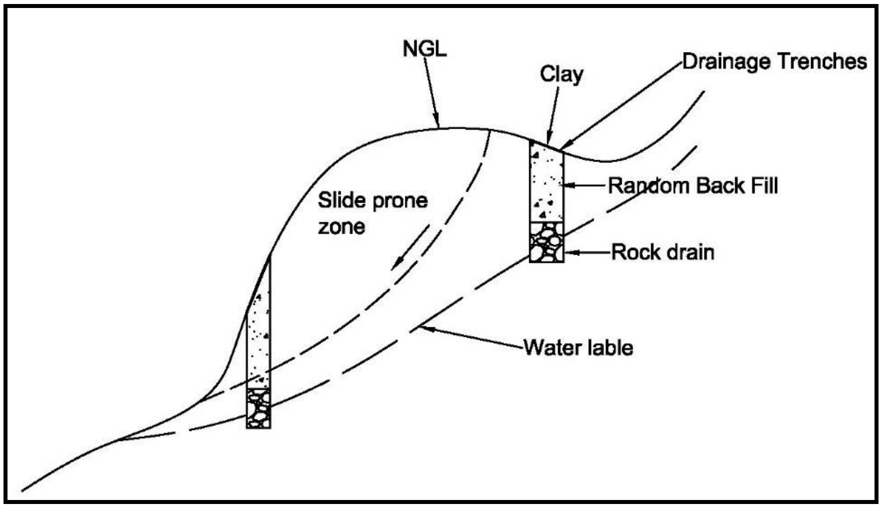


Fig. 38: Arrangement of Lowering Water Table in Slide Prone Zone

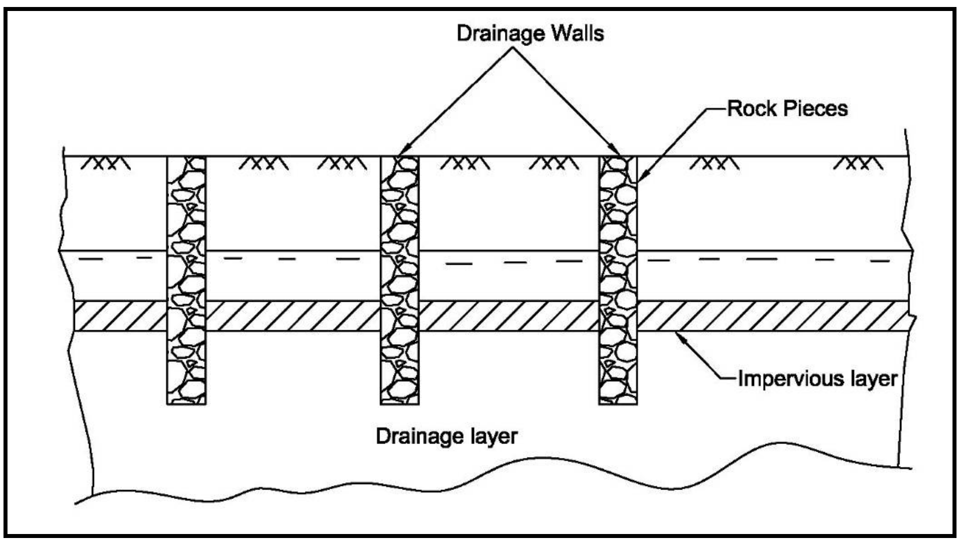


Fig. 39: Arrangement of Lowering Water Table

8.1.1 In hilly areas the canals are aligned along the contours giving required longitudinal slope. Typical cross-sections of unlined canals are shown in Fig. 40 (a) and Fig. 40 (b). The canal is designed for the maximum discharge of the plant, but it will be operated under much smaller discharges in lean season, thus resulting in serious silting. As such a cross-location designed for maximum discharge should have a reasonably high velocity to prevent silting during running with smaller discharge.

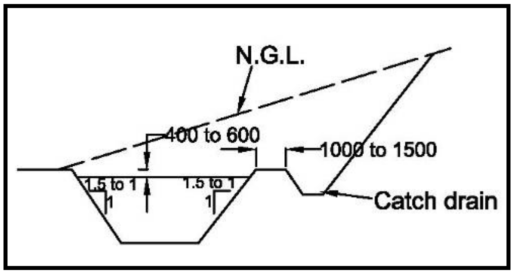


Fig. 40 (a): Canal section in cutting

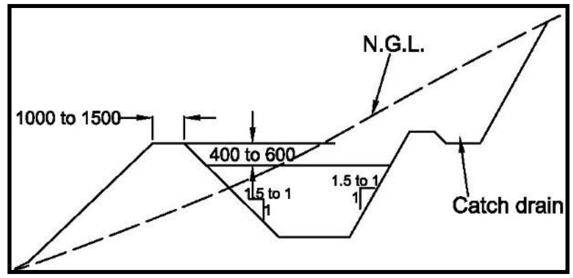


Fig. 40 (b): Canal section partly in filling and partly in cutting

A comparatively narrow canal should be given preference to a wide one because shallow depth in a canal has smaller hydraulic mean depth, thus resulting more deposition of silt. In case the canal passes through ice zones, a narrow and deep canal is preferable and the velocity should not be lesser than 0.9 to 1.2 m/sec.

8.2 Lined Channel

The aims of canal lining are:

(i). To prevent leakage from the canal,
(ii). To properly raise the mean velocity, which will not only reduce the area of cross-section of the canal but also be beneficial against silting and plant growth,
(iii). The reduce possibility of land slides due to leakage,
(iv). To reduce friction loss,
(v). To reduce maintenance cost

The types of canal lining in small hydro power projects may be as follows:

(i). Lining with gravel laid with clay morter in a thickness of about 150 mm.
(ii). Blanketing with hydrated lime-clay mixture in a proportion of 1:4 to 1:7 by weight and of thickness 150 to 200 mm. It should be cured for atleast 14 days with additional spraying and recompacted several times. This type of lining can withstand a velocity of several meters per second after 28 days. It is not suitable for very cold regions.
(iii). Blanketing with 150 to 200 mm thick cement – soil mixture in a proportion of 1:7 to 1:10 by weight & well compacted.
(iv). Stone-pitched lining 150 to 200 mm thick laid over a bed of 12 mm thick 1:2 lime- sand mortar or 1:3 cement – sand mortar.
(v). 200 thick lining with burnt bricks and 1:4 cement sand mortar.
(vi). 100 to 150 mm thick M-15 or M-20 grade cement concrete lining placed in situ.

A particular type of lining is selected on the basis of the following considerations:

(i) Type of sub-grade
(ii) Position of water table
(iii) Climatic conditions
(iv) Availability of materials
(v) Speed of construction and time schedule
(vi) Performance of lining in existing projects in adjoining areas.

In case the channel is aligned through rock, it may be lined with 100 to 150 mm thick M-15 grade cement concrete. In the side, the concrete may be supported with 16 to 20 mm diameter rock bolts at 1500 to 2500 centre to centre both ways. If required weep holes can be left in the concrete to release back side drainage.

8.2.1 Side slopes

Inner side slopes of lined channels should be such that no earth pressure or any other external pressure is exerted over the back of the lining. Sudden drawdown of water level in the lined channel should be controlled by strict operation rules and regulations. In addition, suitable measures like adequate drainage should be provided before the commencement of lining. In general slopes may be adopted as per Table 6.

Table 6: Slope for Different Types Soil

S. No.	Type of Soil	Side Slope (Horizontal : Vertical)
1.	Sandy loam / sandy gravel / murum	1.5:1 to 2:1
2.	Clay soils	1.5:1 to 2:1 in cutting and 1.5:1 to 2.5:1 in embankment
3.	Rock	0.25:1 to 0.5:1

8.2.2 Free board

The minimum free board above full supply level may be provided as per Table 7.

Table 7: Free Board for Different Discharge Capacity

Channel Discharge (cumes)	Free Board (mm)
> 10	600
3 to 10	500
1 to 3	400
< 1	200 to 300

8.2.3 Bank top width

The top width of the banks may be between 1.0 m to 3.0 m as per requirement and site conditions.

8.2.4 Design of the section

The channel section can be designed with the help of Manning’s equation given is para 5.1.

The limiting velocities may vary from 1.5 to 2.75 m/sec. the recommended values for Manning Rugosity coefficient of lining may be taken as per Table 8.

8.2.5 Reference codes

Following Bureau of Indian Standards can be referred for design of canal lining:

(i) IS:10430 – criteria for design of lined canals and guidance for selection of type of lining

(ii) IS:10646 – Canal linings – cement concrete tiles – specifications
(iii) IS:11809 – Lining of canals by stone masonary – code of practice
(iv) IS:4558 – Under drainage of lined canals code of practice

Table 8: Recommended Values of Rugosity Coefficient for Rigid Boundaries

S. No.	Surface	Recommended Value of 'n'
1.	Earthen Channel	0.0225 to 0.0300
2.	Cement Plaster	0.011
3.	Cast Iron	0.013
4.	Steel	0.011
5.	Concrete	0.013 to 0.014
6.	Drainage Tiles	0.012 to 0.017
7.	Brick Work	0.013
8.	Rubble Masonary	0.017 to 0.025
9.	Rock Cuts	0.035 to 0.040

8.3 RCC Channels

RCC channels are constructed as power channels in the following cases:

(i) When the hill slopes happen to be too steep
(ii) When due to freezing temperatures, it becomes necessary to cover the channel
(iii) When due to geological or topographic reasons it becomes necessary to provide a cut and cover section of the power channel. Such a situation normally arises in the upstream reaches of the channel taking off from the trench weir.

The RCC channel may be both covered or open. In both cases contraction joints with 225 to 300 wide water stops should be provided at a spacing of about 30 metres centre to centre. In case the thickness of the concrete happens to be less than 150 mm, the walls, bottom and top slabs should be thickened to atleast 150 mm upto 300 mm length on both sides of the contraction joints to accommodate the water stop as shown in Fig. 41.

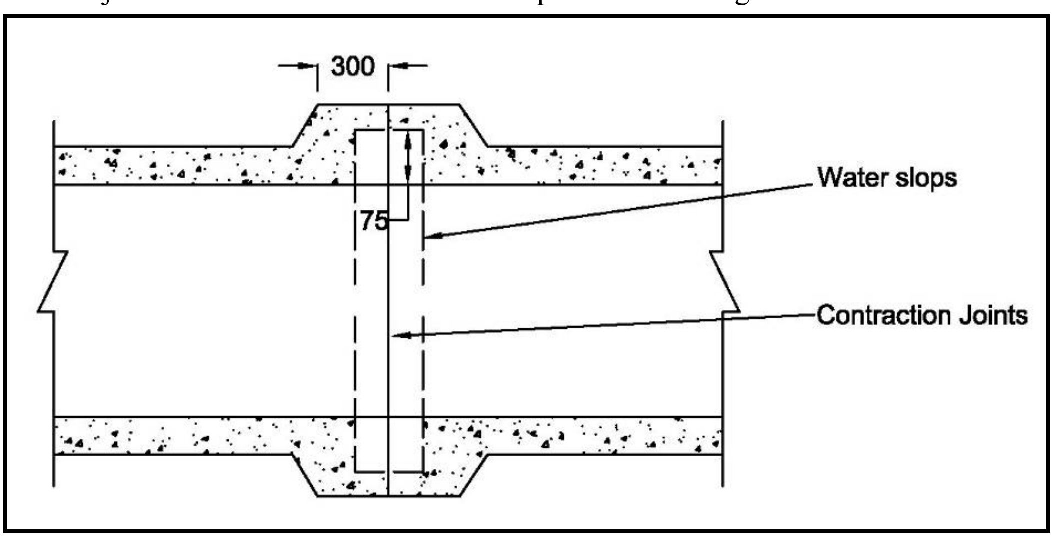


Fig. 41: Section through RCC Channel

In case of covered channels atleast 800 m x 800 m size manholes should be provided at 60 to 100 metres centre to centre for maintenance purposes. In case of RCC channel, a maximum velocity of 2 m / sec. is permissible. However, more velocity means more head loss but small size of channel section. A free board of 300 to 500 mm depending upon the size of channel is normally adequate – Typical sections of open and cut and cover channel sections are shown in Figs. 42 and 43.

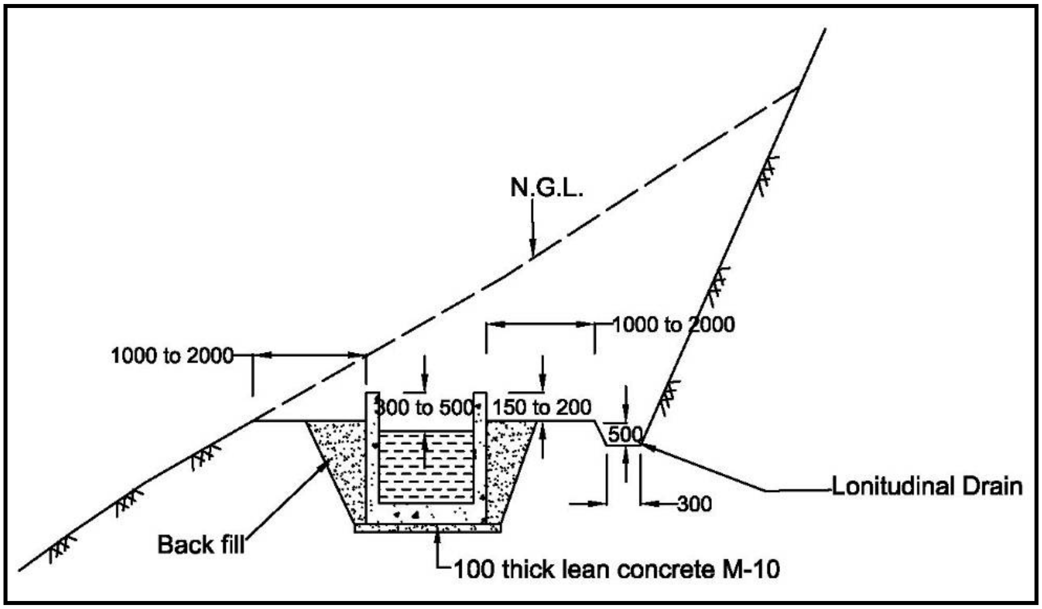


Fig. 42: Typical Section of Open RCC Channel

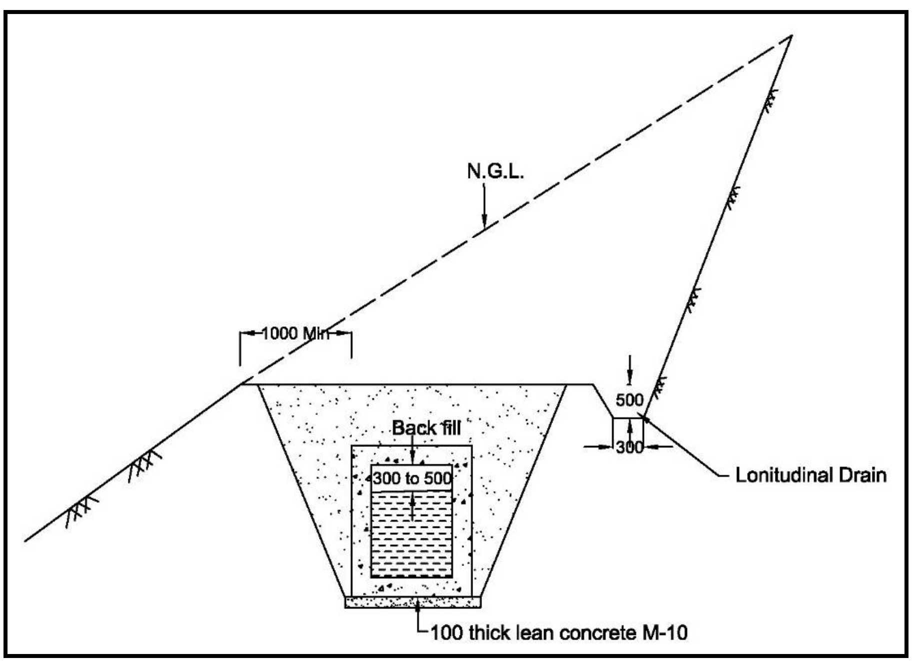


Fig. 43: Typical Section of Cut & Cover RCC Channel

8.3.1 Design

While the open channel section is designed as a trough, the cut and cover section is designed as box section. The channel section should be designed considering the most unfavourable combination of the following loads and forces:

(a) Load due to backfill including surcharge.
(b) Internal water pressure
(c) External water pressure, if any
(d) Live load
(e) Seismic forces

The longitudinal slope of the channel way be decided on the basis of maximum allowable velocity or considering the head loss. The size of the section can be determined by Manning's equation, in which the rugosity coefficient of concrete surface can be taken as 0.018 and the ratio of the width & water depth of channel may vary from 1:1 to 1.5:1.

8.4 Masonary Channel

The power channels can be lined with RR masonary instead of RCC on the basis of:

(i) Economic considerations
(ii) Availability of unskilled labour.

The minimum thickness of RR masonary lining should be 450 mm, it should be carried out in 1:4 cement sand mortar and the inner and top surface should be plastered with 20 mm thick 1:4 cement sand mortar. The inner and top surfaces can alternatively be provided with 75 mm thick RCC in M-15 grade concrete alongwith reinforcement camprising 6 mm ϕ bars @300 c/c at the centre of the concrete. A typical section of masonary channel is shown in Fig. 44.

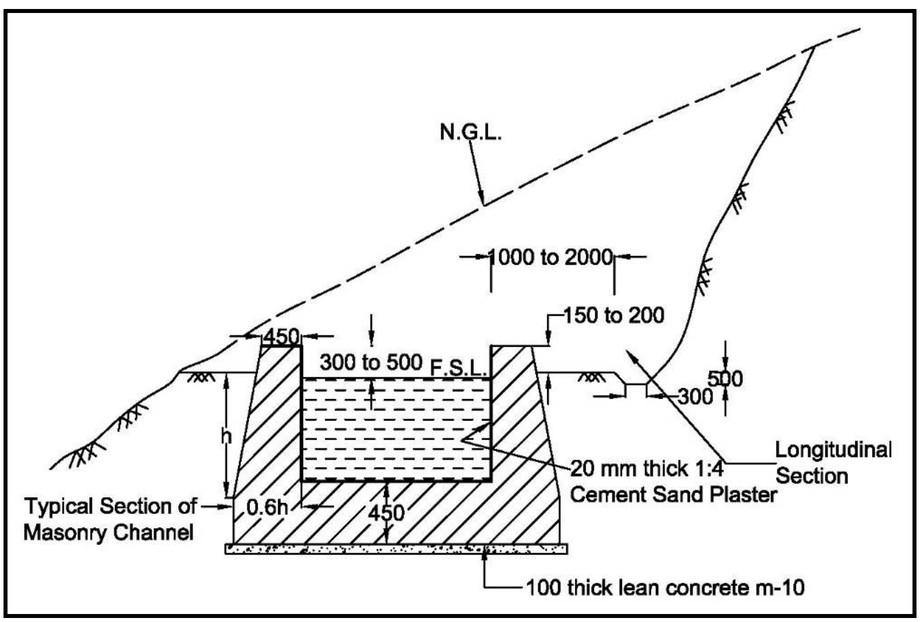


Fig. 44: Typical Section of Masonary Channel

8.4.1 Design

As far as sizing of channel area of the masonry channel is concerned, it will be carried out similar to that of RCC channel. The thickness of the masonary wall can be as shown in Fig. 44.

8.5 Tunnels

Tunnels are introduced in the water conductor system under the following situations:

(i) When the side slopes of the hill are too weak to support a canal
(ii) When side slopes are too steep to construct a canal
(iii) When the river takes a sharp loop and the straight route taken by the tunnel is much shorter than the long route of the canal along the contour.
(iv) When the rock quality and the dip of the rock foliations is favourable for tunneling.

The adoption of the option of tunneling has many inherent advantages as mentioned below:

(i) The natural beauty of the project area along the tunnel route remains almost unaltered.
(ii) The tunnels remain unaffected by land slides and rock falls.
(iii) The tunnels take almost the shortest route and as such, the head loss is lesser than the surface water conductor system which is constructed along the contour.
(iv) The tunnels do not need any cross-drainage works
(v) The maintenance cost of tunnels is almost negligible in comparison of surface channels

8.5.1 Sizing of tunnel

Average permissible velocity in a concrete lined pressure tunnel may be about 4 to 5 metres per second. However, if economic size of the tunnel is determined by adding the operating charges including operation and maintenance charges, interest on capital, ammortisation and revenue loss due to head loss in friction, the economic velocity in the tunnel is likely to be of the order of 2.5 to 3.5 metre per second. Since the carrying out of such are analysis is a tedius process specially at the design stage, when the rates of the various items of work such as underground excavation, shotcreting, concreting, rockbolting, rib fixing, grouting etc. and the geology along the tunnel alignment are uncertain. For small hydro project, the section of the tunnel can be decided by adopting an average velocity of 3 to 3.5 metre per sec.

8.5.2 Tunnel shape

The following shapes (Fig. 45) are generally used for tunnel sections

(a) Circular section
(b) D-section
(c) Horse show section
(d) Modified horse shoe section
(e) Modified horse shoe section for small size tunnels

8.5.2.1 Circular section

The circular section is most suitable from structural consideration. However, it is difficult to excavate, particularly when the cross-sectional area is small.

8.5.2.2 D-section

D-section is suitable for tunnels located in very good quality rock where the external pressures due to water or rock upon the lining are small and lining is not required to be designed from internal pressure. It is most suitable for free flow tunnels. The only advantage of this section is flat invert resulting more working space during construction.

8.5.2.3 Horse shoe and modified horse shoe sections

These sections are a compromise between circular and D-sections. These sections are quite strong against external pressure.

8.5.2.4 Modified horse-shoe section for small size tunnels

The minimum size of tunnel, which can be excavated without much difficulty is, 2.0 m(W) x 2.5 m (H). The section shown in Fig. 45 can be used for small size tunnels which are likely to be subjected to external pressure.

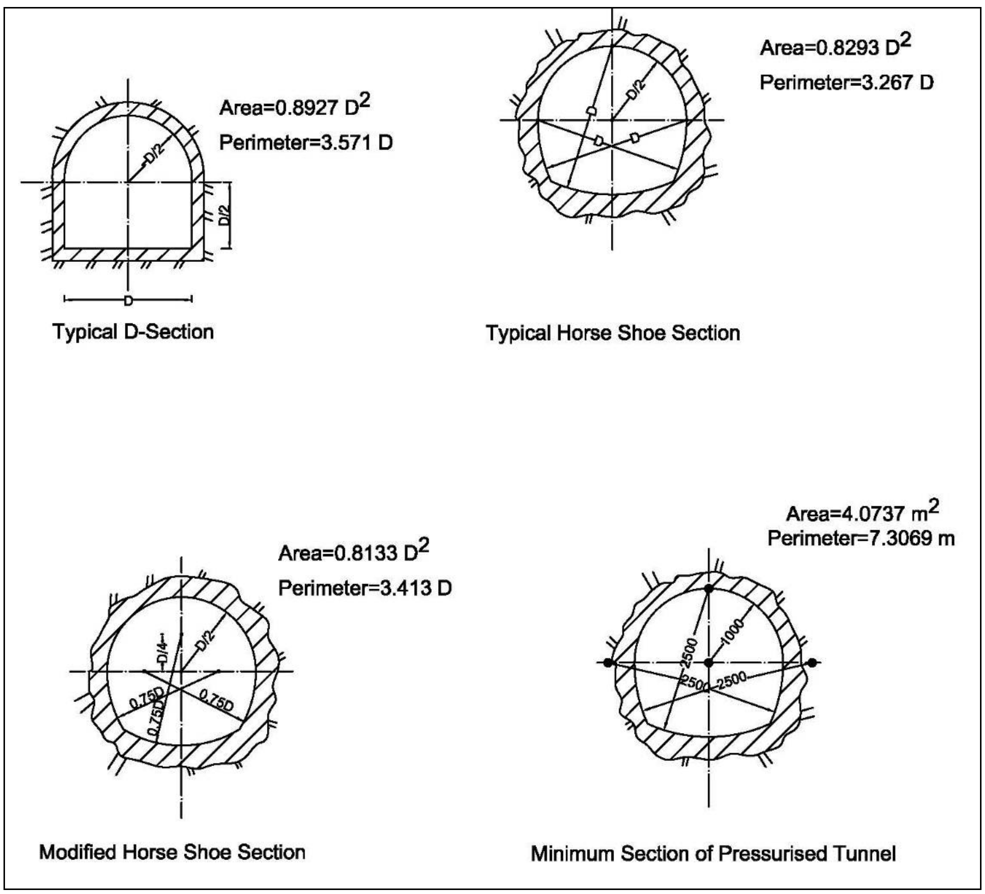


Fig. 45: Typical Sections of Power Tunnel

8.5.3 Hydraulic Design

8.5.3.1 Obligatory levels of tunnel

In case of a pressure tunnel the depth of intake shall be such that no air is sucked in under any condition. The location of outlet of a tunnel shall be such that the entry of air would not adversely affect tunnel operation and safety provided that sufficient precautions for preventing air locks are taken. The tunnels should preferably have positive gradient in the direction of flow, since they may have to be emptied and drainaed from time to time for the purpose of inspection and maintenance.

