STANDARDS/MANUALS/ GUIDELINES FOR SMALL HYDRO DEVELOPMENT

Civil Works –

Guidelines for Hydraulic Design of SHP Projects

Sponsor:

Ministry of New and Renewable Energy
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CONTENTS

1. GUIDELINES FOR HYDRAULIC DESIGN OF SMALL HYDRO PLANTS

This section provides standards and guidelines on the design of the water conductor system. This system includes; head works and intake, feeder canal, desilter (if required), power canal or alternative conveyance structures (culverts, pipelines, tunnels, etc), forebay tank, penstock and surge tank (if required) up to the entry of the turbine, tailrace canal below the turbine and related ancillary works.

1.1 HYDRAULIC DESIGN OF HEAD WORKS

In general head works are composed of three structural components, diversion dam, intake and bed load sluice. The functions of the head works are:

Diversion of the required project flow from the river into the water conductor system.
Control of sediment.
Flood handling.
Typically a head pond reservoir is formed upstream of the head works. This reservoir may be used to provide daily pondage in support of peaking operation or to provide the control volume necessary for turbine operation in the water level control mode. This latter case would apply where the penstock draws its water directly from the head pond. Sufficient volume must be provided to support these functions.

There are three types of head works that are widely used on mini and small hydro projects, as below:
Lateral intake head works
Trench intake head works
Reservoir / canal intakes
Each type will be discussed in turn.

1.1.1 Head Works with Lateral Intakes (Small Hydro)

Head works with lateral intakes are typically applied on rivers transporting significant amounts of sediment as bed load and in suspension.
The functional objectives are:
To divert bed-load away from the intake and flush downstream of the dam (the bed load flushing system should be operable in both continuous and intermittent modes).
To decant relatively clean surface water into the intake.
To arrest floating debris at intake trashracks for removal by manual raking.
To safely discharge the design flood without causing unacceptable upstream flooding.

The following site features promote favourable hydraulic conditions and should be considered during site selection:

The intake should be located on the outside of a river bend (towards the end of the bend) to benefit from the spiral current in the river that moves clean surface water towards the intake and bed load away from the intake towards the centre of the river.

The intake should be located at the head of a steeper section of the river. This will promote removal of material flushed through the dam which may otherwise accumulate downstream of the flushing channel and impair its function.

Satisfactory foundation conditions.

Ideal site conditions are rare, thus design will require compromises between hydraulic requirements and constraints of site geology, accessibility etc. The following guidelines assume head works are located on a straight reach of a river. For important projects or unusual sites hydraulic model studies are recommended.

A step by step design approach is recommended and design parameters are suggested for guidance in design and layout studies. Typical layouts are shown in Figures 2.2.1 to 2.2.3.

1.1.2 Data Required for design.

The following data are required for design:

Site hydrology report as stipulated in Section 1.3 of this Standard giving:

- Q_p (plant flow)
- Q_{100} (design flood flow, small hydro)
- Q_{10} (design flood flow, mini hydro)
- C_w (data on suspended sediment loads)
- H-Q Curves (W.L. rating curves at diversion dam)