8.5.3.2 Cavitation

Design shall be such that negative pressures are avoided. To make sure that cavitation is avoided and to allow for uncertainties, the residual positive pressure shall not be less than 3 m of water head in concrete lined tunnels.

8.5.3.3 Transition shapes

From the tunnel section, either entry into or exit from the tunnel requires transition to reduce the head losses to a minimum and to avoid cavitation. The recommended shapes for entrance, contraction or expansion and exit transitions for pressure tunnels are given below. However, for partly flowing tunnels the methods of design shall be the same as for open transition.

(a) Entrance – To minimize head losses and to avoid zones where cavitation pressures may develop, the entrance to a pressure tunnel shall be streamlined to provide gradual and smooth changes in flow. To obtain best inlet efficiency the shape of entrance should simulate that of a jet discharging into air and should guide and support the jet with minimum interference until it is contracted to the tunnel dimensions. If the entrance curve is too sharp or too short, subatmospheric pressure areas which may induce cavitation, will develop. A bellmounth entrance which conforms to or slightly encroaches upon free jet profile will provide the best entrance shape.

(i) For a circular tunnel the bellmouth shape may be approximated by an elliptical entrance curve represented by the following equation:

$$\frac{x^2}{(0.5D)^2} + \frac{y^2}{(0.15D)^2} = 1,$$

where x and y are coordinates and D is the diameter of the tunnel at the end of entrance transition. The x-axis of the elliptical entrance is parallel to and a distance of 0.65 D from the tunnel centre line; y-axis is normal to the tunnel centre line and 0.5 D downstream from the entrance face.

(ii) The jet issuing from a square or rectangular opening is not as easily defined as one issuing from a circular opening; the top and bottom curves may differ from the side curves both in length and curvature. Consequently, it is more difficult to determine a transition which will eliminate subatmospheric pressures. An elliptical curved entrance which will tend to minimize the negative pressure effects may be defined by the following equation:

$$\frac{x^2}{D^2} + \frac{y^2}{(0.33\,D)^2} = 1,$$

where D is the vertical height of the tunnel for defining the top and bottom curves, and also is the horizontal width of the tunnel for defining the side curves. The major and minor axes are positioned similar to those indicated for the circular bellmouth.

(iii) For rectangular entrance with the bottom placed even with the upstream floor and with curved guide piers at each side of entrance openings, both the bottom and side contractions will be suppressed and a sharper contraction shall take place at the top of the opening. For this condition the top contraction curve may be defined by the following equation:

$$\frac{x^2}{D^2} + \frac{y^2}{(0.67\,D)^2} = 1,$$

where D is the vertical height of the tunnel downstream from the entrance.

(b) Contraction and Expansion – To minimize head losses and to avoid cavitation tendencies along the tunnel surfaces, contraction and expansion transitions to and from gate control sections in a tunnel should be gradual.

(i) For contractions, the maximum covergent angle should not exceed that indicated by the relationship:

$$\tan\alpha = \frac{1}{U},$$

where

α = angle of the tunnel wall surfaces with respect to its centre line,

U = arbitrary parameter $\frac{v}{\sqrt{gD}}$,

v and D = average of the velocities and diameters at the beginning and end of the transition, and

g = acceleration due to gravity

(ii) Expansion should be more gradual than contraction because of the danger of cavitation where sharp changes in the side walls occur. Furthermore, head loss coefficients for expansions increase rapidly after the angle α exceeds about 10^{o}. Expansion should be based on the following relationship:

$$\tan\alpha = \frac{1}{2U}$$

The notations are the same as for equation given in (i) above. For pressure tunnels, the angle α may not normally exceed 10^{o}.

(c) Exit – When a circular tunnel flowing partly full empties into a chute, the transition from the circular section to one with a flat bottom may be made in the open channel downstream from the tunnel portal, or it may be made within the tunnel so that the bottom will be flat at the portal section. Ordinarily, the transition should be made by gradually decreasing the circular quadrants from full radius at the upstream end of the transition to zero at the downstream end. For usual installations the length of the transition can be related to the exit velocity. An empirical rule which may be used to design a satisfactory transition for velocities upto 6 m/s is as follows:

$$L = \frac{2vD}{3},$$

where

L = length of transition in m,
v = exit velocity in m/s, and
D = tunnel diameter in m.

8.5.3.4 Pressure flow losses

(a) Friction Losses – Friction factors for estimating the friction losses shall be based on actual field observations. For tunnels flowing full, friction loss may be computed by the use of Manning's formula.

For concrete lined tunnels the value of rugosity coefficient 'n' varies from 0.012 to 0.018.

The value of rugosity coefficient 'n' for use in the Manning's formula for an unlined tunnel depends on the nature of the rock and the quality of trimming, and is possibly influenced by the amount and distribution of overbreak. Recommended. Values of 'n' for various rock surface conditions are given in Table 9.

Table 9: Manning Values for Different Tunnel Surface

Surface Characteristics	Manning 'n'	
	Min	Max
Very rough	0.04	0.06
Surface trimmed	0.025	0.035
Surface trimmed and invert concreted	0.020	0.030

(b) Trash Rack Losses – Trash rack structure which consists of widely spaced structural members without rack bars will cause very little head loss and trash rack losses in such a case may be neglected in computing tunnel losses. When the trash rack consists of a rack of bars, the loss will depend on bar thickness, depth and spacing and shall be obtained from the following formula:

$$h_t = K_t \frac{v^2}{2g}$$

Where

h_t = trash rack head loss,
K_t = loss coefficient for trash rack

$$= 1.45 - 0.45\frac{a_n}{a_t} - \left[\frac{a_n}{a_t}\right]^2,$$

a_n = net area through trash rack bars,
a_t = gross area of the vent (racks and supports),
v = velocity in net area, and
g = acceleration due to gravity

Where maximum loss values are desired, 50% of the rack area shall be considered clogged. This will result in twice the velocity through the trash rack. For minimum trash rack losses, the openings may not be considered clogged when computing the loss coefficient or the loss may be neglected entirely.

(c) Entrance Losses – Entrance loss shall be computed by the following equation:

$$h_e = K_e \frac{v^2}{2g},$$

where

h_e = head loss at entrance,
K_e = loss coefficient for trash rack,
v = velocity, and
g = acceleration due to gravity

Values of loss coefficient K_e for various types of entrances shall be assmed to be as given in Table 10.

(d) Transition Losses – Head loss in gradual constructions or expansions in a tunnel may be considered in relation to the increase to decrease in velocity head and will vary according to the rate of change of area and length of transition. These losses shall be assumed as specified in IS:2951 (Part II).

Table 10: Loss Coefficient for Tunnel Entrances

S. No.	**Type of Entrance**	**Loss Coefficient for Entrance, K_e**		
		Maximum	**Minimum**	**Average**
(1)	**(2)**	**(3)**	**(4)**	**(5)**
i.	Gate in thin wall-unsuppressed contraction	1.80	1.00	1.50
ii.	Gate in thin wall-bottom and sides suppressed	1.20	0.50	1.00
iii.	Gate in thin wall-corners rounded	1.00	0.10	0.50
iv.	Square-cornered entrances	0.70	0.40	0.50
v.	Slightly rounded entrances	0.60	0.18	0.25
vi.	Fully rounded entrances $\frac{r}{D} \geq 0.15$	0.27	0.08	0.10
vii.	Circular bellmouth entrances	0.10	0.04	0.05
viii.	Square bellmouth entrances	0.20	0.07	0.16
ix.	Inward projecting entrances	0.93	0.56	0.80

For gradual contractions, loss of head h_e shall be computed by the following equation:

$$h_e = K_e \left[\frac{v_2^2}{2g} - \frac{v_1^2}{2g} \right],$$

where

K_e	= loss coefficient for contraction,
v_2	= velocity in contracted section,
v_1	= velocity in normal section, and
g	= acceleration due to gravity

The value of loss coefficient K_e, shall be assumed to vary from 0.1 for gradual contractions to 0.5 for abrupt contractions. Where flare angle does not exceed those specified in 8.5.3.3 (b) the loss coefficient shall be assumed to vary in straight line relationship to a maximum of 0.5 for a right angle contraction.

(e) Bend and Junction Loss – Head loss at bends and junctions shall be assumed as given in IS: 2951 (Part II).

(f) Gate Loss in Pressure Tunnels – No gate loss need to be assumed if the velocity of flow is less than 1 m/s. Where a gate is mounted at either the upstream or downstream side of a thin head wall such that the sides and bottom of jet are suppressed and the top is contracted, loss coefficients given in Table 9 shall be taken. Where a gate is so mounted in a tunnel that the floor, sides and the roof, both upstream and downstream, are continuous with the gate openings, only the losses due to the slot shall be considered as given below assuming the value of loss coefficient K_g not exceeding 0.10:

$$h_e = K_g \frac{v^2}{2g},$$

where

h_e	= head loss at entrance,
K_g	= loss coefficient for gate,
v	= velocity, and
g	= acceleration due to gravity

(g) Exit Losses – Where no recovery of velocity head will occur, such as where the release from a pressure tunnel discharges freely, or is submerged or supported on a downstream floor, velocity head loss coefficient K_{ex} shall be assumed to be equal to 1.0. Head loss at exit shall be computed by the following equation:

$$h_g = K_{ex} \frac{v^2}{2g},$$

where

h_e	= exit head loss,
K_{ex}	= loss coefficient for gate
v	= exit velocity, and
g	= acceleration due to gravity

Where a diverging tube is provided at the end of tunnel, recovery of a portion of the velocity head will be obtained if the tube expands gradually and if the end of the tube is submerged, the loss coefficient K_{ex} shall be reduced from the value of 1.0 by the degree of head recovery.

8.5.3.5 Air locking and remedial measure

The presence of air in a pressure tunnel can be a source of grave nuisance. The following steps are recommended to prevent the entry of air in a tunnel:

(a) Shallow intakes are likely to induce air being sucked in. Through out the tunnel the velocity should either remain constant or increase towards the outlet end. It should be checked that at no point on the tunnel section negative pressures are developed.
(b) Vortices that threaten to supply air to a tunnel should be avoided, however, if inevitable they should be suppressed by floating baffles, hoods or similar devices.
I Partial gate openings that result in hydraulic jumps should be avoided.
(d) Traps or pockets along the crown should be avoided.

8.5.4 Rock cover

The minimum vertical as well as horizontal rock cover should be as shown in Fig. 46 In this minimum value of 'R' in metres shall be equivalent to maximum internal pressure head in metres inside the tunnel.

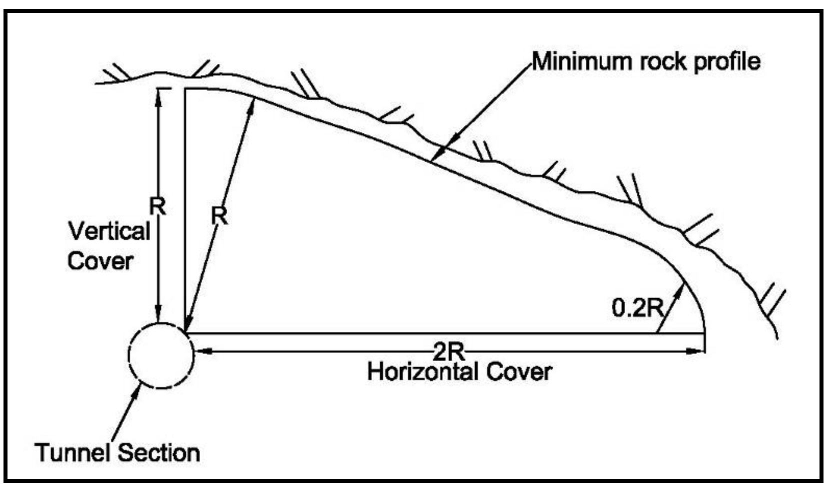


Fig. 46: Minimum Rock Cover in Power Tunnels

8.5.5 Design of concrete lining

The design of concrete lining may be carried out as per IS:4880 (Part IV). The minimum thickness of lining will, however, be governed by requirements of construction. For preliminary design of lining of tunnels, the following criteria may be adopted:

(i) The minimum thickness of unreinforced concrete lining should be 150 mm.
(ii) The minimum thickness of reinforced concrete lining should be 300 mm.
(iii) For big size tunnels in reasonably stable rock, the concrete lining thickness may be taken to be 60 mm per metre of finished diameter of tunnel.

(iv) Where structural steel supports are used, they may be considered as reinforcement by use of high tensile bolts at joints or by welding the joints. A minimum concrete cover of 150 mm should be provided over the inner flanges of steel supports.

8.5.6 Design of rock supports

Once the finished size and thickness of the lining is fixed, the excavated size of the tunnel is determined, the rock supports are required to be designed for this overall size of the bore. As the tunnels generally pass through different types of rock formations, it will be necessary to workout alternative cross-sections of the tunnel depicting acceptable and safe types of support systems. For preliminary as well as for final design of rock supports, Rock Tunnel Quality Index 'Q' method suggested by Grimstad and Barton can be used. The method is explained in Appendix I. Rock Mass Quality 'Q' which is a measure of rock mass characteristics is required to be determined by an experienced geologist. The value of ESR for various categories of excavation can, however, be determined from the Table 11.

Table 11: Estimated rock support categories based on tunneling quality index

Excavation Category		ESR
A.	Water tunnels for hydropower (excluding high pressure penstocks)	1.6
B.	Surge chambers, access tunnels	1.3
C.	Power stations, portal intersections	1.0

8.5.7 Grouting

(a) Purpose

The purposes of grouting are as follows:

(i) To fill up the gap between concrete lining and rock
(ii) To consolidate the rock around the tunnel to enable it to share a sufficient portion of internal pressure in the tunnel.
(iii) To introduce a certain degree of prestress in the lining as well as in the compound system of concrete and rock to avoid / reduce tensile stresses in the concrete.
(iv) To reduce drainage water.

(b) Types of grouting

The types of grouting are as follows:

(i) Backfill (contact) Grouting – The main purpose of contact grouting is to fill up all the voids and cavities between concrete lining and the rock. The contact grouting is normally restricted to the upper part of the tunnel. Grouting is done by drilling holes about 300 mm deep inside the rock, one on either side of the crown at 3 to 5 m apart and staggered in plan and at a longitudinal spacing of 3 to 5 metres. The grout pressure usually ranges from 2 to 3 kg/cm^2. Backfill grouting should be done not earlier than 21 days of concreting when the lining has attained enough strength.

(ii) Consolidation Grouting – Consolidation grouting is done to increase rock elasticity, to plug rock fissures so as to stop leakage and to introduce a compressive stress in the concrete lining and surrounding rock so as to reduce tensile stress in concrete lining caused by inside water pressure. As a thumb rule, a grout pressure of 1.5 times the water pressure in the tunnel may be used subject to the strength of concrete lining and safety against uplift of overburden. For small tunnels, rings of grouting holes may be spaced at about 3 metres centre and each ring may have four grout holes duly staggered in consecutive rings. The depth of holes may vary between 3 to 5 metres inside rock.

(c) Grout Mix

Grout mixes may vary from a very thin mixture of 20:1 to 0.5:1 (ratio of weight of water and cement). The normal range mixtures fall between 5:1 to 0.8:1. The choice of the grout mixtures at the start of grouting operations is based on the water tests before grouting or intake of holes already grouted. If the grout is too thick, passage of grout travel may get obstructed and fine seams may not be filled up. On the other hand if thin grout is continued for too long a time, the grouting operation may get unduly prolonged and may become too expensive. As a guide, the grouting should be started with a thin mix and the mix should be thickened if there is no increase in the pressure after a continuous grouting of about 10 minutes or after about 8 to10 bays of cement have been pumped. If the grout intake is too heavy, the cost of grouting can be reduced by thickening the grout by using inert materials like pozzolaous, fine sand, rock powder, clay or bentonite.

8.5.8 Drainage holes

Drainage holes may be provided in the crown portion in free flowing tunnels and even in the pressure tunnels to release external water pressure at the time of dewatering of tunnel. when the mountain material is likely to be washed into the tunnel. When pressure tunnel water coming out of the drainage holes is likely to endanger the side slopes of the hill mass, drainage holes should not be provided. When provided, at successive sections one vertical drainage hole may be drilled in the crown alternating with two horizontal holes drilled at the springing to a depth of 150 to 300 mm inside the rock.

8.5.9 Unlined tunnels

The power tunnels can be left unlined under the following conditions:

(i) When the rock quality is good and there is no likelihood of rock pieces getting detached due to flowing water

(ii) When there is no possibility of leakage of water through the crevices of the rock mass around the tunnel resulting water loss and destabilization of side slopes of the hill

(iii) When the available water head for power generation is substantial and a little extra head loss occurring due to rough surface of the unlined tunnel does not materially affect the economics of the project. If the tunneling is carried out by Tunnel Boring Machine, the surface roughness of the tunnel will be very much less.

8.5.10 Tunnel plugs

Concrete plugs are provided at the junctions of construction adits and the main tunnel. The plug may be a solid concrete plug or a concrete plug with an inspection gate.

8.5.10.1 Design of plug

(a) Design Head – The design head shall correspond to maximum reservoir level with surge and water hammer effects, if any.

(b) Plug length – The length of the plug shall not be less than the excavated diameter of the tunnel. The plug length may be calculated based on the following formulae:

(i) $$\text{Length of plug} \geq \frac{\text{Hydrostatic force on the plug}}{\text{Permissible average shear stress} \times \text{perimeter of tunnel section}}$$

(ii) $$\text{Length of plug} \geq \frac{\text{Hydrostatic force on the plug}}{\text{Frictional resistance per unit length of plug}}$$

(ii) $$\text{Length of plug} \geq \frac{\text{Hydrostatic force on the plug}}{\text{Frictional resistance of plug and shear resistance of anchors per unit length of plug}}$$

Values of permissible average shear stress / shear friction between the plug and surrounding concrete/ rock should be on conservative side and ample factor of safety should be considered. It is preferable to cast the whole length of plug in one block without any construction joint.

(c) Key in Concrete / Rock – Wedge shaped keys are provided at rock / concrete interface and within concrete (if the concrete has been poured in two stages) to ensure effective plug action by providing adequate bearing of the plug concrete on the tunnel lining or plug concrete / tunnel lining the surrounding rock. The depth of the key may be calculated from the average bearing stress in concrete / rock from the following formula. The permissible stresses in concrete shall be as per IS:456. In many cases the stresses in concrete will govern as rock is generally stronger.

$$\textit{Average bearing stress} = \frac{\textit{Hydrostatic force on the plug}}{\textit{Effective projected bearing area}}$$

The effective projected bearing area shall be assumed 75% of the total projected bearing area of the kays to allow for imperfections in grouting. The depth of any key will not be less than 450 mm.

(d) Grouting – Contact and consolidation grouting as described in clause 8.5.7 shall be done around the plug in order to improve the quality of surrounding rock and to ensure better shear friction.

Typical details of concrete plugs are shown in Fig. 47 and 48.

8.6 Pipes – Steel, RCC, High Density Polyethylene (HDPE) and Glass Fibre Reinforced Plastics (GRP)

Use of different types of pipes in the water conductor systems of small hydro projects is quite common. The use of pipes as a water conductor system has the following inherent advantages:

(i) In case the means of communication upto the project site are good, the construction of the water conductor system can be done very speedily.

(ii) Depending upon the availability of the pipes at a nearby place, the work may be more economical.

(iii) Maintenance cost of the pipe system is lesser.

(iv) The chances of water leakage and thus destabilizing of hill slopes are lesser.

(v) Since the value of rugosity coefficient in any type of the pipe will be lesser than that of the earth surface or concrete surface, the head loss in the water conductor system will be lesser.

8.6.1 Laying of pipes

As a part of water conductor system, the pipes are laid along the contour. The pipes may run under pressure or free flow. In any case, there should not be any hump along the alignment. If designed to be free flow, the pipes should be laid at sufficient slope so as to carry the designed discharge.

8.6.2 Steel pipes

Steel pipes can be installed underground as well as exposed on ground surface.

(i) Underground pipes

(a) The pipes do not need any saddle supports nor any expansion joints.

(b) The pipes are quite safe against rock falls and seismic shocks

(c) The pipes do not need painting and are almost maintenance free.

Since the underground pipes are liable to get rusted, they should be wrapped with some anticorrosive sheet or be provided with a layer of 150 mm thick shotcrete.

(ii) Surface pipes –Pipes laid on surface as a part of water conductor system, are installed at a very mild longitudinal slopes of 1 in 100 or lesser. The pipes as such do not need any anchor blocks but should be installed on saddles at a spacing of 5 to 6 metres. The saddles should be atleast 600 mm above ground, so that the outer surface of the pipes could be painted as and when required and it could be away from vegetation. The pipes would need anchor blocks or straps anchored to the adjoining rock at the locations of horizontal curves. Additionally the surface pipes will be required to be provided with expansion joints at a spacing of 50 to 100 metres, lesser spacing being at the locations of horizontal curves. The design of steel pipes, whether underground or surface can be carried out on lines similar to those of penstocks mentioned in subsequent chapter.

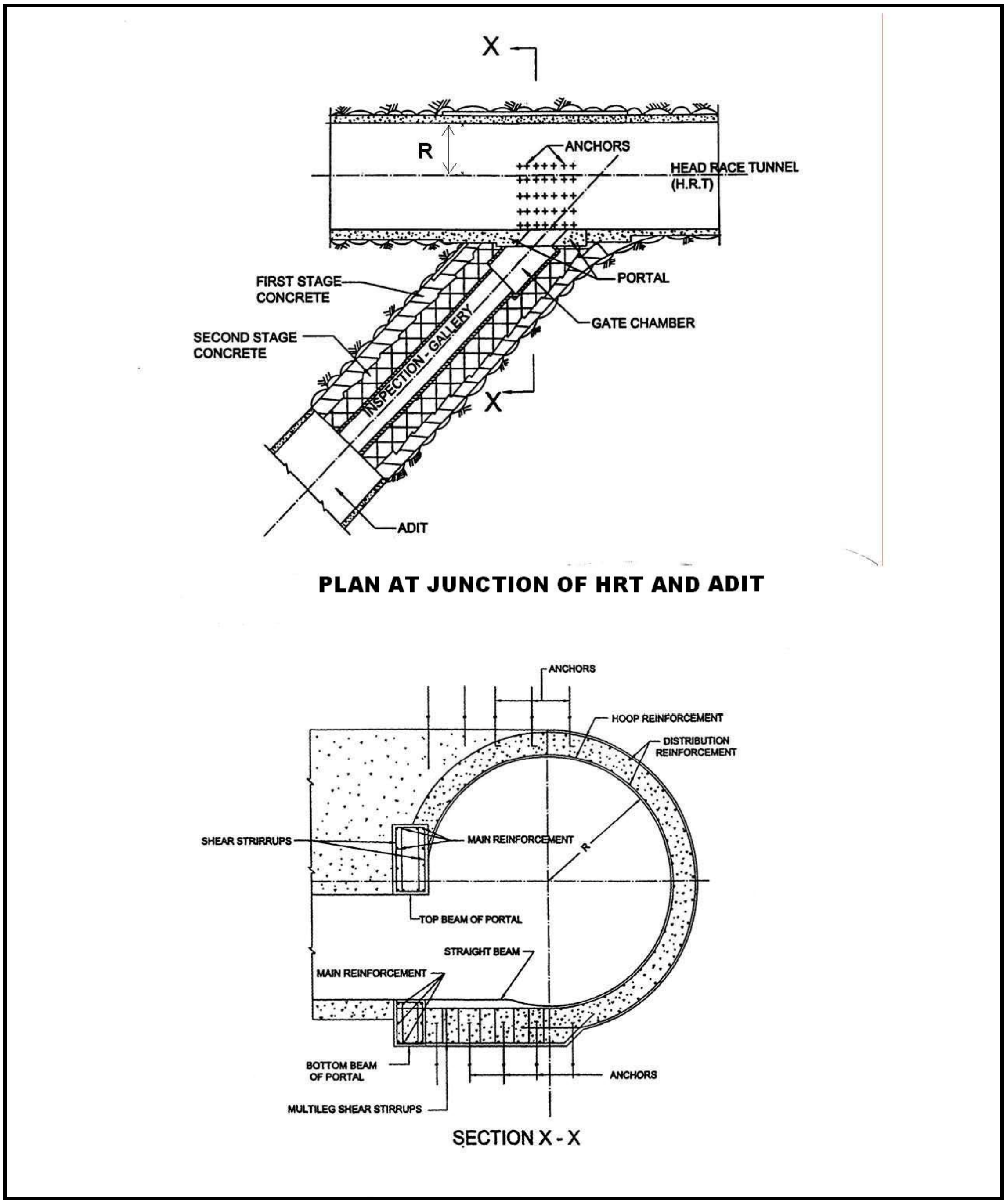


Fig. 47: Details of Plug with Inspection Gallery

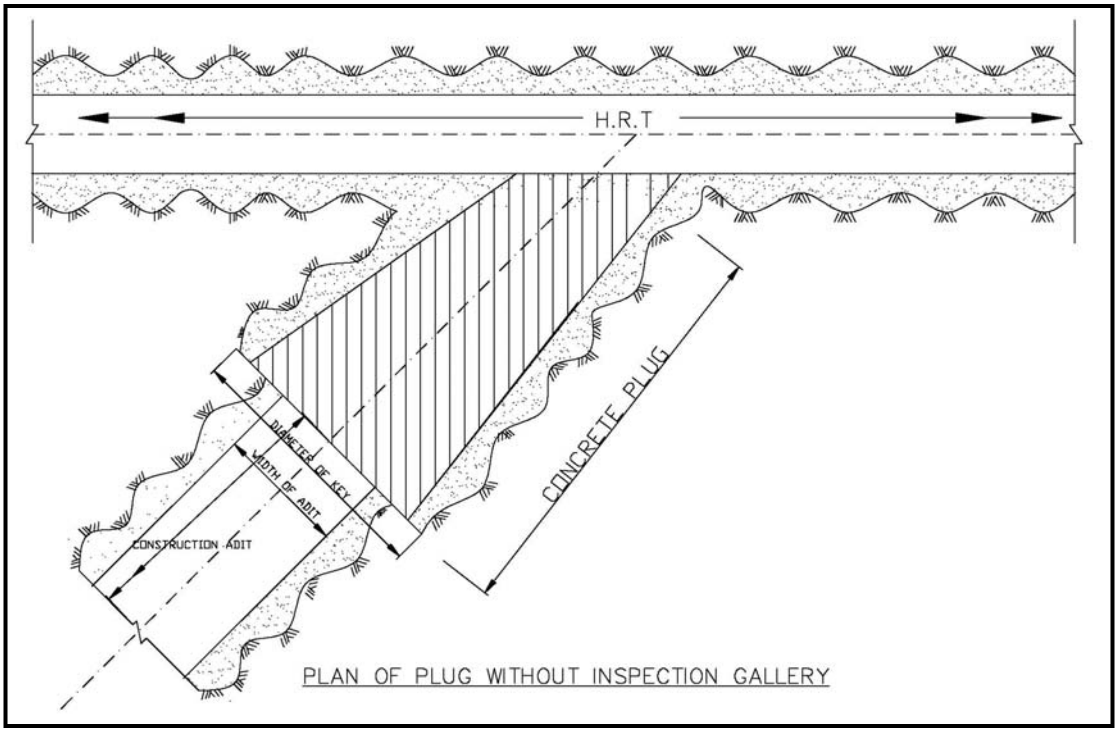


Fig. 48: Details of Plug without Inspection Gallery

8.6.3 R.C.C. pipes

RCC pipes are available in sizes from 300 mm to 2200 diameter and in lengths of 2000 mm to 4000 mm. RCC pipes can be installed above ground or below the ground surface. However, they should preferably be installed underground at a depth of about 1 metre or so. Underground pipes are safe against rock falls and seismic shocks. The individual pieces of the pipes should be jointed together as per instructions of the manufacturer. If installed underground, these are almost maintenance free. These pipes can withstand upto 60 metres of water pressure.

8.6.4 High density polyethylene (HDPE) pipes

HDPE pipes are available in sizes from 200 mm to 1000 mm diameter and in lengths of 5 to 20 metres. These are needed to be protected from sunlight and as such they should be installed underground and individual pieces should be jointed according to the instructions of the manufacturer. The rugosity coefficient these pipes is of order of 0.01 and they can withstand upto 160 metres of water pressure.

8.6.5 Glass fibre reinforced plastics (GRP) pipes

GRP pipes are available in sizes from 200 mm to 3000 mm diameter and in lengths upto 12 metres. These pipes can be installed on the surface as well as underground. The rugosity coefficient of these pipes is of the order of 0.01 and they can withstand upto 30 to 150 metres of water pressure. These are relatively much lighter compared to steel pipes and can be installed at faster pace.

8.7 Cross- Drainage Works

Cross – drainage works of small hydro-electric projects may be of two categories:

(1) Drainage passing on the top of the power channel
(2) Drainage passing under the power channel

8.7.1 Cross – drainage work with the natural drain passing above the power channel

When the conditions so demand, the natural drainage can be passed over the RCC section of the power channel as shown in Fig. 49. In this case the power channel will be designed for the load of the drainage bed material as well as drain water corresponding maximum water level. The sides of the drain will be protected by guide bunds with their foundations at a depth of atleast 1.25 times the scour depth. The clear width of between the guide bunds will have to be sufficient enough to pass the estimated flood water of the drain. The top of the guide bunds should be atleast 0.5 metre above the maximum estimated water level of the drain.

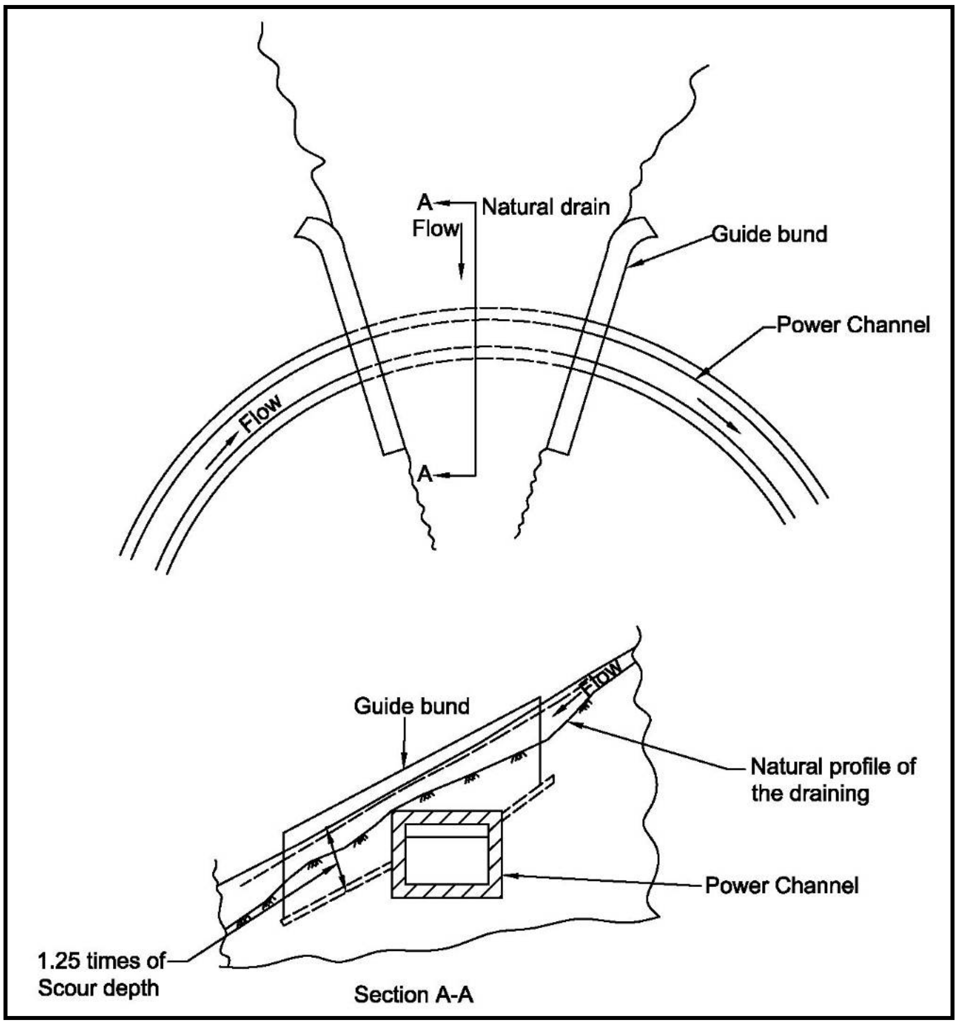


Fig. 49: Typical Arrangement of drain crossing above power channel

8.7.2 Cross drainage work with the natural drain passing below the power channel

This type of cross-drainage works may be divided into three categories:

(i) Drainage crossings to pass rain water of the hill toe drain
(ii) Drainage crossing across major drainages with RCC aqueduct
(iii) Drainage crossing across major drainage with pipe aqueduct.

8.7.2.1 Drainage crossing to pass rain water of the hill toe drain

A typical section of a drainage crossing to pass rain water of the drain constructed along the toe of the hill towards valley side is shown in Fig. 50.

Such types of cross-drainage should be constructed at an interval of 300 to 500 metres. The cross – drainage may comprise an RCC barrel of atleast 1 m x 1 m size or a pipe of atleast 0.9 m internal diameter so that a man can go inside to clear the debris deposited therein. As far as possible, the location of these works should be chosen in such a way, that the outlet end of the drainage barrel is located on good unerodoble rock face. In the absence of such a situation, the hill face at the location of the outlet end of drainage barrel should be protected with stone pitching in sufficient width ie. in a width of atleast 1 to 2 metres on either side of the barrel.

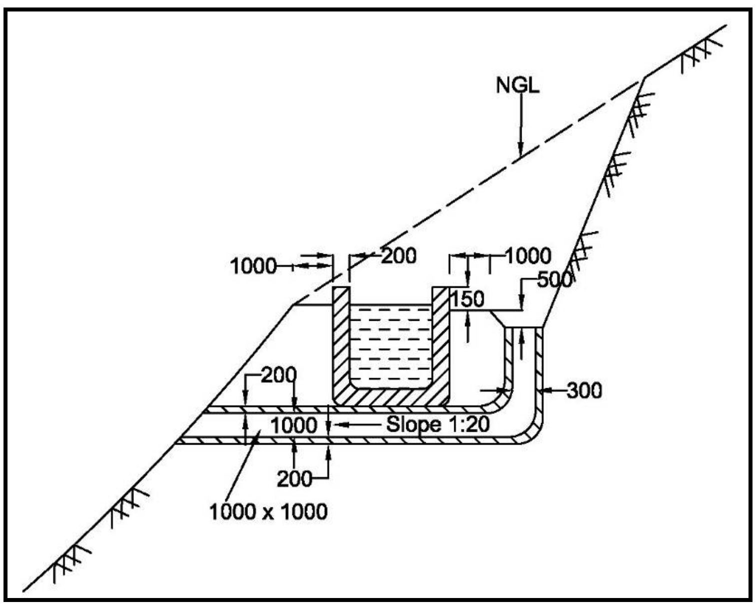


Fig. 50: Cross Drainage work to Pass Hill Drain Water

8.7.2.2 Drainage crossing across major drainage with RCC aqueduct

A typical arrangement of an RCC aqueduct is shown in Fig. 51. Such an arrangement is quite suitable for aqueducts upto 10 metres span. In such a case there is no need to provide a central pier. The foundations of the abutments should either be kept on firm unerodable rock or should be taken upto a depth of 1.75 times the scour depth below high flood level. There should be a headroom of atleast one metre between the high flood level of the drainage and the bottom of the bottom slab of aqueduct. In case the power channel is rectangular, the same size can be maintained in the aqueduct portion also, but if the shape of the power channel is trapezoidal, it will be needed to be converted into a rectangular section keeping the width and water depth in the flumed section almost the same. In case of a power channel of trapezoidal section, the principal dimensions of the entry and exit transitions and the flume section of the aqueduct can be determined in the following manner.

With reference to Fig. 51, A = BD + DS where A is the area of cross-section of the trapezoidal headrace channel and d $\approx$ b $\approx$ D.

The invert elevations across the entry and exit structures can be calculated by Bernauli's equation as below:

For sections (1) & (2)

$$Z_i + D + \frac{V_1^2}{2g} = Z_2 + d + \frac{V_2^2}{2g} + h_{len}$$

Where,

Z_1 and Z_2	are elevations,
V_1 & V_2	are average velocities at sections (1) and (2) respectively, and
h_{len}	$= 0.1\left(1 - \frac{b}{B}\right)\frac{V_2^2}{2g}$

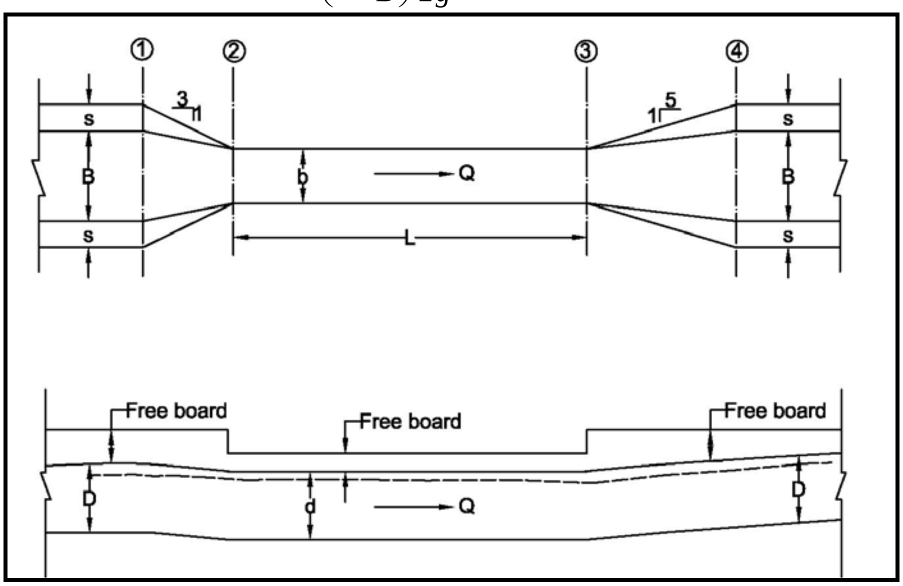


Fig. 51: Geometric Layout of Flumed Aqueduct

From the above equation, Z_2 can be determined as all the remaining parameters are known. For sections (2) to (3), the slope of the flume section can be determined from Manning's equation,

Slope 's' = $\left[\frac{V.n}{R^{2/3}}\right]^2$, the value of n for concrete channels can be taken as 0.014.

For section (3) & (4),

$$Z_3 + d + \frac{V_3^2}{2g} = Z_4 + D + \frac{V_u^2}{2g} + H_{lex}$$

In this $Z_3 = Z_2 - s \times L$ and $h_{lex} = 0.2\left(1 - \frac{b}{B}\right) \times \frac{V_3^2}{2g}$

Since all the other parameters are known, Z_4 can be determined.

The structural design of the flumed section can be carried out as RCC trough section. It would be better if the flumed section is designed as a closed section so that the top slab could be used as a bridge across the drain. A typical section of the flume is shown in Fig. 52.

In this case the side walls are designed as beams to withstand the loads of water as well as dead and live load on top slab. In case the depth of the side walls as beams is found to be insufficient, the side parapets may also be considered as part of the beams. The concrete used should be of atleast M-25 grade and the bottom slabs & side walls should be designed as water retaining structures as per IS: 3370- "Code of Practice for Concrete Structures for the Storage of Liquids".

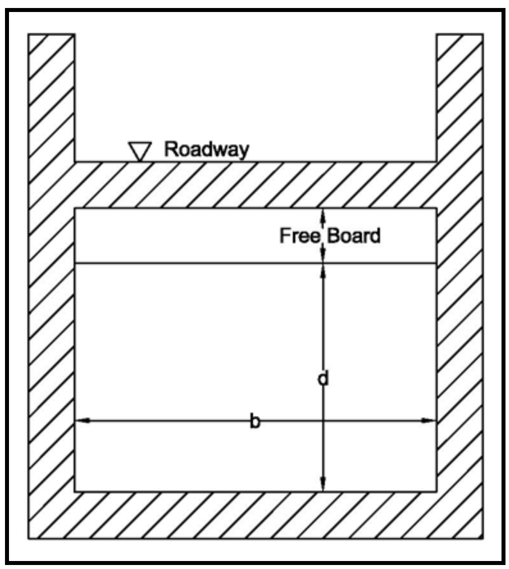


Fig. 52: Typical flume section

8.7.2.3 Drainage crossing across major drainage with pipe aqueduct

A typical arrangement of a pipe aqueduct is shown in Fig. 53.

This arrangement can be used upto 150 to 200 metres of the aqueduct. As mentioned in para 8.7.2.2, the foundations of the abutments and the central piers should either be kept on firm unerodable rock or should be taken upto a depth of 1.75 times the scour depth below high flood level of the drain. There should be a headroom of atleast one metre between the high flood level of the drain and the bottom of the pipe. The distance between the intermediate piers / saddles should not be more than 5 to 6 metres. The velocity in the pipe can be upto 4 metres per second and it can be designed as a pressure pipe. The design of the pipe can be carried out on lines similar to surface penstock as described in subsequent pararas.

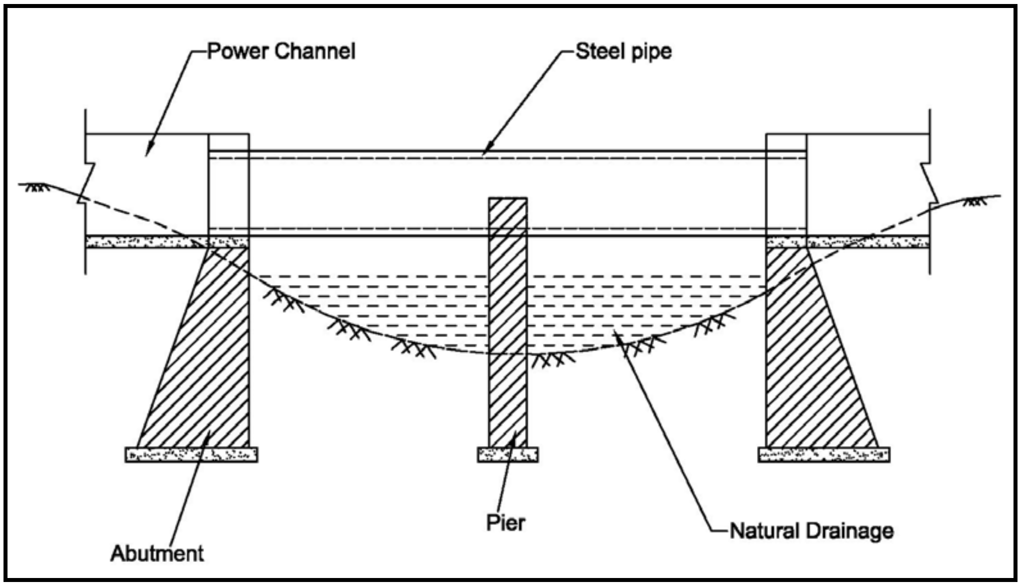


Fig. 53: Typical Section of Pipe Aqueduct

8.8 Desilting Devices

Intakes on rivers and streams are designed to eliminate floating debris and a large amount of bed load. However, they can not prevent the entrance of suspended sediment and the entire amount of bed load. Desilting devices are, therefore, required to avoid sedimentation of the water conductor system and to limit the possible damage of the turbine parts due to abrasion against the sediments. Normally two types of arrangements i.e. (i) Desilting chamber (ii) Vortex type, are provided for silt removal. Sometimes either of these or both of these are provided in series.

8.8.1 Desilting chamber

Desilting chamber is the most commonly used device, to reduce silt load. Removal of silt is affected through reduction in flow velocity by increasing the flow area and thus letting the bigger size silt particles settle to the bottom of the tank. The settled particles are removed continuously through a silt flushing arrangement provided at the bottom of the chamber.

8.8.1.1 Silt flushing discharge

The silt flushing discharge should be able to flush out the pebbles of 30 to 50 mm size at a flushing velocity of 3 to 4 metres / sec. The silt flushing discharge normally ranges from 20 to 25% of the intake discharge. As such, if Q cumecs is the design discharge required for power generation, the intake should draw a discharge of Q/0.8 to Q/0.75 cumecs to allow a flushing discharge of 20 to 25%.

8.8.1.2 Flow through velocity

The limiting flow through velocity can be determined by the relation

$$V = a\sqrt{d},$$

Where,

V = flow through velocity in m/sec.,

d = diameter of particle upto which sediment load is desired to be removed in mm and

a = constant which is equal to
0.36 for d > 1 mm
0.44 for 1 mm > d > 0.1 mm

In small hydro projects normally the silt particles more than 0.25 mm size are removed. For this the flow through velocity works out to 0.44 $\sqrt{0.25}$ = 0.22 m/sec.

8.8.1.3 Fall velocity

Fall velocity of suspended sediment can be determined from the Table 12.

Table 12: Fall Velocity of Suspended Sediment

Particle Size in mm	Fall Velocity in cm / sec. at a Water Temperature of		
	0^oC	15^oC	40^oC
0.15	1.00	1.50	2.00
0.20	1.70	2.10	3.00
0.25	2.10	3.00	4.00
0.30	3.00	4.00	5.00
0.40	4.20	5.30	6.50
0.60	7.00	9.00	11.00
0.80	10.00	12.00	14.00
1.00	13.00	15.00	17.00
2.00	26.00	27.50	30.00
4.00	42.00	43.00	44.00
7.00	60.00	60.00	60.00
10.00	72.50	72.50	72.50

8.8.1.4 Upstream and downstream transitions

The desilting chamber is joined with the water conductor system at its upstream and downstream ends through gradual expansion and contraction transitions respectively. These transitions should not be steeper than 1:6 in expansion transition and 1:3 in contraction transition.