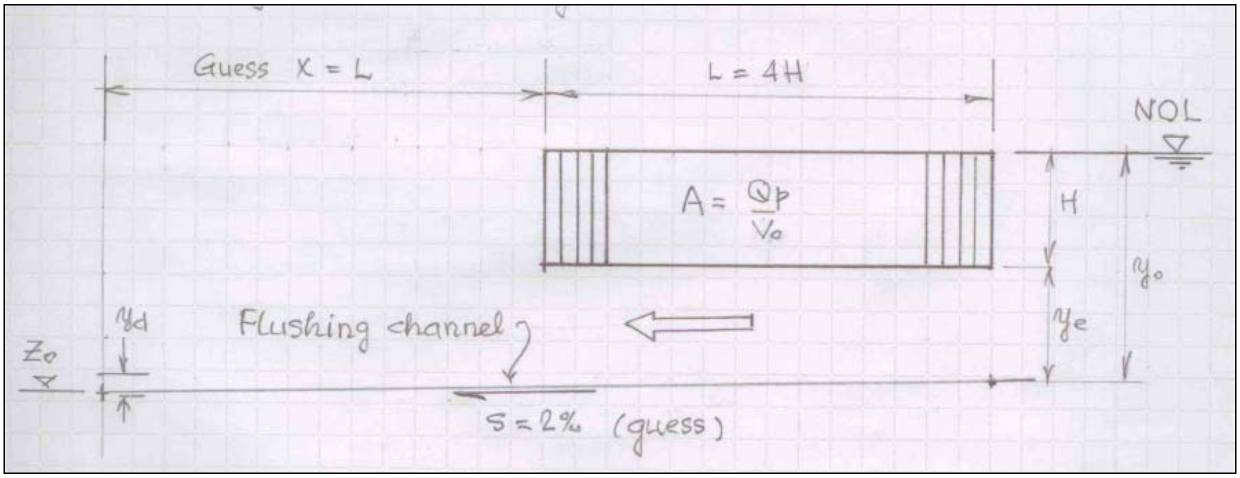
Topographic mapping of the site including river bathymetry covering all head works structure sites.

Site geology report.

1.1.3 Site Selection:

Selection of the head works site is a practical decision which involves weighing of several factors including hydraulic desiderata (Section 2.2.1/1.0), head optimization, foundation conditions, accessibility and constructability factors. Given the importance of intake design to the overall performance of the plant it is recommended that an experienced hydraulic engineer be consulted during studies on head works layout.

1.1.4 Determination of Key Elevations:



For the illustrative example: $Q_d = 10.0\ m^3/s$

Determine $V_0 = 0.5\ Q^{0.2}$ (= 0.792, say 0.80 m/s)

$A_0 = Q \div V_0$ (= 12.5 m^2)

$H = \sqrt{\frac{A_0}{4}}$ (= 1.77 m, say 1.80 m)

Assume $L = 4H$ (= 7.08 m, say 7.0 m)

y_e = greater of 0.5 y_o or 1.5 m (= 1.80m)

$y_d = L.S$ (= 0.28 m)

$NOL = Z_0 + y_e + y_d + H$

NOL = 97.5 + 1.80 + 0.28 + 1.80 (=101.38m, say 101.50 m)

Sill = NOL – H (= 99.7m)

Crest of weir or head pond NOL = 101.5 m

Height of weir = 4.0 m

These initial key elevations are preliminary and may have to be adjusted later as the design evolves.

1.1.5 Head Works Layout

The entry to the intake should be aligned with the river bank to provide smooth approach conditions and minimize the occurrence of undesirable swirl. A guide wall acting as a transition between the river bank and the structure will usually be required. Intake hydraulics are enhanced if the intake face is slightly tilted into the flow. The orientation of the intake face depends on river bank topography, for straight river reaches the recommended values for tilt vary from 10^o to 30^o depending on the author. When this angle becomes too large the intake will attract excessive amounts of sediment and floating debris. It is also recommended that the main river current be maintained in mid stream beyond the intake and bed load flushing structures. This is usually achieved by releasing major flows via large spillway gate/s in mid-river. Deflection of the main current by the flushing channel sill and divider wall has a further beneficial effect of establishing (to some degree) curved flow paths away from the intake. This effect is more pronounced when spillway flow is much greater than flow entering the intake and flushing channel (Nigam pages 357 – 365). A spur on the bank opposite the head regulator may enhance the curvilinear flow but its design cannot be done reliably without a hydraulic model study.

For development of the head work plan, it is recommended that the following parameters be used for layout:
Axis of intake should be between 105° to axis of river (tilt = 15°).
Divider wall should be completely “cover” 80% to 100% of the intake.
Assume flushing flow equal to twice project flow then estimate the width and height of the flushing gate from orifice formula,:

$$Q_f = 0.6 \times 0.5W^2\sqrt{2gy_0}$$

Where: Q_f = flushing flow
W = gate width
H = gate height (= 0.5W)
Y_o = normal flow depth as shown in 2.2.1.1/2.0

Sill should be straight and perpendicular to the flow direction.