8.8.1.5 Data required for design

The following data is required for the design of desilting chamber:

(i) Design discharge 'Q_d' in cumecs at the outlet end of desilting chamber
(ii) Silt flushing discharge 'Q_f' in cumecs
(iii) Total discharge 'Q' in cumecs at inlet end = $Q_d + Q_f$
(iv) Minimum particle size 'd' to be removed in mm and corresponding fall velocity

(v) Type of flow i.e. free flow or pressure flow
(vi) Available width in metres for constructing the chamber

8.8.2 Design

The design of desilting chamber involves following steps

(i) Determine flow through velocity in m/sec corresponding minimum particle size to be removed.
(ii) Determine the area of the desilting area by dividing the total discharge 'Q' by flow through velocity.
(iii) Fix the width 'W' of the desilting chamber in accordance with the space available at site. In case of underground desilting chamber, the width of the cavity will have to be decided on the basis of geological considerations. A lesser width will result greater depth of desilting chamber and larger length and vice-versa
(iv) With area and width of the desilting chamber, determine the depth 'D' of the chamber.
(v) Determine the fall velocity 'V_f' of silt particles as per Table 5.
(vi) Calculate moderated setting velocity 'V_{fm}' to take into account the turbulence at the inlet transition by the following relation

$$V_{fm} = V_f - \frac{0.132 \times V_f}{\sqrt{D}}$$

(vii) Calculate the settling length 'L' of the chamber by the following relation

$$L = \frac{Flow\ through\ Velocity \times D}{V_{fm}}$$

(viii) Divide the length of the chamber into a suitable number of hoppers, generally the shape of hoppers is a square as shown in Fig. 54. In case the width of the desilting chamber is too large, there can be two or more number of rows of hoppers along the length of the chamber. The slope on all the four sides of the hoppers is kept at an angle of 45^o so that any silt that falls into the hopper slides down into the hole provided at the bottom of the hopper. Alternatively instead of providing hoppers only side slopes of 45^o are provided at the bottom of desilting chamber as indicated in section A – A of Fig. 54. In this arrangement a slab with holes segregates the desilting chamber from the silt flushing channel. The sizes of holes varies from upstream to downstream, the upstream holes being bigger and downstream holes being smaller.

(ix) The sizes of the holes provided at the bottom of the hoppers is fixed with the following criteria:

(a) The first hole / the first row of holes, if there are more than one row of hoppers, should be able to carry a minimum of 10% of the flushing discharge.

(b) The size of other holes / other row of holes shall be decided on the criteria of passing equal discharge through each hole. It will be done by equalizing the head losses from inlet of the hole upto the end of desilting chamber in the silt flushing channel. As such the sizes of holes will go on reducing from upstream to downstream.

(x) Determine the size of the silt flushing tunnel in such a way that the velocity in the tunnel is never less than 3.0 m per sec.

(ix) Silt flushing tunnel should preferably be lined with 10 to 12 mm thick steel plate and it should discharge into the river at such a location that the invert of the tunnel remains atleast 1.0 metre above the high flood level of the river at that location.

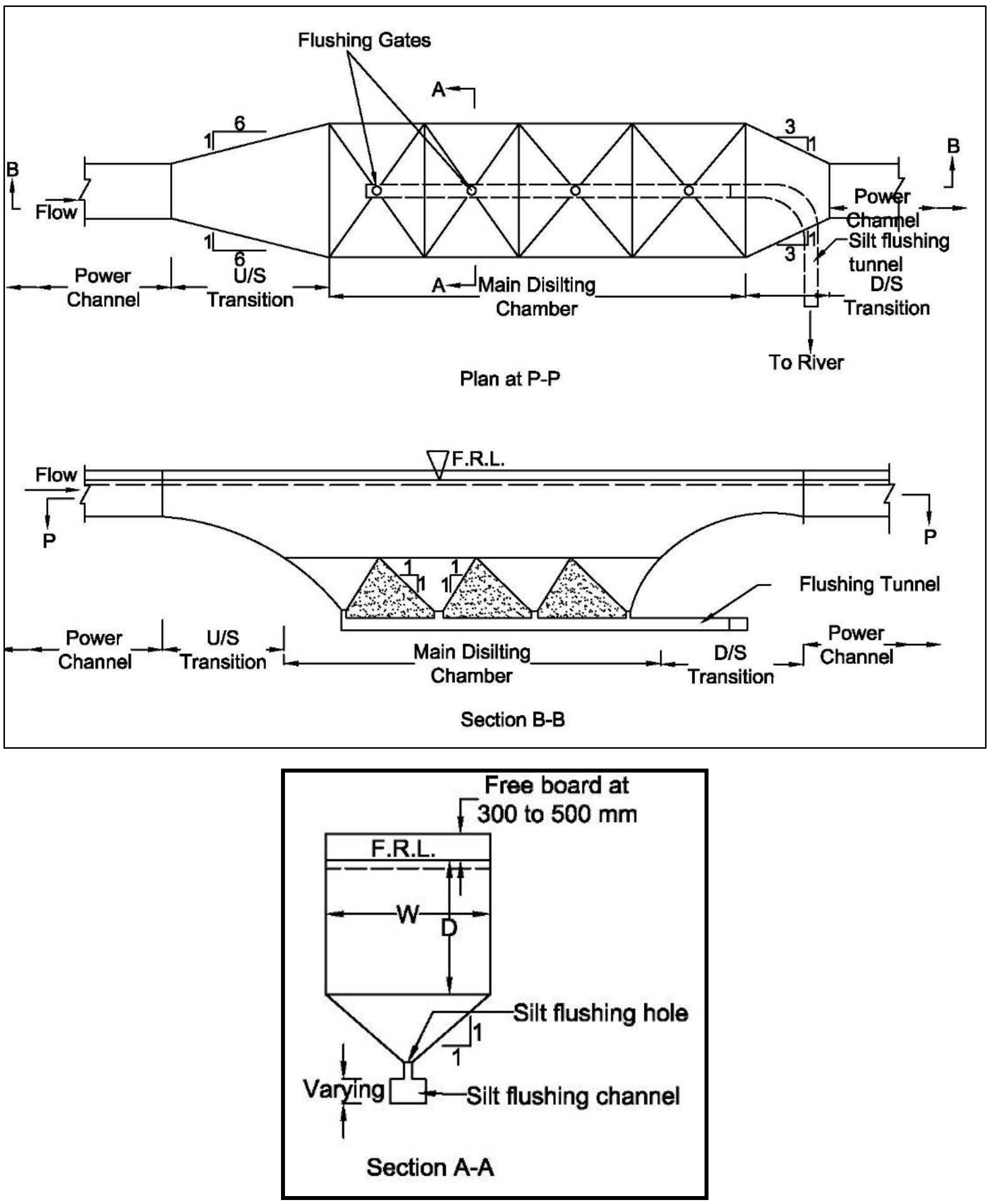


Fig. 54: Schematic Arrangement of Desilting Chamber

8.8.3 Vortex tube

The vortex tube is an open tube placed across the power channel bed either normal to flow or at angle of 30^o to 45^o to the flow. As the water flows tangentially over the tube, vortex flow with a speed of rotation of the order of 200 to 500 rpm is set up in the tube, which is sufficient enough to eject coarse gravel and even cobbles. The rate of outflow at the downstream end of the vortex tube is controlled by a gate. For maximum efficiency both the lips of the tube should be kept at the same level and Froude number $\frac{V}{\sqrt{gd}}$ should be around 0.8. The diameter of the vortex tube should be equal to about the depth of flow. In case the Froude number happen to be less, the same can be increased by contracting the width of the channel at the location of the tube. Vortex tubes can be used with or without desilting chamber, which, if provided, should be provided in series at the downstream side of the tube.

8.8.3.1 Design data

The following data is required for the design of vortex tube:

(i) Cross-section of channel at the proposed location of vortex (this should be modified as per recommendation given in para 8.8.3).

(ii) Design discharge 'Q' of the channel.

(iii) Allowable flushing discharge, which may vary from 10 to 20% of the total discharge at the location of the tube.

(iv) Material size distribution of bed load

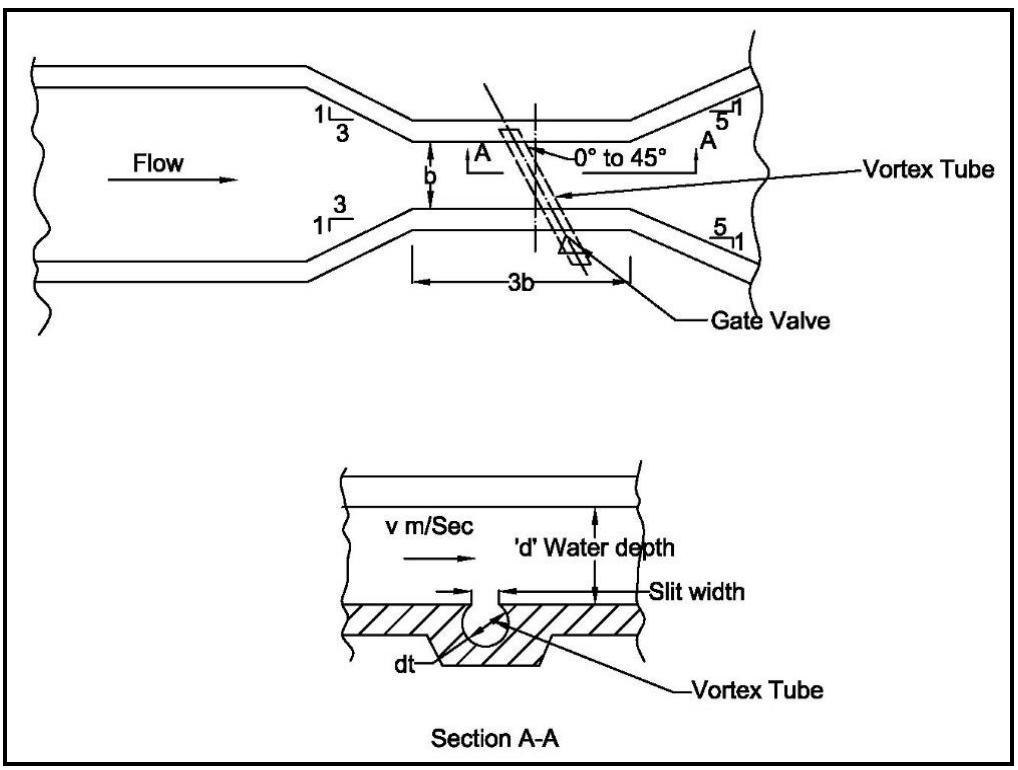


Fig. 55: General Arrangement of Vortex Tube

8.8.3.2 Design procedure

The design can be carried out in following steps:

(i) Determine the fall velocity of sediment fractions d_{10}, d_{30}, d_{50}, d_{70}, d_{75} and d_{90} and let they be V_{f10}, V_{f30}, V_{f50}, V_{f70}, V_{f75} & V_{f90} respectively.

(ii) Choose operating head 'h_o' i.e. the head difference between the centre line of tube in the channel and that at the outlet end. This can vary from 0.1 m to 0.7 m or so according to site conditions.

(iii) Calculate the minimum tangential velocity U_o at the closed end of tube using the formula:

$$U_o = \frac{0.693}{Cosh\ r}\sqrt{2gh_o},$$

in which $r = \frac{0.2422\ L}{dt}$, L being the length of the tube and d_t its diameter. For efficient flushing of the vortex tube, U_o should be greater than V_{f90}. Thus substituting $U_o = V_{f90}$, the above equation can be solved for cosh r and thus $\frac{L}{dt}$ can be determined. Since L is known, dt can be found out. For the value of $\frac{L}{dt}$ obtained above and an assumed value of h_o, determine the average velocity in the tube U_a as

$$U_a = 0.721 \tanh_r \sqrt{2gh_o}$$

This equation is valid for a slit width equal to 0.3 d_t, which is the recommended value. The flushing discharge flowing in the tube Q_f can now be determined from the following relation:

$$Q_f = \left(\frac{\pi}{4} d_t^2\right) U_a$$

(iv) The ratio of $\frac{Q_f}{Q}$ can thus be determined. In case it does not lie between the acceptable limit of 0.1 to 0.2, the design will be required to be repeated by modifying either h_o or L. Possibility of using two or three tubes instead of one can also be considered.

9.0 FOREBAY

The forebay is a small storage pond located in between head race channel and penstocks. It provides immediate water demand on starting the generating units and required water seal over the penstock inlet against air entrainment. Suitable spillway is provided on one side of forebay to dispose off safely the excess inflows during load rejection.

9.1 Functions of Forebay

Following functions are sewed by forebay tank:

(a) Distribution of Flow – The forebay should have enough length to distribute flow smoothly and uniformly and necessary width for arranging the intake, trash racks, penstock and spillway.

(b) Regulation of flow – The forebay should have certain storage volume to regulate the flow when the plant is suddenly loaded.

(c) Protection against silting and floating debris – Silt sluice and trash racks should be provided in the forebay against harmful particles of silt and floating debris pasing into the turbines.

(d) Provision of water seal over intake - The forebay provides minimum required water seal over the intake at MDDL against air entrainment.

(e) It should have adequate provision to escape the design discharge in the event of sudden closure of machines.

9.2 Layout of Forebay

The layout and dimensioning of the forebay depend on the topographic and geological conditions and layout of its appurtenances such as spillway, silt sluices, intake structure etc. It should be located as near to the power house as possible so as to have minimum length of penstock.

9.3 Dimensioning of Forebay

The forebay should be so dimensioned to meet out its functions and normal operation of the plant. The minimum capacity of forebay is normally kept corresponding to a storage of 2 to 3 minutes of the design discharge with an operating depth of 2 to 3 metres. The dimensioning of forebay is so adjusted to meet out required volume of storage and accommodate its appurtenances. Thus, the hydraulic design parameters of forebay are determined as follow:

(a) Capacity of Forebay in cubic metre = 120 to 180 Q_d, where, Q_d is the design discharge of head race channel in cumec

(b) Width of forebay is determined in accordance with the topographical conditions at the site. In case of steep terrain the forebay can have less width and more length and should be aligned along the contours. In case the penstocks emerge from the other end of the forebay, the width of the forebay should be enough to accommodate the penstocks. The penstocks can also take off perpendicular to the length of the forebay depending upon the location of power house.

(c) IMaximum water depth of flow in forebay is generally kept from 2 m to 3 m below steady state level with an average flow velocity in forebay less than 0.5 m/s or even less so that it can be expected to settle out the harmful sediment particles.

(d) After fixing the width and depth of the tank, the length can be determined. The length should be aligned along the existing contour.

(e) Submergence depth of intake

The minimum submergence required at the centre line of penstock below MDDL against air entrainment may be calculated by the following formula given by Gordon,

$$\frac{h_s}{D} = 0.5 + 2F_r \qquad (4)$$

Where,

h_s = depth of submergence at intake centre line below MDDL

D = depth of opening at intake gate axis

F_r = Fronde number = $\frac{V}{\sqrt{gD}}$

V = Velocity of flow through intake

g = acceleration due to gravity

In order to achieve the required submergence, the bottom of the forebay floor is normally depressed at the location of the penstock. The forebay floor may be further depressed by about 0.5 m to 0.75 m below the bottom level of penstock to accommodate the silt flushing sluice.

(f) Transition between HRC and forebay

A suitable expansion transition having a side splay of 5:1 and bed slope of 4 (H) : 1 (V) may be provided to join headrace channel with forebay.

(g) Spillway

In case of small hydropower projects an ungated spillway crest of suitable length at the steady state level of forebay is designed for the design discharge of head race channel for a limited head of about 0.3 m to 0.5 m over the crest in order to dispose off the incoming discharge at the time of sudden tripping of the machines. The spillway crest is ogee shaped and is aligned along the wall of forebay in such a way so that it could be discharge safely through a spill channel / pipe into a nearby drain or river. A typical layout of forebay tank shown in Fig. 56.

10.0 SURGE TANK

10.1 Introduction

Surge tank is provided at the junction of headrace tunnel / pressure conduit and the penstock. The surge tank absorbs the water hammer or elastic shock waves coming from the penstock as a result of hydraulic transient conditions during load rejection or acceptance by the power house and also to supply / store additional water during load demand / rejection until the pressure conduit velocity has accelerated / decelerated to the new steady state value. Surges or mass oscillations occur in the surge tank. The hydraulic design of surge tank should be so as to keep the surges within reasonable limits. There are five types of surge tanks; simple surge tank, restricted orifice surge tank, double chamber surge tank, surge tank with over flow weir and differential surge tank. Out of these a simple or restricted orifice type surge tank is commonly used in small hydropower projects. General details of surge tanks are shown in Fig. 57 (a) and 57 (b).

10.2 Data Required

The following data are required for the hydraulic design of surge tank:

(a) Length, diameter and maximum discharge of head race tunnel
(b) Length, diameter, number of penstocks and discharge of each.
(c) Net head on turbine
(d) Maximum and minimum operating levels of reservoir
(e) The maximum and minimum head losses in head race tunnel
(f) Critical transient conditions of load changes.
(g) If the power station operates in isolation or in grid.

10.3 Design

10.3.1 Design conditions

The surge tank is designed to accommodate the maximum and minimum surge levels under worst transient conditions of power house. The following conditions are considered for design:

(a) Maximum upsurge level in the surge tank is worked out corresponding to full load rejection with minimum friction losses in HRT at highest operating reservoir level

or

Where considered necessary specified load acceptance followed by full load rejection at the instant of maximum velocity in HRT and higher of the two is adopted.

(b) For minimum down surge level, the worst of the following two conditions is considered.

(i) Full load rejection at MDDL followed by a specified load acceptance at the instant of maximum negative velocity in HRT.
(ii) Specified load acceptance at MDDL with maximum friction losses in HRT.

10.3.2 Coefficient of hydraulic loses

For tunnels flowing full, friction loss may be determined by using Manning's formula. For various types of tunnels, the value of Manning's Rugosity coefficient 'n' may be taken as given in Table 13:

Table 13: Manning 'n' for Different Tunnel Surface

S. No.	Tunnel Surface Condition	Value of 'n'	
		Minimum	Maximum
1.	Concrete lined	0.012	0.014
2.	Unlined very rough	0.04	0.06
3.	Unlined trimmed	0.025	0.035
4.	Side faces unlined and trimmed and invert concreted	0.02	0.03

The minimum value of 'n' gives worst upsurge while the maximum value gives the worst down surge. As such maximum and minimum values of 'n' should be used for the relevant conditions.

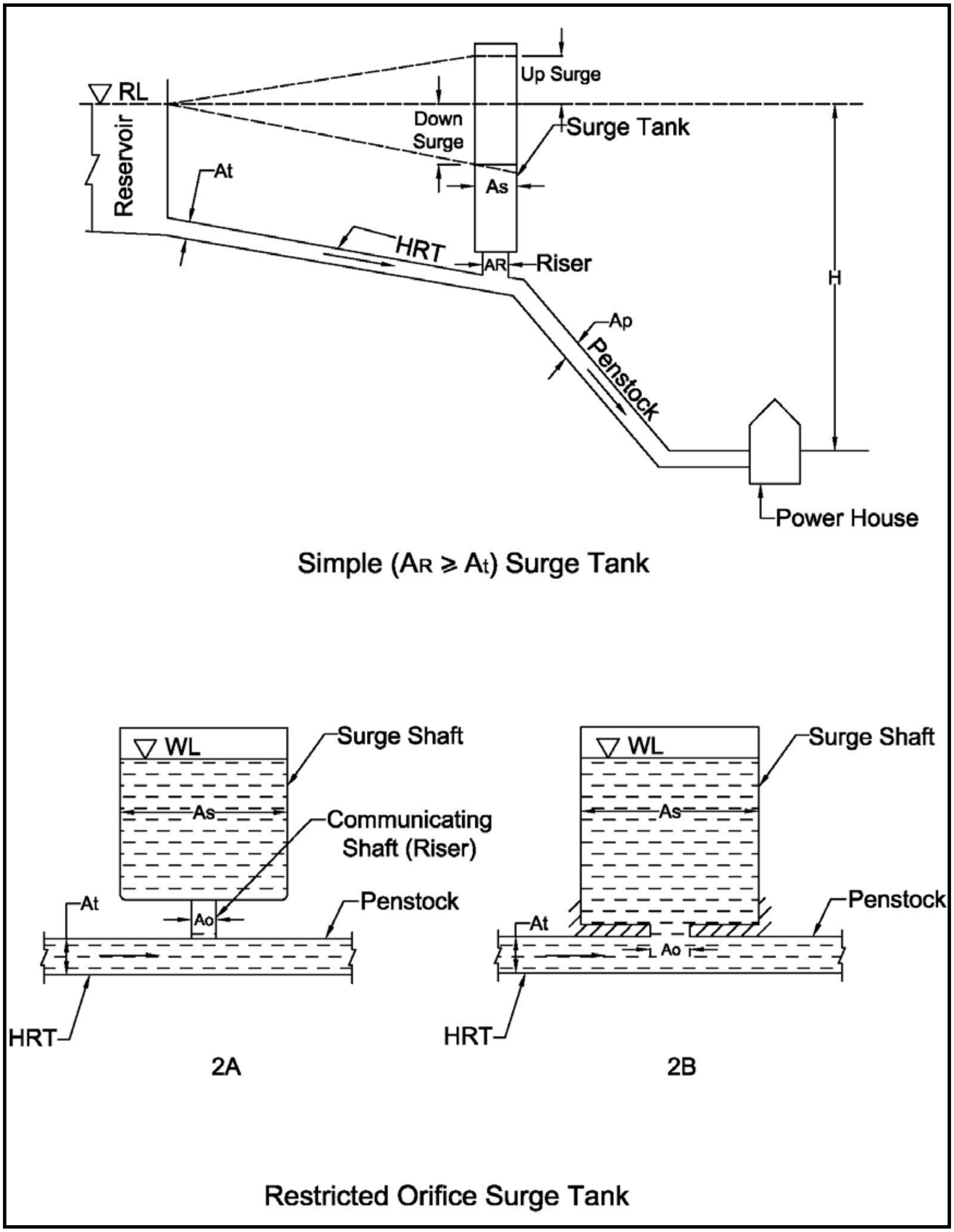


Fig. 56: Surge Tank Details

10.3.3 Area of surge tank

To ensure the hydraulic stability of surge tank, its area is governed by Thoma criteria applicable for a power station in isolation, which is;

$$A_{th} = \frac{L\ A_t}{\beta V_t^2\ H_o}\ \frac{V_t^2}{2g}$$

Where,

- A_{th} = Thoma area of surge tank.
- A_t = Cross sectional area of head race tunnel
- H_o = Net head on turbine
- L = Length of head race tunnel
- V_t = Velocity of flow in head race tunnel
- β = Coefficient of hydraulic losses in HRT such that $h_f = \beta V_t^2$
- g = Acceleration due to gravity
- h_f = Hydraulic losses in HRT

If power station always operates in a grid, the stabilizing effect of the grid may be taken into account and area of surge tank (A_s) is modified by following formula:

$$A_s = A_{th}\ [1\text{-}1.5\ (1\text{-}K)]$$

Where K = ratio of total power generated by the station to that of the grid

The area of surge tank calculated by adopting above said equation is the minimum area of surge tank. A factor of safety of 2.0 and 1.6 on the minimum area should be used in case of simple and restricted orifice surge tank respectively to secure rapid rate of damping required in practical operation.

10.3.4 Simple surge tanks

The extreme water levels in the surge tank for the conditions mentioned in para 10.3.1 are determined by integrating the equations of mass oscillations of the system.

The maximum up surge level for the case of complete load rejection may be determined from the following formula:

$$\frac{L}{2g\phi\beta^2 V_0^2} - \frac{Z_m}{\beta V_0^2} - \frac{L}{2g\phi\beta^2 V_0^2}\left[e - \frac{2g\phi}{L}\left(Z_m + \beta V_0^2\right)\right] = 0$$

where,

- ϕ = Ratio of area of surge tank to that of conduit = $\frac{As}{At}$,
- V_o = Velocity of flow in tunnel corresponding to maximum steady flow upstream of surge tank,
- Z_m = Maximum surge level above maximum reservoir level

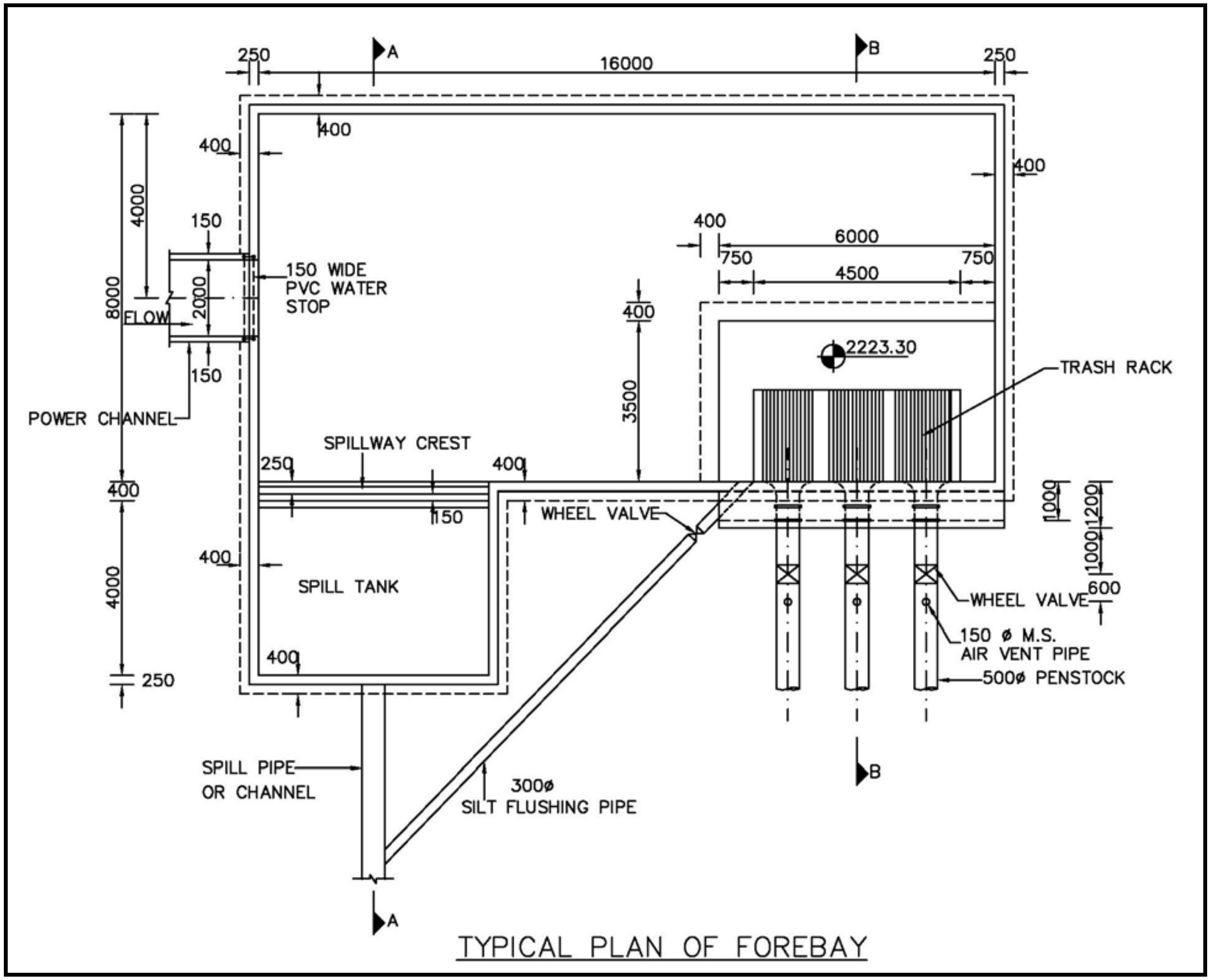
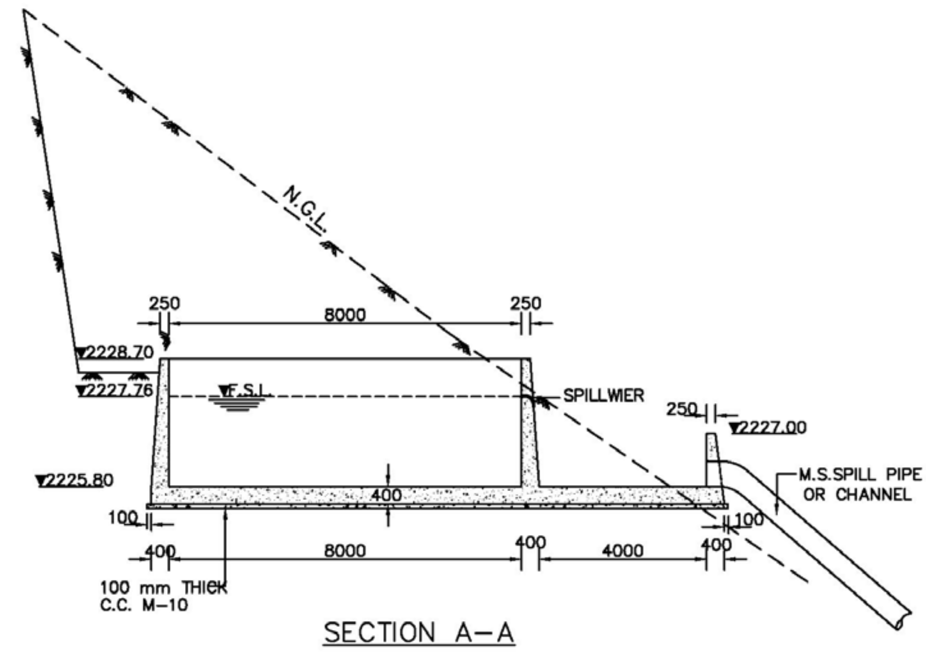


Fig. 57 (a): Typical Details of Forebay (Plan)



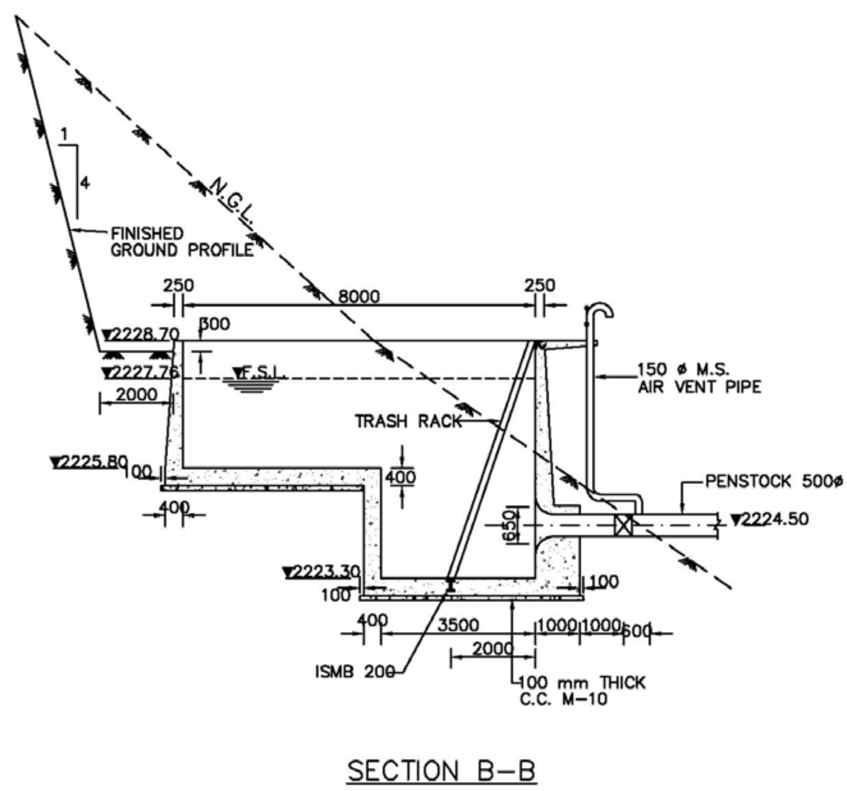


Fig. 57 (b): Typical Details of Forebay (Section)

The minimum downsurge may be approximately determined from the following formula:

$$Z_{min} = h_f \left[1 + \frac{1}{2m'\left(e^{m''}\,\pi - Cos^{-1}m'\right)} \right],$$

where,

m' = $\frac{hf}{2V_o}\sqrt{\frac{gA_s}{LA_t}}$ and $m'' = \frac{m}{\left(1-m^2\right)}$

m = damping factor = $2\beta \frac{gA_s}{LA_t}$

Z_{min} = Minimum surge level measured from the minimum reservoir level in the starting

10.3.5 Restricted orifice surge tank

The surges will depend on the resistance offered by the orifice. The resistance offered by the orifice may be calculated by the following formula:

$$h_{or} = \frac{Q_o^2}{C_d^2\, A_o^2\, 2g}$$

Where,

A_o = Area of orifice

Q_o = Maximum discharge through the turbines

C_d = Discharge coefficient of orifice usually varies between 0.6 to 0.9 depending upon the shape of orifice

The area of orifice is so chosen as to satisfy the condition given by Calame and Gaden for maximum flow as follows:

$$\frac{Z_*}{\sqrt{2}} + \frac{1}{4}h_f \le h_{o_r} \le \frac{Z_*}{\sqrt{2}} + \frac{3}{4}h_f$$

Where,

Z_* = is surge height corresponding to change in discharge neglecting friction and orifice losses, is given by following equation;

$$Z_* = V_t \sqrt{\frac{L}{g}\frac{A_t}{A_s}}$$

A_s = Area of surge tank provided

10.3.6 Maximum and minimum surge levels

The maximum and minimum surge levels can be calculated by an approximate method given by Parmakian (1960) as follows.

(a) Maximum upsurge level

The maximum upsurge height (Z_{max}) above steady state level in surge tank for total rejection of load is given by;

$$Z_{max} = 1.05 b_0^{-0.89} h_f$$

Where,

$$b_o = \frac{h_f}{Q_t}\sqrt{\frac{A_s\, g\, A_t}{L}}$$

h_f = Total head losses in HRT with minimum friction + $V_t^2 / 2g$,

Q_t = Discharge through the tunnel

and Maximum up surge level = Maximum reservoir level in upstream of HRT – h_f + Z_{max}

A free board of 1.5 to 2.0 m may be provided above the maximum up surge level to fix up the top level of surge tank.

(b) Minimum down surge level

The lowest down surge height (Z_{min}) below the steady state level corresponding MDDL is given by the following equation:

$$Z_{min} = 0.88 b_0^{-0.91} h_f$$

Where,

h_f = Total head losses with maximum friction in HRT + $V_t^2 / 2g$

and Minimum down surge level = Steady state level in surge tank with maximum friction in HRT at MDDL – Z_{min}

The bottom of surge tank may be kept 2.0 m below the lowest surge level or corresponding to required water seal above the orifice equal to 1.5 to 2.0 times the velocity head through orifice whichever is maximum. In all important works, it is recommended that the results are verified by model studies.

11.0 PENSTOCKS

Pressure conduits carrying water from forebay / surge tank to the power house are termed as penstock. Penstocks are required to bear maximum water pressure including water hammer, they are quite costly and are a very important part of the water conductor system. As such they should be designed and installed at the site with utmost care.

11.1 Types of Penstocks

(A) According to location – The penstocks may be buried inside the ground or they can be installed on surface along the natural terrain.

(i) Buried penstocks – Such penstocks are installed inside a trench and continuously supported on the soil at the bottom of the trench. After installation, the trench is backfilled with the excavated material. In this case these should be an earth cover of atleast 1.0 metre over the top of the penstock. In case the penstock pipe happen to be of steel, the steel pipe should be protected against corrosion by wrapping it with some anticorrosive sheet or the outer surface of the pipe can be covered with atleast 75 mm thick shortcrete or 150 thick PCC. The inside of the such steel penstock should be painted with long life heavy duty apoxy paint. In case of HDPE or GRP penstock pipes, they can be buried without any treatment, but the top of these pipes should also be atleast 1 m below the finished ground level. There are certain advantages and disadvantages of this type of penstocks.

Advantages:

(a) Protection against the effect of temperature and as such no expansion joints are required.
(b) Protection against freezing of conveyed water
(c) Landscape remains unchanged
(d) Immunity against rock slides, falling trees etc
(e) Protection against earthquakes
(f) Being continuously supported on soil, they are not required to be designed for beam action resulting reduced plate thickness.

Disadvantages:

(a) Need special coating against corrosion
(b) Though almost maintenance free, but if required, maintenance and repairs will be very difficult and costly

(ii) Exposed penstocks

They are laid along the profile of the existing terrain on the hill slope preferably on the ridge of the hill. They are supported on anchor blocks and saddles and bottom of the penstocks is kept atleast 0.6 m above the natural ground in order have ease in painting and to keep the penstock away from the vegetation. Penstocks comprising HDPE pipes should not be installed on surface. The advantages and disadvantages of exposed penstocks are:

Advantages:

(a) Easy inspection, maintenance and repairs
(b) Less expensive installation

Disadvantages:

(a) Full exposure to temperature variation and thus requiring expansion joints at the downstream side of each anchor block.
(b) Chances of water conveyed being frozen in cold climates
i Development of longitudinal stress due to beam action and friction at the saddle blocks
(d) Vurnerability in case of earthquake, rock falls etc.
(B) According to material being used

The penstocks may comprise steel pipes, HDPE pipes, GRP pipes or wooden staves. Presently wooden staves pipes are not in common use and as such these are not covered in this code. The merits and demerits of the other three types of penstocks are as shown in Table 14.

Table 14: Comparison of GRP Pipes, Epoxy Coated MS and HDPE Pipes

S. No.	Description	GRP Pipes	Epoxy Coated MS	HDPE
1.	Corrosion resistance	Good	To avoid inside corrosion additional epoxy coating is required. For outside corrosion protection wrapping, coating, Cathodic protection is required.	Good
2.	Maintenance	Almost maintenance free	External coating may peel off in due course of time due to rough handling and environment. Periodical maintenance is required.	Almost maintenance free
3.	Life	GRP pipes are designed for 50 years of life	Life of the pipe is 50 years	Life of HDPE pipe is 20 to 25 years depending upon the service condition
4.	Underground Application	Generally kept underground	Needs to be wrapped with some anticorrosive sheeting	Generally kept underground
5.	Mannings Rugosity Coefficient (μ)	0.01	0.011	0.01
6.	Design	Available from 200 to 3000 diameter & upto 3.0 Mpa pressure (IS:127093)	Upto 400 NB-Standard size. Above 400 NB design includes corrosion allowance	Available from 20 to 1000 mm diameter & upto 1.60 Mpa
7.	Specific Gravity	1.8-1.9	7.85	0.95
8.	Wight	Light in weight	4 times higher	Weight is higher than

S. No.	Description	GRP Pipes	Epoxy Coated MS	HDPE
			than GRP	GRP due to higher wall thickness
9.	Handling	Handling is very easy since very light in weight	Difficult, due to heavy weight	Handling is very easy
10.	Wall thickness	Lesser wall thickness due to proper fiber orientation	For a particular pressure rating, wall thickness is higher than GRP	For a particular pressure rating, wall thickness is higher than GRP
11.	Tensile strength	Hoop tensile strength = 375 Mpa	Min 400 Mpa	35 Mpa
12.	Modulus of elasticity	Hoop tensile modulus = 35 Gpa	219-240 Gpa	5 Gpa

11.2 Number of Penstocks

Either a separate penstock can be provided for each machine or a single penstock may serve more than one machine or all the machines of the power house. In case one penstock serves more than one unit, a suitable branching arrangement is required to be provided at the power house side of the penstock. The various factors affecting the choice are size, transport limitation, economy etc.

11.2.1 As the plate thickness of the penstock is directly proportional to the diameter, a single penstock of bigger size will need thicker plates or special type of steel, which will entail problems of procurement and fabrication.

11.2.2 Transport and installation problems may limit the size of penstocks.

11.2.3 While separate penstock for each unit affords a more convenient arrangement with better operational facility, in case when a single penstock serves a member of machines, if there is damage to the penstock, all the machines suffer closure. Additionally the design, fabrication and installation of branching arrangement is difficult and there will be significant head loss at the manifold.

11.2.4 From purely economic point of view, if the same discharge is passed through a number of penstocks, cost will be greater as compared to single penstock whether in terms of cost of steel or increased friction head loss. In case where the length of penstock is small, adoption of individual penstock for each machine may be more economical and vice-versa.

11.3 Specials for Penstock

A number of specials are normally required for use in the penstocks i.e.

(a) Bends
(b) Reducer pieces
(c) Branch pieces
(d) Expansion joints and dresser couplings

(e) Manholes
(f) Bulk heads
(g) Air vents and air valves, and
(h) Other miscellaneous penstock accessories like piezometric connections, flanged connections, filling connections, drainage connections.

Detailed description and design procedure of these items are available in IS:11639 (Part 3), IS:11639 (Part 1) and IS:2825.

11.4 Economical Diameter of Penstock

The economic diameter of penstock is the diameter for which the annual cost, which includes the cost of power lost due to friction and charges for amortization of construction cost, maintenance, operation, etc. is minimum. The economic diameter is calculated by evaluating the annual power loss and annuated cost of penstock and equating first derivative with respect to 'D' i.e. the penstock diameter in metres to zero. On this basis the economic diameter of an underground steel lined penstock of which the gap between steel ferrules and rock has been backfilled with concrete can be determined by the following formula:

$$D^{22/3} = \frac{2.36 \times 10^6 \times Q^3 \times n^2 \times e \times pt \times c_p}{\left[1.39 C_e + 0.6 C_c + \frac{121 H \times C_s \left(1+i\right)}{\sigma_a \times ej}\right] p},$$

Where,

D	= diameter of penstock in metres,
Q	= weighted discharge through penstock in m^3/sec,
n	= Manning's Rugosity coefficient of penstock,
e	= Overall efficiency of plant,
pt	= annual load factor,
C_p	= cost of 1 kWh of energy in rupees,
C_e	= unit cost of excavation in rupees / m^3,
C_c	= unit cost of backfilled concrete in rupees / m^3,
C_s	= Cost of steel in rupees / kg,
H	= head on penstock including, water hammer in m,
I	= percentage by which steel in penstock is over weight due to provision of stiffeners, corrosion allowance etc.,
σ_a	= allowable tensile stress in seel in kg/cm^2,
e_j	= joint efficiency of penstock, and
p	= ratio of annual fixed operation and maintenance charges to construction cost of penstock

(Note: The derivation of the above formulae is available in Appendix A of IS:11625)

Determination of economical diameter by the above criteria at the design stage is too difficult because of the uncertainty of the cost and quantities of various items, sale cost of energy, load factor etc. As such it would be enough to determine the economical diameter using empirical formulae. A few empirical formulae are given below:

(i) GS Sarkaria's formula (1958)

$$D = 0.62 \frac{P^{0.43}}{H^{0.65}},$$

Where,

D = diameter of penstock in metres,
P = rated metric horse power of turbine and
H = rated head on turbine in metres

Or D $= 3.55 \left[\frac{Q^2}{2gH} \right]^{1/4}$

Where

Q = rated discharge in penstock in cumecs

(ii) PJ Bier's formula (USBR – 1949)

$$V = 0.125\sqrt{2gH},$$

where, V = permissible velocity in penstock in m/sec.

(iii) PJ Bier's formula (USBR – 1958)

$$D = 0.176 \left(P/H\right)^{0.466}$$

(iv) G Isakassons formula

$D = Q^{0.40}$ – for steel lined penstocks at depths from 30 to 80 metres

(v) F. Fahbush's formula (Water Power Feb. 1987)

$D_C = 0.62\ Q^{0.48}$
$D_S = 1.12\ H^{-0.12} \times Q^{-.45}$,

Where,

D_C = diameter of concrete lined conduits in metres
D_s = diameter of steel lined conduits in metres

11.5 Hydraulic Design of Penstock

The hydraulic design of penstocks covers hydraulic design of penstock intake, hydraulic losses in penstock, pressure rise or pressure drop due to turbine operations, etc.

11.5.1 Hydraulic design of penstock intake

The hydraulic design of the various components of the penstock intake is similar to that already described in paras 7.4 and 7.5. IS:9761, IS:4880 and IS:11625 can also be referred in this regard.

11.5.2 Hydraulic losses in penstock

The hydraulic losses in the penstock comprise the following:

(a) Head loss at trash rack,
(b) Head loss at entrance,
(c) Friction losses,
(d) Other losses at the bends, bifurcations, transitions, valves etc.

11.5.2.1 Head loss at trash rack

It may be determined as already given in para 8.5.3.4 (b)

11.5.2.2 Head loss at entrance

It may be determined as already given in para 8.5.3.4 I

11.5.2.3 Friction losses

Friction losses may be determined by Manning's formula:

$$h_f = \frac{V^2 n^2 L}{R^{4/3}},$$

where,

h_f	= friction head loss in metres,
V	= velocity through the penstock in metres / sec.,
L	= Length of penstock in metres,
R	= hydraulic radius $\left(\frac{Area}{wetted\ \ perimeter}\right)$ in metre,
n	= Rugosity coefficient

11.5.2.4 Bend loss

The bend loss excluding friction loss for a circular conduit depends upon the shape of bends, deflection angle and ratio of radius of bend to the diameter of the conduit. The bend loss may be calculated from the following formula:

$$h_b = k_b \frac{V^2}{2g},$$

where,

h_b	= head loss due to bend in metres,
k_b	= bend loss coefficient, which can be obtained from Fig. 58
V	= velocity in the pipe in metres/ sec.

11.5.2.5 Loss due to contraction

For gradual contractions, loss of head can be computed by the following formula:

$$h_c = k_c \left[\frac{V_2^2}{2g} - \frac{V_1^2}{2g}\right],$$

where,

h_c = head loss due to contraction in metres,

v_2 = velocity in m/sec. at the contracted section,

v_1 = velocity in m/sec. at the initial section, and

k_c = contraction loss coefficient which will vary from 0.1 for a flare angle of 10^o to 0.5 for a right angle contraction. In between, the value of k_c may be assumed to vary linearly

11.5.2.6 Loss due to expansion

Expansion transitions should normally be avoided in penstocks. In case they are essential, the flare angle should not be more than 10^o and in that case the head loss can be computed from the following formula:

$$h_e = k_e \left[\frac{V_2^2}{2g} - \frac{V_1^2}{2g} \right],$$

where,

h_e = head loss due to expansion in metres,

v_1 = velocity in m/sec. at the initial section, and

v_2 = velocity in m/sec. at the expanded section,

k_e = 0.2

11.5.2.7 Loss in valves

Either butterfly or spherical valves are provided in the penstocks. These valves remain either in fully closed or fully open position. Under fully open position, the head loss in spherical valve in negligible. Head loss in fully open butterfly valve can be determined from the following formula:

$$h_v = k_v \times \frac{V^2}{2g}$$

where,

h_v = head loss due to valve in metres,

V = velocity in m/sec. corresponding gross area of valve, and

k_v = valve loss coefficient to be determined from Fig. 59

11.5.2.8 Losses in penstock branches and wyes

Head losses in branched connections are given by the following formula:

$$h_y = k_y \frac{v^2}{2g}$$

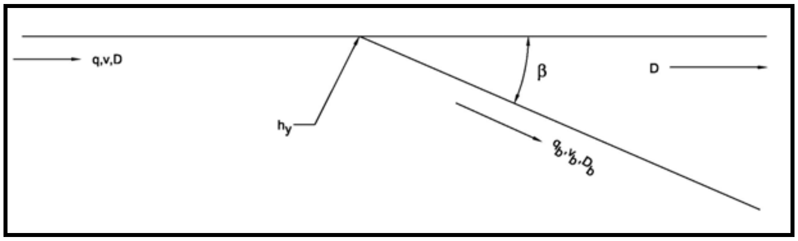
where,

h_y = head loss at junction in metres,

v = velocity before Y-junction in m/sec, and

k_y = resistance coefficient as given in Table 15.

Table 15: Head Loss in Branched Connections (Divided Flow)



Head loss at branching (h_y) = $K_y \dfrac{v^2}{2_g}$

Where,

K_y is resistance coefficient, and

q, v, D and q_b, v_b, D_b are discharge average velocity and diameter of original and branch pipes resistively.