In the sample design (Fig.2.2.1.1) the axis of the intake = 105° & $Q_f = 2.0\times10.0 = 20 m^3/s$

$$\therefore\ 20.0 = 0.6 \times 0.5\ W^2\sqrt{2 \times 9.8 \times 5.6}$$

$\therefore$ W = 2.8 m (say 3.0m) and H = 1.5 m.

1.1.6 Flood Handling, MFL and Number of Gates.

For small hydro a simple overflow diversion weir would be the preferred option if flood surcharge would not cause unacceptable upstream flooding. For purpose of illustration, the following design data are assumed (see Figure2.2.2):

Design flood, $Q_{100} = 175\ m^3/s$
A review of reservoir topography indicated that over bank flooding would occur if the flood water level exceeded 103.0 m. Select this water level as the MFL. This provides a flood surcharge (S) of 1.20 m.
Assume weir coefficients as below:
Gate, C_w = 1.70 - - - sill on slab at river bottom.
Weir, C_w = 1.80 - - - - - - -ogee profile.
Assume gate W/H ratio = 1:2
H = 4.0 m $\therefore$ W = 4.8 (say 5.0 m)
MFL. = NOL + 1.50 (= 103.0m)
$Q_{gate} = C_w.\ W.\ (MFL - Z_S)^{1..5}$
$Q_{weir} = C_w.L_w.S^{1.5}$
Capacity check for MFL = 103.0 m

No. of Gates	Length of Overflow Section (m)	Q_G (m^3/s)	Q_W (m^3/s)	Q_T (m^3/s)
0	35.0	0.0	82.8	82.8
1	29.0	109.6	68.6	178.2 >175

Therefore one gate is sufficient.

Where:

MFL = Maximum flood level (m)
NOL = Normal operating level (m)
S = flood surcharge above NOL (m)
W = width of gate (m)
H = height of gate (m)
Z_S = elevation of gate sill (m)
C_w = weir coefficient ($m^{0.5}s^{-1}$)
Q_G, Q_W, Q_T = gate, weir and total flows

The flow capacity of the sediment flushing gate may also be included in calculating flood handling capacity.

1.1.7 Diversion Dam and Spillway

Hydraulic Design of Weir:

For weir having design heads of 0.5 m or greater use of an ogee profile should be considered. A suitable methodology is provided in *"Design of Small Dams", Chapter 9- Section 9.8* (USBR 1987). Stability and stress design should be done in accordance with Section 2.3.3 of this Standard.

Plains Rivers:

Stability of structures founded on alluvial foundations typical of plains rivers, is governed by the magnitude of the exit gradient. The critical gradient is approximately 1.0 and shall be reduced by the following safety factors:

Types of foundation	Safety factor	Allowable Exit Gradient
Shingles / cobbles	5	0.20
Coarse sand	6	0.167
Fine sand	7	0.143

Also barrages or weirs on plains rivers will normally require stilling basins to dissipate the energy from the fall across the dam before the water can be returned safely to the river.

Design of diversion weirs and barrages on permeable foundation should follow *IS 6966 (Part 1).* Sample calculations in Chapter 12 of *"Fundamentals of Irrigation Engineering" (Bharat Singh,* 1983) explain determination of uplift pressure distributions and exit gradients. Further details on structural aspects of design are given in Section 2.3.3 of this Standard.

***Mountain Rivers*:**

Bedrock is usually found at relatively shallow depths in mountain rivers permitting head works structures to be founded on rock. Also the beds of mountain rivers are often boulder paved and are much more resistance to erosion than plains rivers. Therefore there may be no need for a stilling basin. The engineer may consider impact blocks on the downstream apron or simply provide an angled lip at the downstream end of the apron to “flip” the flow away from the downstream end of the apron. A cut-off wall to bed rock of suitable depth should also be provided for added protection against undermining by scour. The head works structures would be designed as gravity structures with enough mass to resist flotation. For low structures height less than 2.0 m anchors into sound bedrock may be used as the prime stabilization element in dam design. Stability and stress design shall be in accordance with requirements of Section 2.3.3 of this Standard.

1.1.8 Sediment Flushing Channel

The following approach is recommended for design of the flushing channel:

Select flushing channel flow capacity (Q_f) = 2×Q_p

Estimate maximum size of sediment entering the pocket from site data or from transport capacity of approaching flow and velocity. In case of diversion weir without gates assume sediment accumulation to be level with the weir crest. (Assume continuous flushing with 3×Q_p entering the pocket, for this calculation).

Establish entrance sill elevation and channel slope assuming an intermittent flushing mode (intake closed) with $Q_s = 2Q_p$, critical flow at the sill, supercritical flow downstream ($F_N \geq 1.20$) and a reservoir operating level 0.5m below NOL. Determine slope of channel to provide the required scouring velocity, using the following formula which incorporates a safety factor of 1.5:

$$i = 1.50\, i_o$$

$$i_0 = 0.44 \frac{d^{9/7}}{q^{6/7}}$$

Where: i_o = critical scouring velocity

d = sediment size

q = flow per unit width (m^3/s per m)

Verify that flow through pocket in continuous flushing mode ($Q_s = 3Q_s$) will be sub critical, if not lower entrance sill elevation further.

Determine height of gate and gate opening based on depth of flow at gate location and corresponding gate width. Increase the above theoretical gate height by 0.25 m to ensure unrestricted open channel flow through the gate for intermittent flushing mode and a flushing flow of 2 Q_p. For initial design a width to height ratio of 2:1 for the flushing gate is suggested.

1.1.9 Intake/Head Regulator:

In intake provides a transition between the river and the feeder canal. The main design objectives are to exclude bed-load and floating debris and to minimize head losses. The following parameters are recommended:

Approach velocity at intake entrance (on gross area)

$$V_e = 0.5 Q_p{}^{0.20}\ m/s$$

For trashracks that are manually cleaned, V should not exceed 1.0 m/s.

Convergence of side walls 2.5:1 with rate of increase in velocity not exceeding 0.5 m/s per linear m.

- Height of sill above floor of flushing channel (y_e) = greater of 1.5m or 50% flow depth.
- The floor of the transition should be sloped down as required to join the invert of the feeder canal. Check that the flow velocity in the transition is adequate to prevent deposition in the transition area. If sediment loads are very high consider installing a vortex silt ejector at the downstream end of the transition.
- Provide coarse trashracks to guard entry to the head gate. The trashrack would be designed to step floating debris such as trees, branches, wood on other floating objects. A clear spacing of 150 mm between bars is recommended. Trashrack detailed design should be in accordance with *IS 11388.*
- The invert of the feeder canal shall be determined taking into consideration head losses through the trashrack and form losses through the structure. Friction losses can be omitted as they are negligible:

  Calculate form losses as: $H_L = 0.3 \dfrac{V_2^2}{2g}$

  Where: V_2 = velocity at downstream end of contraction.

  Calculate trashrack losses as:

  $$H_L = K_f \left(\frac{t}{b}\right)^{4/3} .Sin\beta. \frac{V^2}{2g}$$

  Where: K_f = head loss factor (= 2.42 assuming rectangular bars)

  T = thickness of bars (mm)

  B = clear bar spacing (mm)

  β = angle of inclination to horizontal (degrees)

  V = approach velocity (m/s)

1.1.10 References on Lateral Intakes and Diversion Weirs.

IS Standards Cited:

IS 6966 (Part 1)	*Hydraulic Design of Barrages and Weirs - Guidelines*
IS 11388	*Recommendations for Design of Trashracks for Intakes*
USBR (1987)	*Design of Small Dams*
Singh, Bharat	*Fundamentals of Irrigation Engineering* Nem Chand & Bros.-Roorkee (1983)
Nigam, P.S.	*Handbook of Hydroelectric Engineering* (Second edition) …..pages 357 to 365 Nem Chand & Bros.- Roorkee (1985)