Angle or Divergence β in Degrees	$\overbrace{\qquad\qquad}^{q_b/q=0.3}$		$\overbrace{\qquad\qquad}^{q_b/q=0.3}$		$\overbrace{\qquad\qquad}^{q_b/q=0.3}$	
	Sharp Edged	*Rounded* $r = 0.1D_b$	*Sharp Edged*	*Rounded* $r = 0.1D_b$	*Sharp Edged*	*Rounded* $r = 0.1D_b$
(1)	**(2)**	**(3)**	**(4)**	**(5)**	**(6)**	**(7)**
90	$\begin{cases} D_b = D \\ \upsilon_b = 0.3\upsilon \\ K_y = 0.85 \end{cases}$	$\begin{cases} D_b = D \\ \upsilon_b = 0.3\upsilon \\ K_y = 0.76 \end{cases}$	$\begin{cases} D_b = D \\ \upsilon_b = 0.5\upsilon \\ K_y = 0.87 \end{cases}$	$\begin{cases} D_b = D \\ \upsilon_b = 0.5\upsilon \\ K_y = 0.74 \end{cases}$	$\begin{cases} D_b = D \\ \upsilon_b = 0.7\upsilon \\ K_y = 1.00 \end{cases}$	$\begin{cases} D_b = D \\ \upsilon_b = 0.7\upsilon \\ K_y = 0.80 \end{cases}$
60	$\begin{cases} D_b = D \\ \upsilon_b = 0.3\upsilon \\ K_y = 0.7 \end{cases}$	$\begin{cases} D_b = 0.61D \\ \upsilon_b = 0.8\upsilon \\ K_y = 0.59 \end{cases}$	$\begin{cases} D_b = D \\ \upsilon_b = 0.5\upsilon \\ K_y = 0.59 \end{cases}$	$\begin{cases} D_b = 0.79D \\ \upsilon_b = 0.8\upsilon \\ K_y = 0.54 \end{cases}$	$\begin{cases} D_b = D \\ \upsilon_b = 0.7\upsilon \\ K_y = 0.57 \end{cases}$	$\begin{cases} D_b = D \\ \upsilon_b = 0.7\upsilon \\ K_y = 0.52 \end{cases}$
45	$\begin{cases} D_b = 0.58D \\ \upsilon_b = 0.9\upsilon \\ K_y = 0.43 \end{cases}$	$\begin{cases} D_b = 0.58D \\ \upsilon_b = 0.9\upsilon \\ K_y = 0.35 \end{cases}$	$\begin{cases} D_b = D \\ \upsilon_b = 0.5\upsilon \\ K_y = 0.42 \end{cases}$	$\begin{cases} D_b = 0.75D \\ \upsilon_b = 0.9\upsilon \\ K_y = 0.32 \end{cases}$	$\begin{cases} D_b = D \\ \upsilon_b = 0.7\upsilon \\ K_y = 0.34 \end{cases}$	$\begin{cases} D_b = D \\ \upsilon_b = 0.7\upsilon \\ K_y = 0.3 \end{cases}$

(Source IS: 2951 (Part II) – 1965(Reaffirmed 2003))

Note: These values are based on the experiments conducted at the Hydraulic Laboratory of the Technical University of Munich, Germany, for most efficient case.

11.5.2.9 Pressure rise and pressure drop

Pressure rise & pressure drop in the penstock due to turbine operation should be enquired from the turbine manufacturer and should be taken care of in the design of penstock ferrules and the anchor blocks.

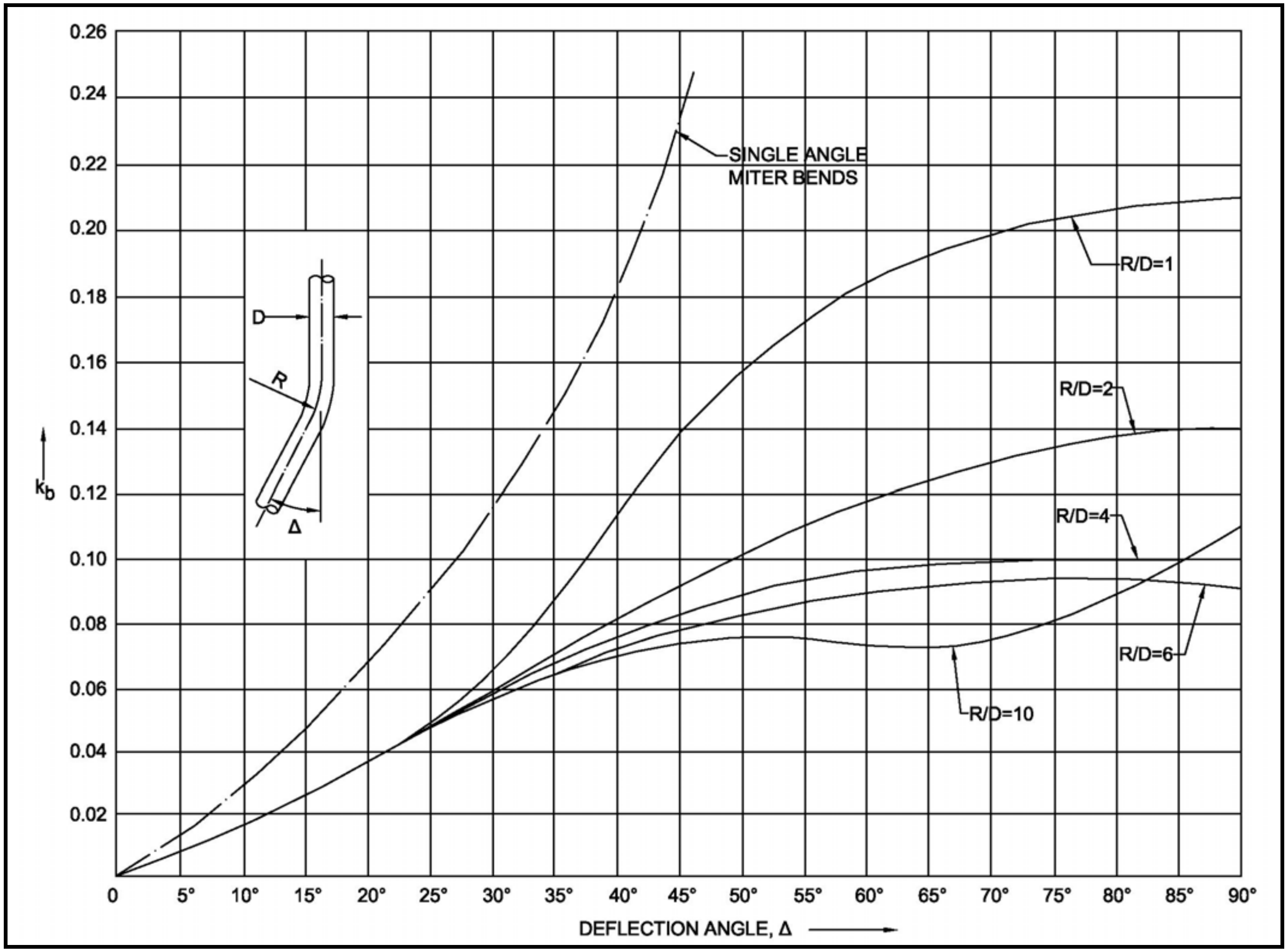


Fig. 58: Loss Coefficients for Pipe Bends of Smooth Interior
(Source IS: 2951 (Part II) – 1965 (Reaffirmed 2003))

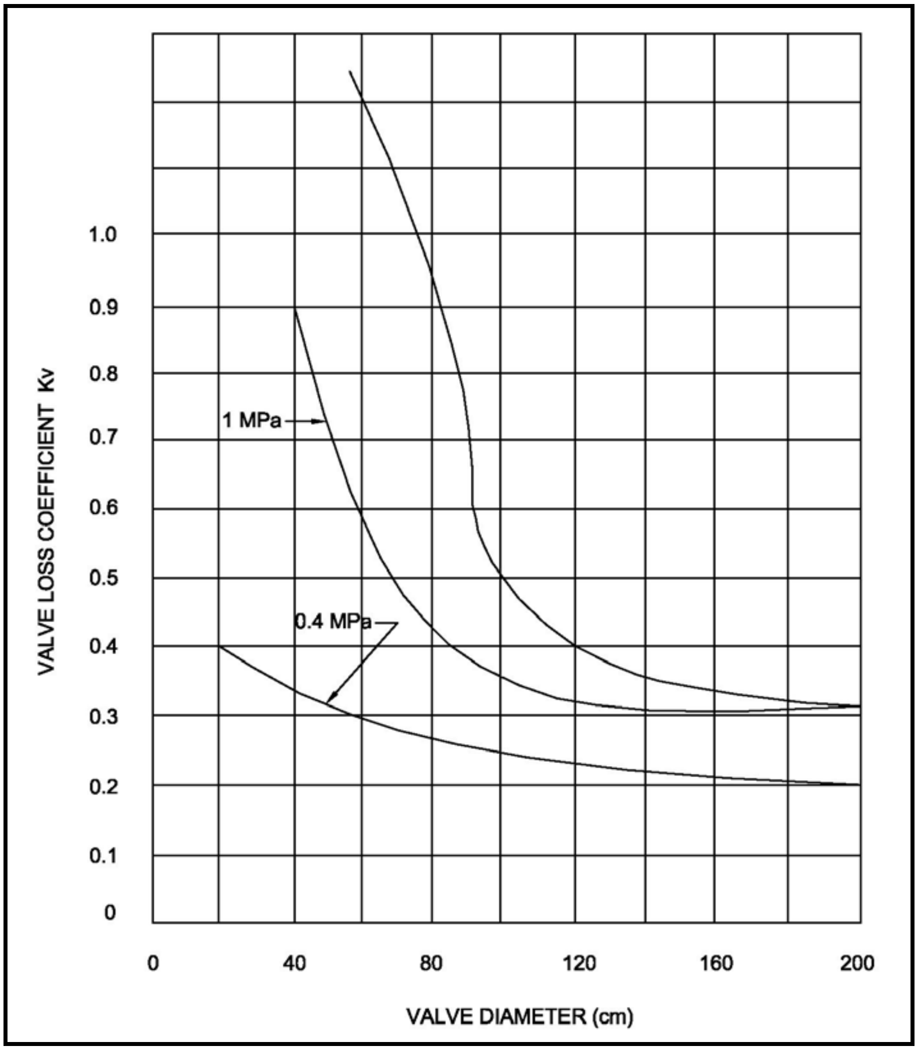


Fig. 59: Valve Loss Coefficient for Butterfly Valves (Source IS: 2951 (Part II) – 1965(Reaffirmed 2003))

11.6 Structural Design of Surface Penstocks

Surface penstocks are laid either along the natural terrain above ground or in open tunnels. These are provided with anchors at all bends and at intermediate points where the distance between any two bends requiring anchors exceeds 150 metres or so. Expansion joints are required to be provided in all surface penstocks just at the downstream side of the anchor blocks in order to absorb contraction or expansion of the penstock due to temperature changes. The expansion joints can be designed as per IS:11639 (Part 3). For design of anchor blocks IS: 5330 may be referred to. Saddle supports are provided in between the anchor blocks at such an interval as not to produce excessive bending stresses in the penstock shell. The saddle supports are generally spaced at 5 to 6 m centre to centre. The saddle supports are designed for the weight of the penstock with or without inside water, to resist an axial friction force caused by expansion and contraction of the penstock due o temperature changes and seismic forces. Friction coefficient between the penstock and the saddle surface may be taken as given in Table 16.

Table 16: Friction Coefficients for Different Surfaces

S. No.	Type of Surface	Friction Coefficient
1.	Steel on concrete	0.60
2.	Steel on concrete with asphalt roofing paper in between	0.50
3.	Steel on steel (rusty)	0.50
4.	Steel on steel (greased)	0.25
5.	Steel on steel with two layers of graphite service sheets in between	0.15
6.	Rocker support	0.15
7.	Roller support	0.10

The penstock is generally supported on the saddles at a supporting angle of 120^o. The top of the concrete saddle support is lined with a curved steel plate, the curve of which matches with the outer surface of the penstock. This curved plate has serrations on its surface with grease nipples at their ends for periodical greasing of the steel plate to reduce friction between the penstock and the saddle and to resist rusting. The general details of the concrete saddle are shown in Fig. 60.

The structural design of surface penstocks involving determination of thicknesses of the ferrules at various locations can be done with the help of IS:11639 (Part I).

11.7 Structural Design of Buried Penstocks

The structural design of buried/ embedded steel penstocks can be done in accordance with IS:11639 (Part II).

12.0 POWER HOUSE

The functions of the power house are to support and house the generating units and their accessories as well as the water passages with the purpose of (i) good performance of the plant, (ii) cost saving and (iii) easy inspection and maintenance. In the initial stage, the layout and dimensions of the power house are carried out on the basis of data of the existing power houses or as per IS:12800 Part I&III. In the final design stage, the power house should be planned and dimensioned based on the data of the supplier of the electro-mechanical equipment.

12.1 Location of Power House

The power houses related to small hydro are normally surface power houses. In case of run off the river schemes, the power house is generally located on some flat terrace near the river. The power house building should be located well beyond the hill toe so that while excavating the power house pit, the hill mass may not get disturbed. The approach level of the power house should be atleast one metre above the design flood level in the river. As such it would be ideal if level of the terrace happens to be just above the HFL. In the case of canal based scheme, the power house is normally located on a byepass channel. The actual location of a canal based power house should be so fixed that dewatering effort during power house pit excavation even when the canal is running at FSL is manageable and within economic limit.

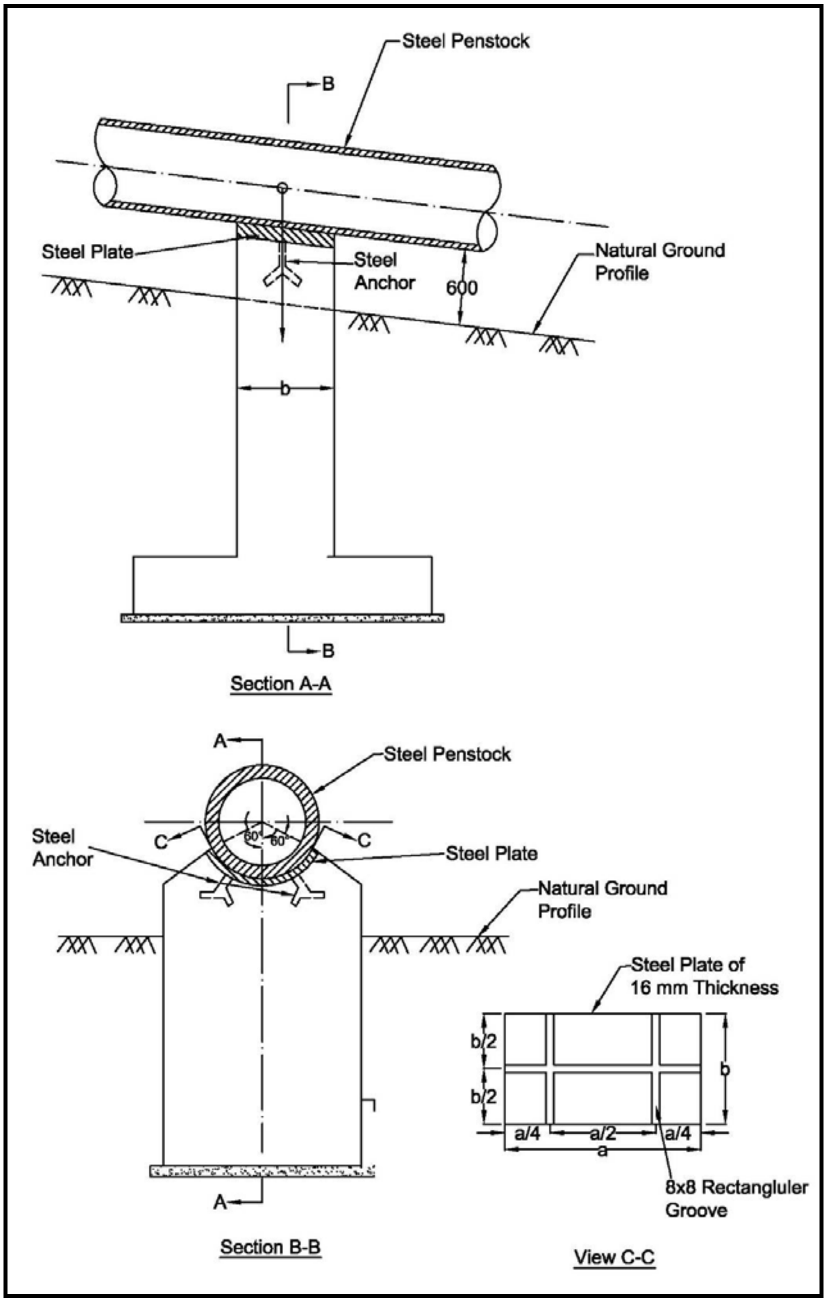


Fig. 60: General Details of CC Saddle Block

12.2 Dimensioning of Power House

The dimensions of the power house in plan are determined by the dimensions of generating equipment including governor. In the case of power houses with very low heads, the dimensions of the of the spiral case and draft tube govern the size of power house. The power house normally comprises unit bays, service bay and the control block. For a power house having 2 to 3 units, the width of the service is generally kept equal to one unit bay width. In case of a power house with very small capacity units, the unit can be planned to be overhauled outside the powerhouse building and as such there can be no service bay at all. The centre to centre spacing of the unit bays is so determined that the clearance between the turbine & generator including governor and the walls is enough for the erection and disassembly of the units. It is generally 1.5 to 2 metres. The passage way for operators should be 1 to 1.5 m. The determination of the centre line of the runner i.e. setting of turbine is very important for the power house design. For reaction turbines, the centre line of runner is determined by the parameters of the turbine with respect to minimum tail water level just downstream of the powerhouse. For this suction head which is a function of specific speed of turbine, barometric pressure at the location of the power house and prevailing temperature can be determined in accordance with the formulae and curves given in IS:12800 Part I & III. The elevation of the entry into the power house is fixed taking into account the highest flood level in the tailrace. The entry level should be atleast 1 m above the high flood level. In case of Pelton turbine, the runner is set at atleast 2 to 3 metres above the highest tail water level. The height of the power house is determined by the size of the largest piece of equipment to be lifted above the machine hall / service bay floor providing the necessary clearance required by the crane operating hook to lift equipment by attachement of the sling, the height of the EOT/HOT crane itself above the crane girder and a clearance of about 150 mm of the crane from the bottom of the roof members. The width and length of the power house are needed to be checked for adequacy considering the extreme positions of the lifting hooks of the crane.

12.3 Canal Fall based Power House

Canal fall based power houses can be of the following two categories

(i) Power house located in the main canal
(ii) Power house located adjacent to main canal in a separate bypass channel

(i) Power house located in the main canal – This category is applicable when the construction of the power house is taken alongwith the construction of the fall structure of the canal. The power station is planned adjacent to the fall structure in the canal. With this arrangement the power house is constructed adjoining the fall as an integral structure. The canal flow is diverted into the power house by widening and deepening the canal. The widening and deepening is achieved by suitable splays at the sides and deepening with gentle slopes of the order of 1(V) : 4 (H) or flatter. The intake and draft tubes are kept at suitable levels on the basis water cushion requirements for the intake and the cavitation requirements as advised by the turbine manufacturer. Gates are normally provided at the intake and at the downstream ends of draft tubes. Trash racks are provided at the upstream end of intake in order to eliminate the possibility of passing the floating debris through the turbine. The gates and trash racks are supported by piers and abutments at the sides of the turbines which are suitably extended for this very purpose. The service bay is provided at one end of the power house at a level above the FSL of the canal at the downstream side of the powerhouse. Typical plan and cross-section of the powerhouse is shown in Fig. 61.

(ii) Power house located in bypass channel – In this case while the fall remains in the main canal, the power house is constructed across a bypass channel which takes off just upstream of the fall structure and is taken parallel to the main canal and then joins the canal at the downstream of the fall. Typical layout of such scheme is shown in Fig. 62.

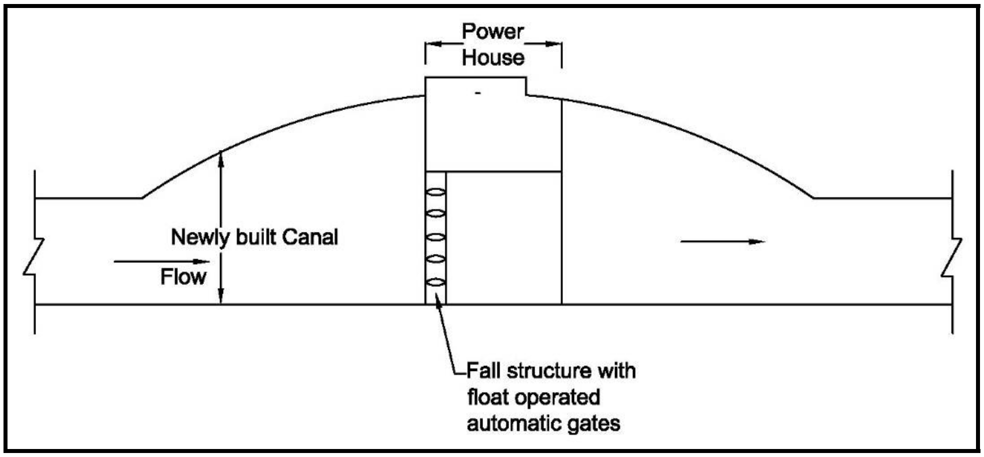


Fig. 61: Typical Layout Plan of Power House on Existing Canal

In both of the above options, automatic gates are required to be provided in the fall structure so that water can pass through them during instantaneous closure of turbines. The gates operate with a system of floats and as soon as the water rises behind the gates generally by about 150 mm, the gates automatically come into operation and release discharge into the main canal. Similarly, when the turbines start working and the water upstream of gates begins to fall, the gates corresponding begin to close.

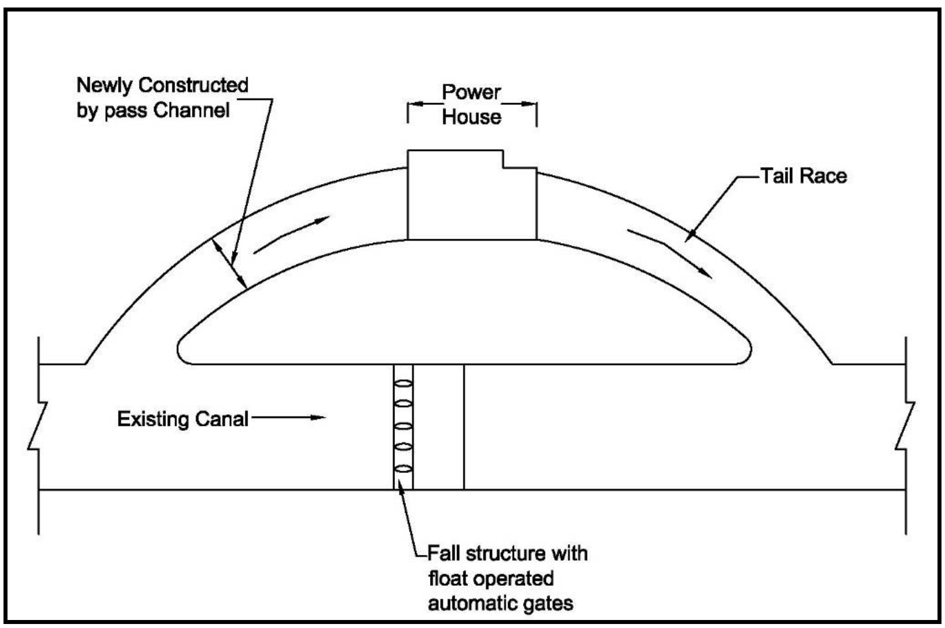


Fig. 62: Typical Layout Plan of Power House on Existing Canal

12.4 Design of Power House

The substructure and the super structure of the power house are designed for the following loads

(a) Dead load
(b) Live load

(c) Wind load
(d) Earthquake forces
(e) Crane load including transverse & longitudinal normal and striking crane surges as per IS: 875 (Part III)
(f) Penstock thrust corresponding maximum water hammer
(g) Weight of water in substructure
(h) Uplift pressure
(i) Equipment loads
(j) Upstream and downstream earth and water pressures

The various load and load combinations for design are given in IS:4247 (Part I to III). The design of a surface power house involves following steps:

(i) Preparation of general layout drawings
(ii) Stability analysis of unit bays and service bay
(iii) Design of substructure
(iv) Design of superstructure

12.4.1 Preparation of general layout drawings

General layout drawings of the power house are prepared mainly on the basis of the requirements of and the data of electro-mechanical equipment supplied by the machine supplier. Firstly the level of the centre line of the runner should be fixed on the basis of the suction head required by the machine manufacturer and the minimum water level of the tailrace which is obtained by developing the gauge discharge curve just at the downstream of the power house. In case of a horizontal machine, there is normally one floor of the machine hall, where the turbines and generators are installed. In case of vertical machine, there may be two to three floors depending upon the requirement of auxiliary equipment. Each of floor should have a minimum head room of 4.0 metres. The level of the service bay is fixed atleast 1.0 metre above the high flood level. The service bay can be located either on the left or right side of the machine hall on the basis of the location of the approach road as all the power house equipment is first unloaded and erected in the service bay. Normally gates are required to close the draft tube outlet openings for segregating the machines from downstream water at the time of erection, maintenance and repairs. These gates are required to be operated from the draft tube deck of the machine hall by either a gantry crane, monorail or fixed rope drum hoists for each gates. The level of this deck is also normally kept atleast 1.0 metre above the maximum downstream water level. The control block can either be kept on the upstream or downstream side of the machine hall or in a separate building adjoining the power house. The main unit transformers can either be kept on the draft tube or at a location upstream of the power house or in the switchyard itself. A drainage and dewatering of sump is normally required to collect all the drainage of the power house & dewatering the draft tubes for maintenance and repairs. The sump can be located either at the end of the machine hall, or somewhere in the middle, in the erection bay or below the draft tube deck. The portion of the power house below the service bay floor is termed as substructure and that above the erection bay which houses the overhead gantry crane required for erection and maintenance of machines is known as superstructure. The height of the superstructure is fixed on the basis of the clearances required to lift and carry the biggest size machine part through out the power house. The upstream and downstream columns, that support the gantry crane and the roof above it, are known as gantry columns. The locations of these columns in plan is fixed on the following basis:

(i) The upstream and downstream gantry column should be just in front of each other so that the roof trusses could be installed on their top.
(ii) They should be free from the locations of penstocks on the upstream side and draft tube openings on the downstream side.
(iii) As per as possible the gantry column should be in the line of the counter forts to support the substructure walls of the power house on the upstream side and the draft tube gate piers on the downstream side.

The roof of the power house normally comprises steel trusses covered with G.C. sheeting. Various layouts of the power houses having Francis, Pelton, Bulb and Tubular turbines are shown in Figs. 63 to 66. Sometimes in case of very low head power houses such as canal based power houses both intake and draft tube gates are eliminated by keeping the turbine runner above upstream water level. In such cases, the turbine in run by creating siphoning action with the help of a suction pump. An arrangement of such a power house is shown in Fig. 67. Normally transverse contraction / expansion joints are provided between service bay and the machine hall as the structures accommodating the machines and service bay are quite different. In such case twin gantry columns are provided at the joint. In some cases, when the length of the machine hall is too long i.e. more than 35 metres or so, additional joints should be provided in the machine hall also. In big power houses, normally with vertical machines, the transverse joints with twin columns are provided between each machine or at the junction of every two machines. Normally contraction joints are provided in the substructure and expansion joints in the superstructure. These joints are required to be water proofed with the help of double PVC water stops upto atleast 1.0 metre above the ground level and single water stop upto roof level.

12.4.2 Stability analysis of unit bays and service bay

The stability of the unit bays between transverse joints and the erection bay should be checked for the worst combination of the loads and forces mentioned in clause 12.4. When on one hand dead loads of the structure and those of the permanently embedded machine parts will always be there, the other loads and forces may or may not be there. The structures should be checked for the following:

(i) Bearing pressures – Bearing pressures should be within allowable limits both with or without uplift.
(ii) Shear Friction Factor – Shear friction factor ‘Q’ between the structure and the foundation strata should be determined from the following equation:

$$Q = \frac{CA + f.\sum V}{\sum H},$$

where,

C = cohesion or unit shearing strength in kg/cm^2 applied on the area under compression,
A = Area of base under compression in cm^2,
f = friction coefficient or tan ϕ, where $\phi$is the angle of internal friction of the foundation material in submerged condition,
$\sum V$ = algebraic sum of normal forces and uplift in kg., and
$\sum H$ = algebraic sum of the forces parallel to the plane of sliding in kg.

The value of shear friction factor should not be less than the following;

Condition	Completed	Construction Stage
For normal loading condition	3.5	2.5
For extra-ordinary loading conditions	2.0	1.1

(iii) Floatation Factor – Floatation of the structure may take place on account of uplift forces caused by water. The factor of safety against floatation ‘F’ is given by the following equation:

$$F = \frac{\sum V}{U},$$

where,

$\sum V$ = algebraic sum all vertical loads except uplift, and

U = Uplift

The value F should be atleast 1.1 in construction or extraordinary loading condition and 1.25 in completed condition.

12.4.3 Design of substructure

The primary function of the substructure is to distribute the superimposed loads to the foundation strata in such a way that the foundation reactions are within permissible limits. The shape of the substructure is quite complex and the acting loads and forces are also not simple. As such an exact analysis of the structure is not possible. The structure is thus divided into simpler sub-divisions which are supposed to have similar structural ehavior. For simplification, the analysis is made simpler by splitting it in two parts – one in transverse direction (along the direction fo flow), and the other in the longitudinal direction (perpendicular to the direction fo flow).

12.4.3.1 Analysis of substructure in transverse direction

In transverse direction, the superimposed loads vary considerably from one point to the other. Besides vertical loads, there are considerable horizontal forces like penstock thrust, wind, earth and water pressure etc. The effect these loads in to vary the foundation reaction from upstream to downstream. In transverse direction, the structure can be designed as a continuous footing which is supposed to be an endless contilever. The moment at any point of the raft being the algebraic sum of the moments of the loads on the top of the substructure and that of the soil reaction from the bottom. Similarly the shear force at any section can be determined by the algebraic sum of all the loads from the top and reaction from bottom on one side of the section.

12.4.3.2 Analysis of substructure in longitudinal direction

The analysis in longitudinal direction is required in order to check and provide for the structural strength of draft tube substructure in order that it may support the equipment and other superimposed loads in-spite of various cavities. The various sections in longitudinal direction can be designed as box sections or U–sections for the loads from top, sides and reactions from bottom.

12.4.4 Design of superstructure

The superstructure of a power house normally consists of a frame work of columns and longitudinal beams which support the gantry crane, panel walls and the roof, which normally consists of steel trusses covered by GC sheeting. The two ends of the trusses are generally hinged at the top of the upstream and downstream gantry columns. The transverse forces are usually assumed to be resisted by individual portal frames comprising upstream and downstream gantry columns and the roof truss. In longitudinal direction, the forces are resisted by multi-storeyed frame work consisting of columns and beams on each side of the building. The centre to centre spacing of the longitudinal beams is kept from 5 to 6 metres and one longitudinal beam is necessarily provided at the top of the columns. Besides the upstream and downstream gantry columns, there may be, in some cases, intermediate columns and beams in the lower portion to support lower floors.

13.0 TAILRACE

After passing through the turbines, the water is discharged into the stream through a short channel called tailrace. In case of reaction turbines, the tailrace should be designed to ensure minimum tail water level required to maintain safe suction head for smooth operation of the turbine. In case of impulse turbine, maximum water level in the tailrace should not rise to a limit that it interferes with the turbine runner. For power houses running on relatively high discharges, common tailrace is provided for all the units as shown in Fig. 60 but for installations running on relatively low discharges and high head, normally separate tailrace channels are provided for individual machines. Normally and specially in vertical machines the bottom of the draft tube exit end happens to be much below the river bed level, where the power house discharge is required to be dropped. In such a case a reverse slope not steeper than 4H to IV is provided in the upstream reaches of the tailrace, though there are examples where this reverse slope is of the order of 0.5 H to 1 V.

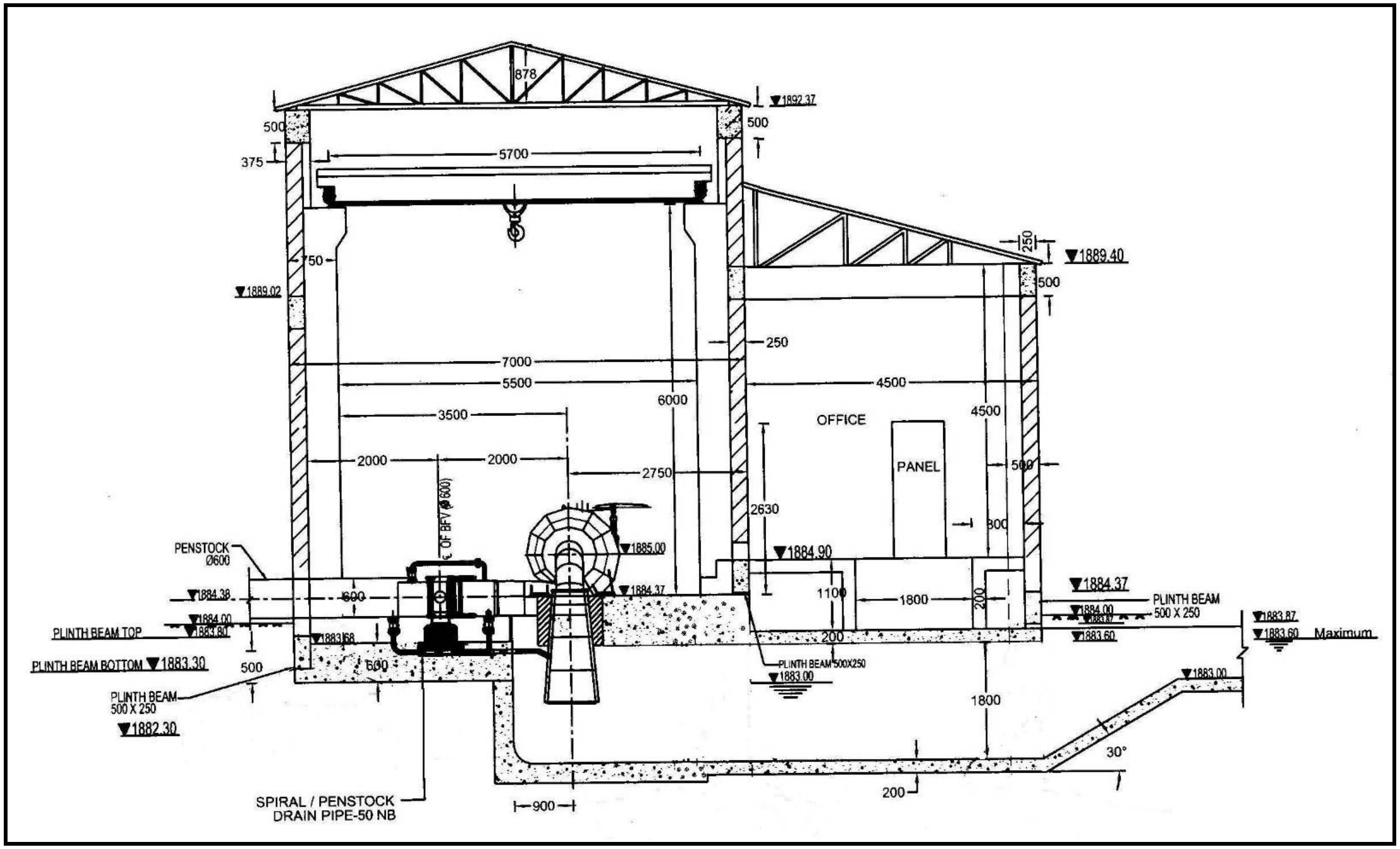


Fig. 63: Typical Transverse Section of Power House with Horizontal Francis Turbine

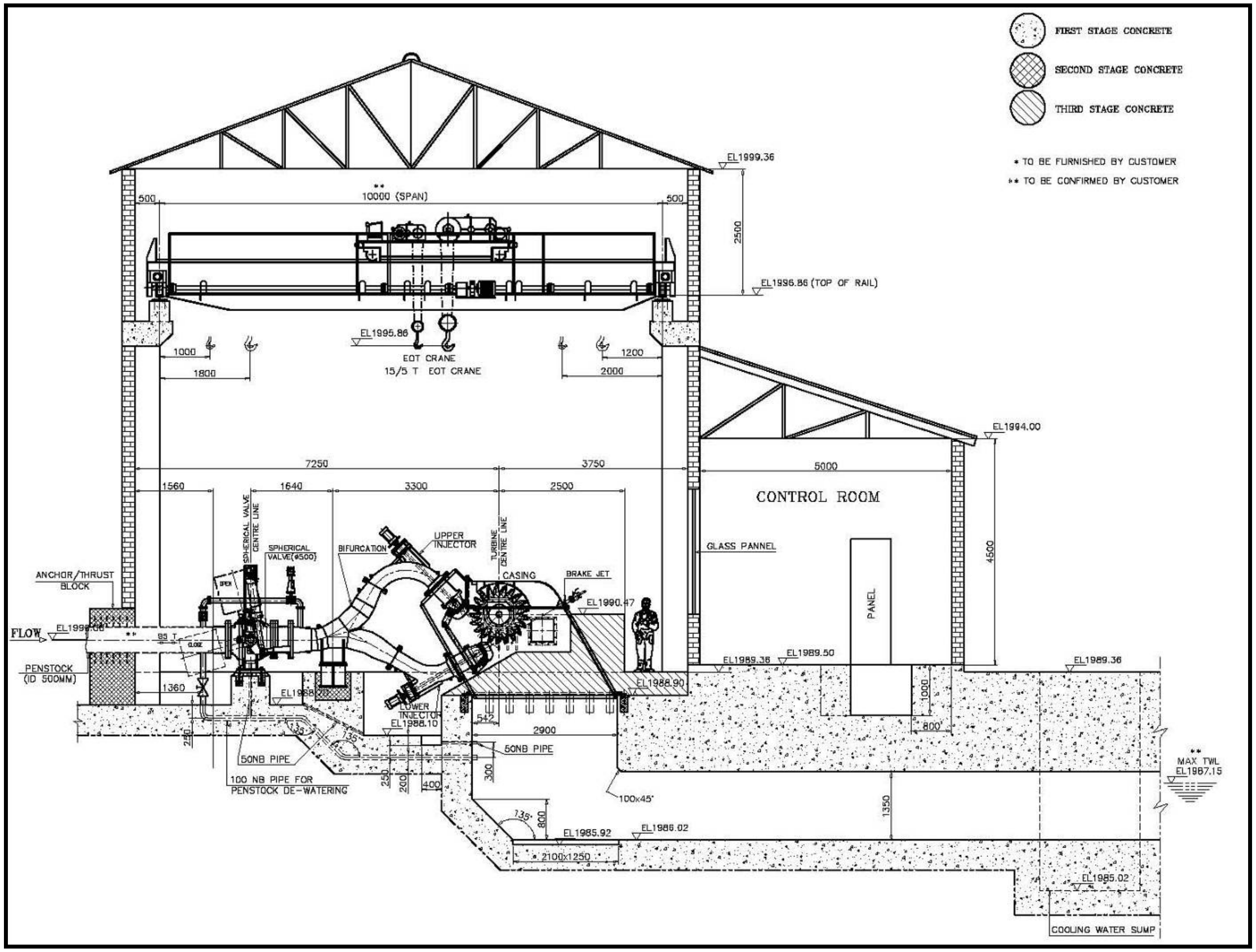


Fig. 64: Typical Transverse Section of Power House with Horizontal Pelton Turbine