1.1.11 Other References:

Bucher and Krumdieck	*Guidelines for the Design of Intake Structures for Small Hydro Schemes*; Hydro '88/3rd International Conference on Small Hydro, Cancun – Mexico.
Bouvard, M.	*Mobile Barrages and Intakes on Sediment Transporting Rivers*; IAHR Monograph, A.A. Balkema – Rotterdam (1992)
Razvan, E.	*River Intakes and Diversion Dams* Elsevier, Amsterdam (1988)

1.2. SEMI PERMANENT HEADWORKS (MINI HYDRO)

For mini hydro projects the need to minimize capital cost of the head works is of prime importance. This issue poses the greatest challenge where the head works have to be constructed on alluvial foundations. This challenge is addressed by adoption of less rigorous standards and the application of simplified designs adapted to the skills available in remote areas. A typical layout is shown in Figure 2.2.3.

1.2.1 Design Parameters

Hydraulic design should be based on the following design criteria:

Plant flow (Q_p) $= Q_T + Q_D$

Where:

Q_T = total turbine flow (m^3/s)
Q_D = desilter flushing flow (= 0.20 Q_T) m^3/s
Q_{FC} = feeder canal flow (= 1.20 Q_T) m^3/s
Q_F = gravel flushing flow (= 2.0 Q_P)

Spillway design flow (SDF) = Q_{10}

Where: Q_{10} = flood peak flow with ten year return period.

1.2.2 Layout

Intake approach velocity = 1.0 m/s
Regulator gate W/H = 2
Flushing channel depth (H_D) = 2H + W/3
Flushing channel minimum width = 1.0 m
Assumed flushing gate W/H = 2, determine H from orifice equation, as below:

$$Q_f = 0.53 \times 2H^2 . \sqrt{2gY_1}$$

$Y_1 = H_D$ for design condition

Where:

W = width of gate (m)
H = height of gate (m)
Y_i = upstream depth (m)

H_D = depth of flushing channel (m)

Select the next largest manufactures standard gate size above the calculated dimensions.

1.2.3 Weir

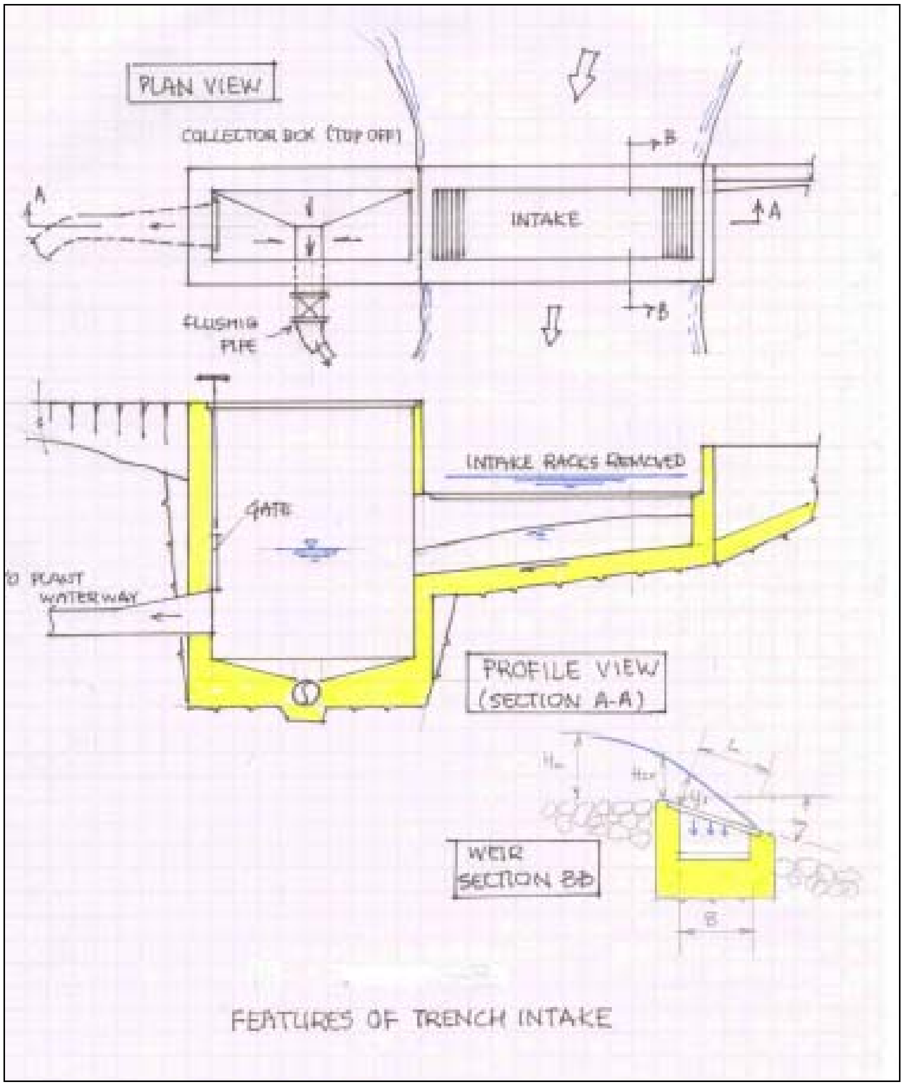
Determine weir height to suit intake gate and flushing gate dimensions, as shown in Figure 2.2.3. For weirs founded on permeable foundations the necessary structure length to control failure by piping should be determined in accordance with Section 2.2.1/4.1 of this Standard. A stepped arrangement is recommended for the downstream face of the weir to dissipate hydraulic energy. The height of the steps should not exceed 0.5 m and the rise over run ratio should not less than 1/3, the stability of the weir cross-section design should be checked for flotation, over turning and sliding in accordance with Section 2.3.1.