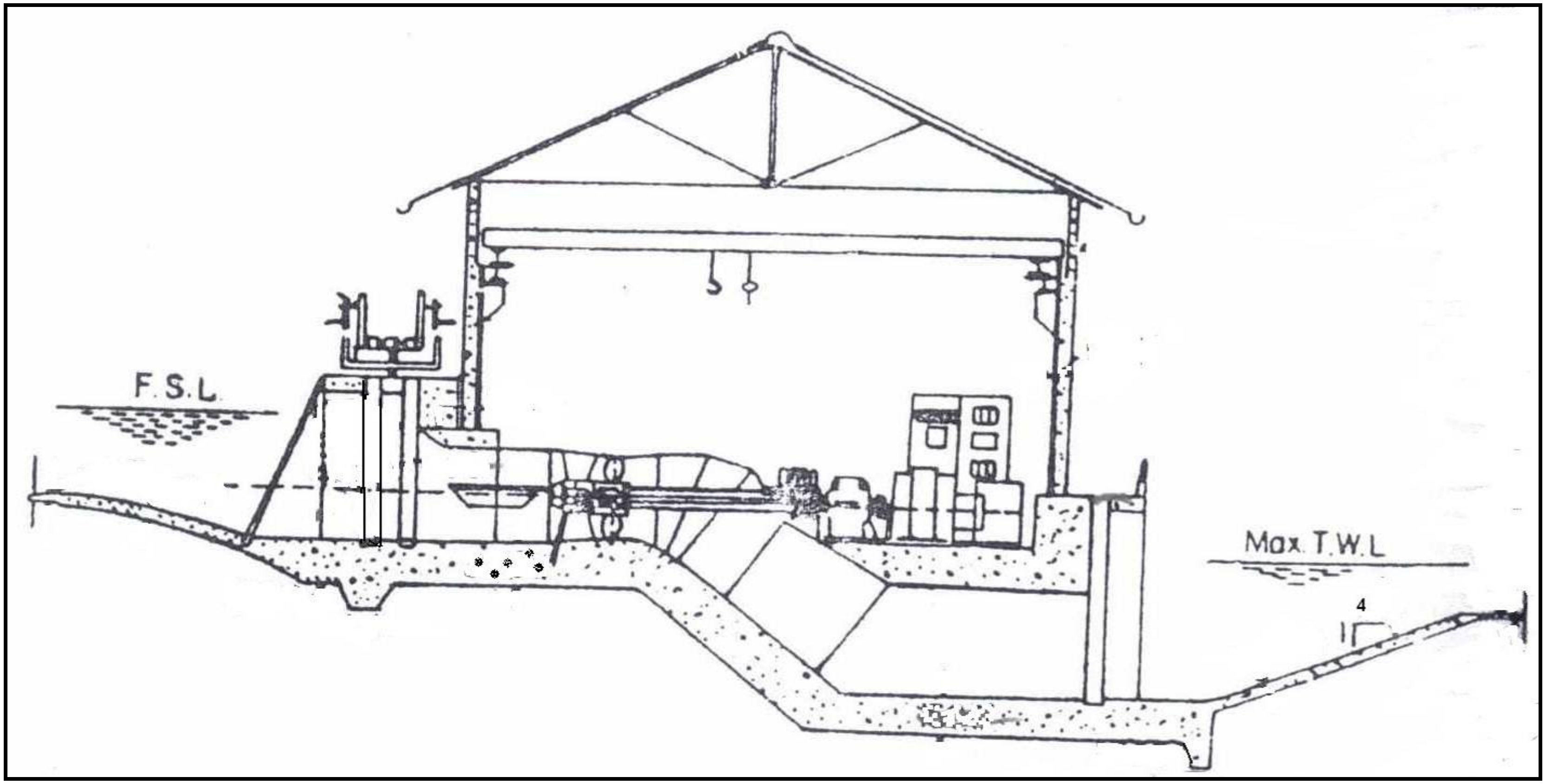


Fig. 65: Typical Transverse Section of Power House with Tubular Turbine

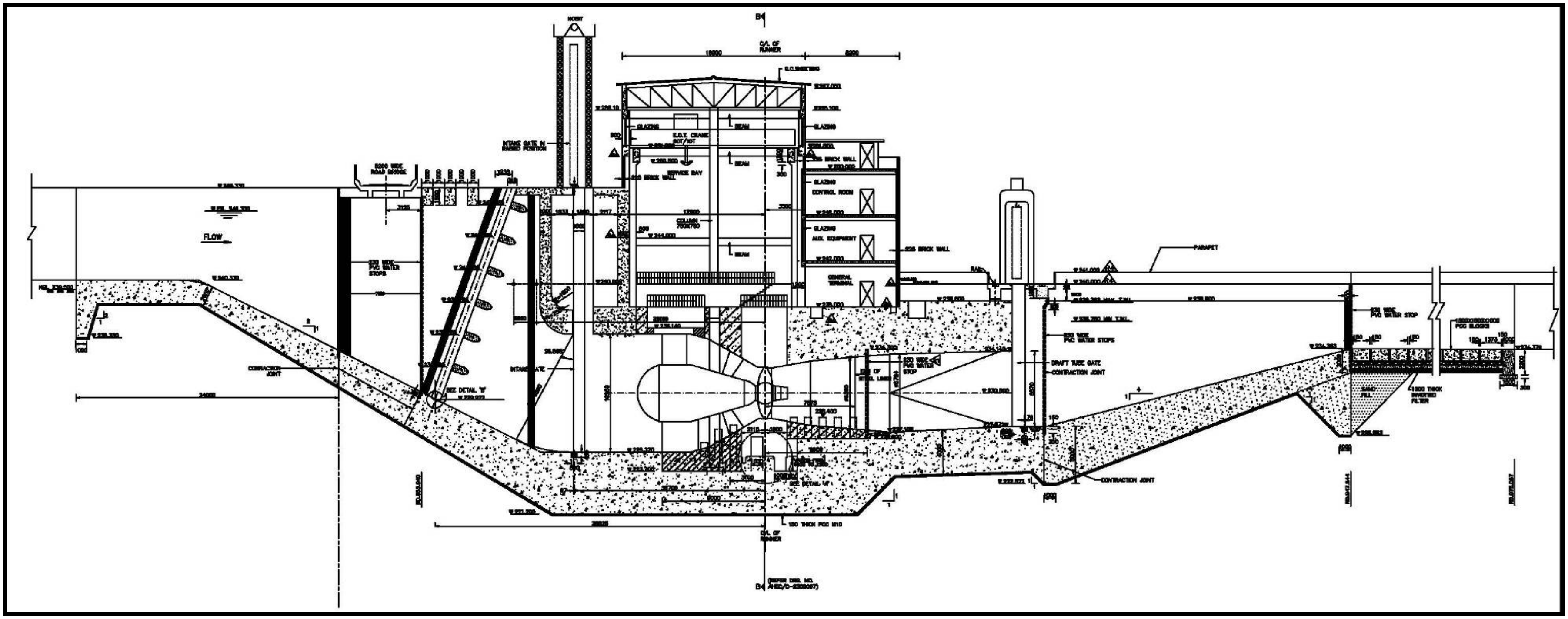


Fig. 66: Typical Transverse Section of Power House with Bulb Turbine

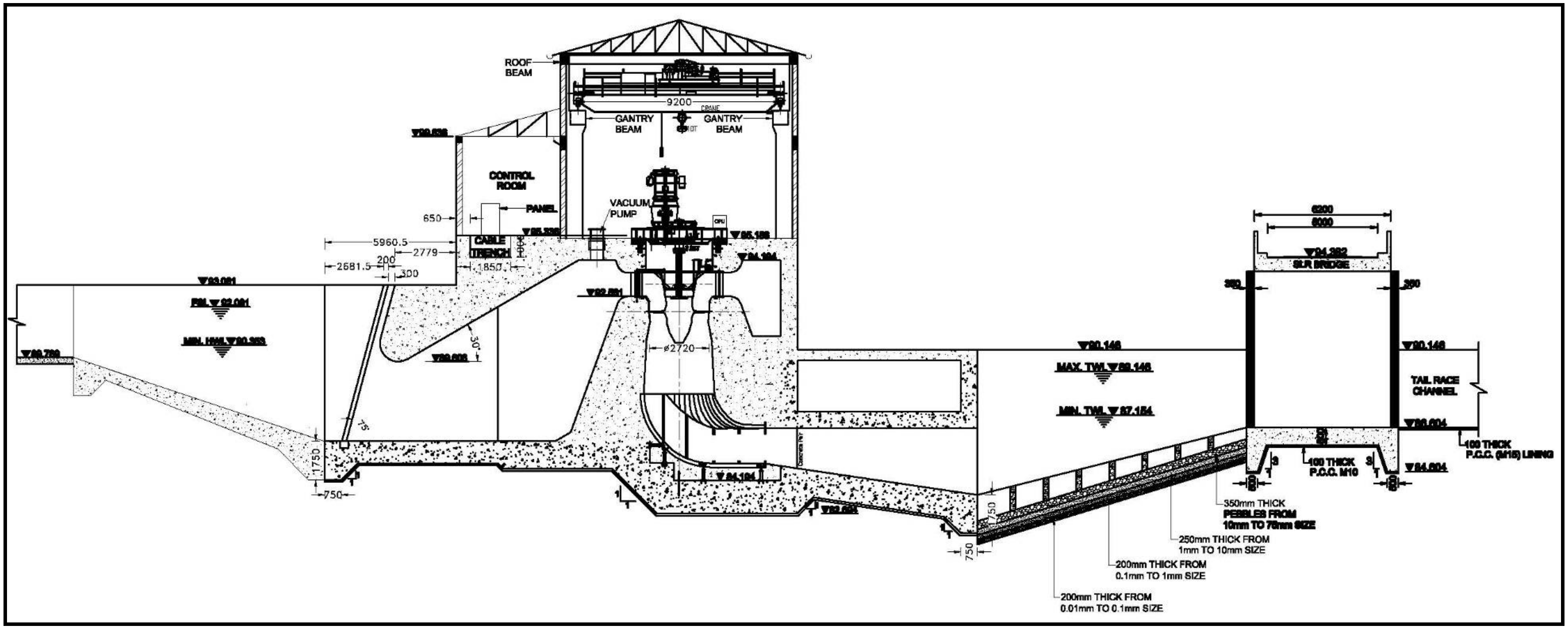


Fig. 67: Typical Transverse Section of Power House with Syphon Kaplan Turbine

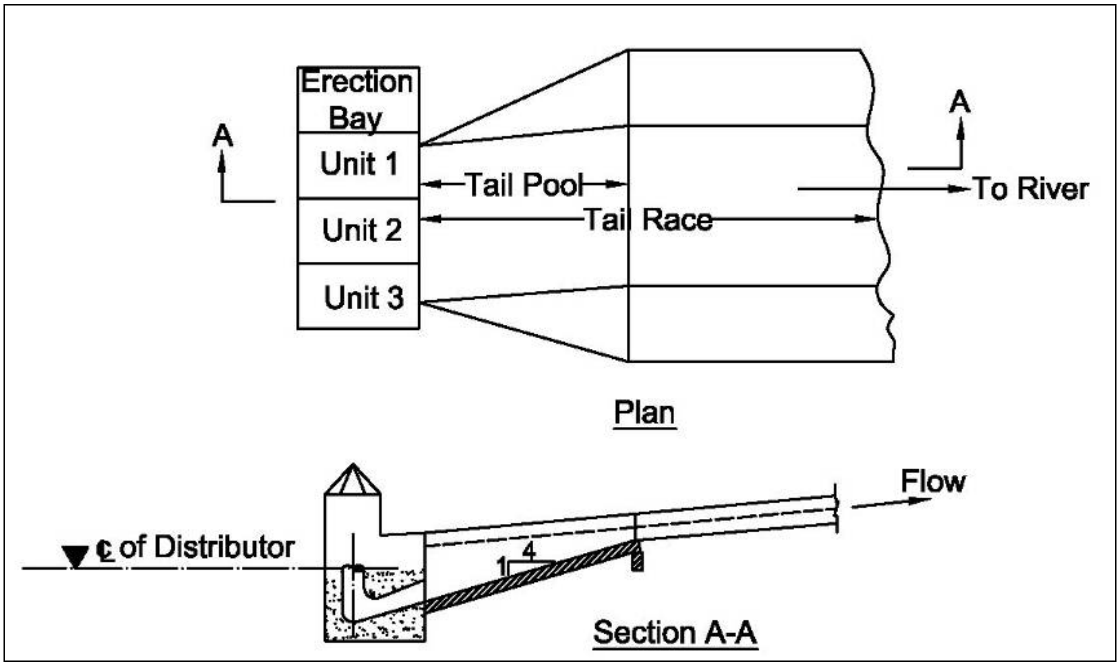


Fig. 68: Typical tailrace for high Discharge Installation

ANNEXURE-I

ROCK TUNNELLING QUALITY INDEX, Q

(Ref. The Development of Rock Engineering by Evert Hoek)

On the basis of an evaluation of a large number of case histories of underground excavations, Barton of the Norwegian Geotechnical Institute proposed a Tunnelling Quality Index (Q) for the determination of rock mass characteristics and tunnel support requirements. The numerical value of index Q varies on a logarithmic scale from 0.001 to a maximum of 1,000 and is defined by:

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SFR} \qquad \text{(A1)}$$

where,

RQD is the Rock Quality Designation
J_n is the joint set number
J_r is the joint roughness number
J_a is the joint alteration number
J_w is the joint water reduction factor
SRF is the stress reduction factor

In explaining the meaning of the parameters used to determine the value of Q, Barton offer the following comments:

The first quotient (RQD/J_n), representing the structure of the rock mass, is a crude measure of the block or particle size, with the two extreme values (100/05 and 10/20) differing by a factor of 400. If the quotient is interpreted in units of centimeters, the extreme 'particle sizes' of 200 to 0.5 cm are seen to be crude but fairly realistic approximations. Probably the largest blocks should be several time this size and the smallest fragments less than half the size. (Clay particles are of course excluded).

The second quotient (J_r/J_a) represents the roughness and frictional characteristics of the joint walls or filling materials. This quotient is weighted in favour of rough, unaltered joints in direct contact. It is to be expected that such surfaces will be close to peak strength, that they will dilate strongly when sheared, and they will therefore be especially favourable to tunnel stability.

When rock joints have thin clay mineral coatings and fillings, the strength is reduced significantly. Nevertheless, rock wall contact after small shear displacements have occurred may be a very important factor for preserving the excavation from ultimate failure.

Where no rock wall contact exists, the conditions are extremely unfavourable to tunnel stability. The 'friction angles' (given in Table A2) are a little below the residual strength values for most clays, and are possibly down-graded by the fact that these clay bands or fillings may tend to consolidate during shear, at least if normal consolidation or if softening and swelling has occurred. The swelling pressure of montmorillonite may also be a factor here.

The third quotient (J_w/SRF) consists of two stress parameters. SRF is a measure of: 1) lossening load in the case of an excavation through shear zones and clay bearing rock, (2) rock stress in competent rock, and (3) squeezing loads in plastic incompetent rocks. It can be regarded as a total stress parameter. The parameter J_w is a measure of water pressure, which has an adverse effect on the shear strength of joints due to a reduction in effective normal stress. Water may, in addition, cause softening and possible out-wash in the case of clay-filled joints. It has proved impossible to combine these two parameters in terms of inter-block effective stress, because paradoxically a high value of effective normal stress may sometimes signify less stable conditions than a low value, despite the higher shear strength. The quotient (J_w/SRF) is a complicated empirical factor describing the 'active stress'.

It appears that the rock tunneling quality Q can now be considered to be a function of only three parameters which are crude measures of:

1. Block size (RQD/Jn)
2. Inter-block shear strength (Jr/Ja)
3. Active stress (Jw/SRF)

Undoubtedly, there are several other parameters which could be added to improve the accuracy of the classification system. One of these would be the joint orientation. Although many case records include the necessary information on structural orientation in relation to excavation axis, it was not found to be the important general parameter that might be expected. Part of the reason for this may be that the orientations of many types of excavations can be and normally are, adjusted to avoid the maximum effect of unfavourably oriented major joints. However, this choice is not available in the case of tunnels, and more than half the case records were in this category. The parameters Jn, Jr and Ja appear to play a more important role than orientation, because the number of joint sets determines the degree of freedom for block movement (if any), and the frictional dilatational characteristics can vary more than the down-dip gravitational components of unfavourably oriented joints. If joint orientations had been included the classification would have been less general, and its essential simplicity lost.

Table A2 (After Barton 1974) gives the classification of individual parameters used to obtain the Tunnelling Quality Index Q for a rock mass.

In relating the value of the index Q to the stability and support requirements of underground excavations, Barton (1974) defined an additional parameter which they called the Equivalent Dimension, De, of the excavation. This dimension is obtained by dividing the span, diameter or wall height of the excavation by a quantity called the Excavation Support Ratio, ESR. Hence:

$$D_e = \frac{\text{Excavation span, diameter or height } (m)}{\text{Excavation support ratio ESR}} \qquad \text{(A2)}$$

The value of ESR is related to the intended use of the excavation and to the degree of security which is demanded of the support system installed to maintain the stability of the excavation. Barton (1974) suggest these values as per Table A1:

Table A1: ESR Values for Various Excavation Category

Excavation Category		ESR
A.	Temporary mine openings	3-5
B.	Permanent mine openings, water tunnels for hydro power (excluding high pressure penstocks), pilot tunnels, drifts and headings for large excavations	1.6
C.	Storage rooms, water treatment plants, minor road and railway tunnels, surge chambers, access tunnels	1.3
D.	Power stations, major road and railway tunnels, civil defence chambers, portal intersections.	1.0
E.	Underground nuclear power stations, railway stations, sports and public facilities, factories.	0.8

Table A2: Classification of individual parameters used in the Tunnelling Quality Index Q

DESCRIPTION		VALUE	NOTES
1.	**Rock Quality Designation**	**RQD**	
A.	Very Poor	0-25	1. Where RQD is reported or measured as ≤ 10 (including 0), a nominal value of 10 is used to evaluate Q. 2. RQD intervals of 5, i.e. 100, 95, 90 etc. are sufficiently accurate.
B.	Poor	25-50	
C.	Fair	50-75	
D.	Good	75-90	
E.	Excellent	90-100	
2.	**Joint Set Number**	**J_n**	
A.	Massive, no or few joints	0.5-1.0	
B.	One joint set	2	
C.	One joint set plus random	3	
D.	Two joint sets	4	
E.	Two joint sets plus random	6	
F.	Three joint sets	9	1. For intersections use (3.0 x J_n) 2. For portals use (2.0 x J_n)
G.	Three joints sets plus random	12	
H.	Four or more joint sets, random, heavily jointed, 'sugar cube', etc.	15	
J.	Crushed rock, earthlike	20	
3.	**Joint Roughness Number**	**J_r**	1. Add 1.0 if the mean spacing of the relevant joint set is greater than 3 m 2. J_r = 0.5 can be used for planar, slickensided joints having lineations, provided that the lineations are oriented for minimum strength.
	a. Rock wall contact		
	b. Rock wall contact before 10 cm shear		
A.	Discontinuous joints	4	
B.	Rough and irregular, undulating	3	
C.	Smooth undulating	2	
D.	Slickensided undulating	1.5	
E.	Rough or irregular, planar	1.5	
F.	Smooth, planar	1.0	
G.	Slickensided, planar	0.5	
	c. No rock wall contact when sheared		

DESCRIPTION		VALUE		NOTES
H.	Zones containing clay minerals thick enough to prevent rock wall contact	1.0 (nominal)		
J.	Sandy, gravely or crushed zone thick enough to prevent rock wall contact	1.0 (nominal)		
4.	**Joint Alternation Number**	**J_a**	**ϕr degrees (approx.)**	1. Values of ϕr, the residual friction angle, are intended as an approximate guide to the mineralogical properties of the alteration products, if present.
	a. Rock wall contact			
A.	Tightly healed, hard, non-softening, impermeable filling	0.75		
B.	Unaltered joint walls, surface staining only	1.0	25-35	
C.	Slightly altered joints walls, non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc.	2.0	25-30	
D.	Silty, or sandy-clay coatings, small clay-friction (non-softening)	3.0	20-25	
E.	Softening or low-friction clay mineral coatings, i.e. kaolinite, mica.Also chlorite, talc, gypsum and graphite etc., and small quantities of swelling clays. (Discontinuous coatings, 1-2 mm or less)	4.0	8-16	
	b. Rock wall contact before 10 cm shear			
F.	Sandy particles, clay-free, disintegrating rock etc.	4.0	25-30	
G.	Strongly over-consolidated, non-softening clay mineral fillings (continuous < 5 mm thick)	6.0	16-24	
H.	Medium or low over-consolidation, non-softening clay mineral fillings (continuous < 5 mm thick)	8.0	12-16	
J.	Swelling clay fillings, i.e. montmorillonite, (continuous < 5 mm thick). Values of J_a depend on percent of swelling clay-size particles, and access to	8.0-12.0	6-12	

DESCRIPTION		VALUE		NOTES
	water.			
	c.No rock wall contact when sheared			
K.	Zones or bands of disintegrated or crushed	6.0		
L.	Rock and clay (see G, H and J for clay) M condition	8.0		
M.	Rock & clay	8.0-12.0	6-24	
N.	Zones or bands of silty or sandy-clay, small clay friction, non-softening	5.0		
O.	Thick continuous zones or bands of clay	10.0-13.0		
P & R.	See G, H and J for clay conditions	6.0-24.0		
5.	**Joint Water Reduction**	**J_w**	**Approx. water pressure (kgf/cm^2)**	
A.	Dry excavation or minor inflow i.e. < 5 l/m locally	1.0	< 1.0	1. Factors C to F are crude estimates; increase J_w if drainage installed. 2. Special problems caused by ice formation are not considered
B.	Medium inflow or pressure, occasional outwash of joint fillings	0.66	1.0-2.5	
C.	Large inflow or high pressure in competent rock with unfilled joints	0.5	2.5-10.0	
D.	Large inflow or high pressure	0.33	2.5-10.0	
E.	Exceptionally high inflow or pressure at blasting, decaying with time	0.2-0.1	>10	
F.	Exceptionally high inflow or pressure	0.1-0.05	>10	
6.	**Stress Reduction Factor**	SRF		
	a. Weekness zones intersecting excavation , which may cause lossening of rock mass when tunnel is excavated			
A.	Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock any	10.0		1. Reduce these values of SRF by 25-50% but only if the relevant shear zones influence do not

	DESCRIPTION	VALUE			NOTES
	depth				intersect the excavation 2. For strongly anisotropic virgin stress field (if measured): when $5 \le \sigma_1/\sigma_3 \le 10$, reduce σ_c to 0.8 σ_c and σ_t to 0.6 σ_c and 0.6 σ_t, where σ_c = unconfined compressive strength, and σ_t = tensile strength (point load) and σ_1 and σ_3 are the major and minor principal stresses. 3. Few case records available where depth of crown below surface is less than span width. Suggest SRF increase from 2.5 to 5 for such cases (see H).
B.	Single weakness zones containing clay, or chemically distegrated rock (excavation depth <50 m)	5.0			
C.	Single weakness zones containing clay, or chemically distegrated rock (excavation depth >50 m)	2.5			
D.	Multiple shear zones in competent rock (clay free), loose surrounding rock (any depth)	7.5			
E.	Single shear zone in competent rock (clay free) (depth of excavation < 50 m)	5.0			
F.	Single shear zone in competent rock (clay free) (depth of excavation > 50 m)	2.5			
G.	Loose open joints, heavily jointed or 'sugar cube', (any depth)	5.0			
	b. Competent rock, rock stress problems			**SRF**	
		σ_c/σ_1	$\sigma_t\sigma_1$		
H.	Low stress, near surface	>200	>13		
J.	Medium stress	200-10	13-0.66	1.0	
K.	High stress, very tight structure (usually favourable to stability, may be unfavourable to wall stability)	10-5	0.66-0.33	0.5-2	
L.	Mild rockburst (massive rock)	5-2.5	0.33-0.16	5-10	
M.	Heavy rockbrust (massive rock)	<2.5	<0.16	10-20	
	c. Squeezing rock, plastic flow of incompetent rock under influence of high rock pressure				

DESCRIPTION		VALUE			NOTES
N.	Mild squeezing rock pressure			5-10	
O.	Heavy squeezing rock pressure			10-20	
	d. Swelling rock, chemical swelling activity depending on presence of water				
P.	Mild swelling rock pressure			5-10	
R.	Heavy swelling rock pressure			10-15	

ADDITIONAL NOTES ON THE USE OF THESE TABLES

When making estimates of the rock mass quality (Q), the following guidelines should be followed in addition to the notes listed in the tables:

1. When borehole core is unavailable, RQD can be estimated from the number of joints per unit volume, in which the number of joint per metre for each joint set are added. A simple relationship can be used to convert this number to RQD for the case of clay free rock masses: RQD = 115-3.3 J_v (approx.), where J_v = total number of joints per m^3 ($0<RQD < 100$ for $35 > J_v > 4.5$).
2. The parameter J_n representing the number of joint sets will often be affected by foliation, schistosity, slaty cleavage or bedding etc. If strongly developed, these parallel 'joints' should obviously be counted as a complete joint set. However, if there are few 'joints' visible, or if only occasional breaks in the core are due to these features, then it will be more appropriate to count them as 'random' joints when evaluating J_n.
3. The parameters J_r and J_a (representing shear strength) should be relevant to the weakest significant joint set or clay filled discontinuity in the given zone. However, if the joint set or discountinuity with the minimum value of J_r / J_a is favourably oriented for stability, then a second, less favourably oriented joint set or discontinuity may sometimes be more significant, and its higher value of J_r/J_a should be used when evaluating Q. The value of J_r/J_a should in fact relate to the surface most likely to allow failure to initiate.
4. When a rock mass contains clay, the factor SRF appropriate to loosening loads should be evaluated. In such cases the strength of the intact rock is of little interest. However, when jointing is minimal and clay is completely absent, the strength of the intact rock may become the weakest link, and the stability will then depend on the ratio rock-stress/rock-strength. A strongly anisotropic stress field is unfavourable for stability and is roughly accounted for as in note 2 in the table for stress reduction factor evaluation.
5. The compressive and tensile strengths (σ_c and σ_t) of the intact rock should be evaluated in the saturated condition if this is appropriate to the present and future is situ conditions. A very conservative estimate of he strength should be made for those rocks that deteriorate when exposed to moist or saturated conditions.

The equivalent dimension, De, plotted against the value of Q, is used to define a number of support categories in a chart published in the original paper by Barton (1974). This chart has recently been updated by Grimstad and Barton (1993) to reflect the increasing use of steel fibre reinforced shotcrete in underground excavation support. Fig. A is reproduced from this updated chart.

Loset (1992) suggests that, for rocks with $4 < Q < 30$, blasting damage will result in the creation of new 'joints' with a consequent local reduction in the value of q for the rock surrounding the excavation. He suggests that this can be accounted for by reducing the RQD value for the blast damaged zone.

Barton (1980) provide additional empirical equations for the determination of rockbolt length, maximum unsupported spans and roof support pressures to supplement the support recommendations published in the original 1974 paper. For selection of the support measures using the Q-system, the designer should consult the original paper by Barton et.al. or the book by Hoek and Brown.

$$\text{Rockbolt length (L)} = 2.0 + 0.15\frac{Excavation\ width}{ESR} \qquad \text{(A3)}$$

$$\text{The maximum unsupported span} = 2\ ESR\ Q^{0.4} \qquad \text{(A4)}$$

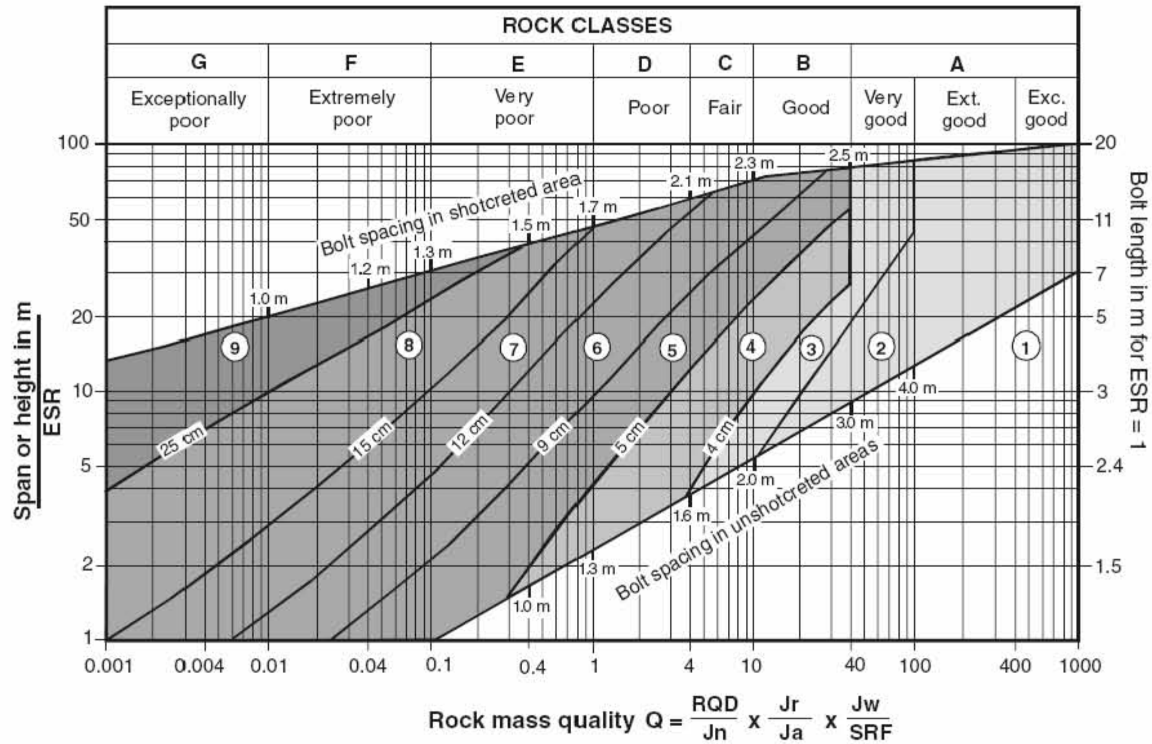
Based upon analyses of case records, Grimstad and Barton (1993) suggest that the relationship between the value of Q and the permanent roof support pressure P_{roof} is estimated from:

$$P_{roof} = \frac{2\sqrt{Jn}\,Q^{-\frac{1}{3}}}{3Jr} \text{ for less than three joint sets}$$

$$= \frac{2Q^{-0.22}}{J_r} \text{ for three or more joint sets} \qquad \text{(A5)}$$

Rock mass quality (Q_{wall}) for computing wall pressure is given by:

$$Q_{wall} = 5.0\ Q \quad \text{for } Q>10$$
$$= 2.3\ Q \quad \text{for } 0.1 < Q < 10 \text{ and}$$
$$= 1.0\ Q \text{ for } Q < 0.10 \qquad \text{(A6)}$$



REINFORCEMENT CATEGORIES:

1) Unsupported
2) Spot bolting
3) Systematic bolting
4) Systematic bolting, (and unreinforced shotcrete, 4 - 10 cm)
5) Fibre reinforced shotcrete and bolting, 5 - 9 cm
6) Fibre reinforced shotcrete and bolting, 9 - 12 cm
7) Fibre reinforced shotcrete and bolting, 12 - 15 cm
8) Fibre reinforced shotcrete, > 15 cm, reinforced ribs of shotcrete and bolting
9) Cast concrete lining

Fig. A1: Estimated Support Categories Based on the Tunneling Quality Index Q
(Source: The Development of Rock Engineering by Evert Hoek)

Design Steps of Support Requirements

The various steps involved in arriving at the rock support requirement using Barton's classification are as given below:

(i) Assess the various rock parameters and their corresponding values using Table A2 and find out the value of rock mass quality 'Q' by equation A1
(ii) Workout the value of equivalent dimension (D_e) using equation A2.
(iii) Find out roof support pressure using equation A5
(iv) Depending upon the value of 'Q' and D_e select the suitable support requirements from Fig. A1.