1.3 TRENCH INTAKES

Trench intakes are intake structures located in the river bed that draw off flow through racks into a trench which conveys the flow into the project water conductor system. A characteristic of trench intakes is that they have minimum impact on river levels. Trench intakes are applied in situations where traditional headwork designs would be excessively expensive or result in objectionable rises in river levels. There are two quite different applications: on wide rivers and on mountainous streams, but the basic equations are the same for both types. The trench intake should be located in the main river channel and be of sufficient width to collect the design project flow including all flushing flows. If the length of the trench is less than the width of the river, cut off walls will be required into each bank to prevent the river from bypassing the structure. Trench weirs function best on weirs with slopes greater than 4%-5%, for flatter slopes diversion weirs should be considered. The spacing between racks is selected to prevent entry of bed load into the trench. The following terms are sometimes used in referring to trench intake designs.

- Trench weir, when the trench is installed in a raised embankment.
- Tyrolean or Caucasian intakes, when referring to trench intakes on mountainous streams.

Features:



1.3.2 Design Parameters

The following design parameters are suggested for the dimensioning of trench weirs.

- ***Design Flows:***

The following design flows are recommended:

• Bedload flushing flow (from collector box)	$= 0.2\ Q_T$
• Desilter flushing flow	$= 0.2\ Q_T$
• Turbine flow	$= 1.0\ Q_T$
Total design flow	$= 1.4\ Q_T$

Some mountainous rivers, categorized as "sand starved" rivers, have low suspended sediment loads and may not require a desilter; in such cases the requirement for the desilter flushing flow can be omitted.

- ***Dimensional Layout***

  The following factors should be considered in determining the principal dimensions: length, breadth and depth of a trench weir:
  - Minimum width (B)= 1.25 m (to facilitate manual cleaning)
  - Length should be compatible with river cross section. It is recommended that the trench be located across main river channel.
  - Maximum width (B)$\cong$ 2.50m. Trashrack bars longer than about 2.50 m may require support as slenderness ratios become excessive.
  - Invert of collector box should be kept a high as possible.

- ***Racks***
  - The clear spacing between bars should be selected to prevent entry of bed-load particles that are too large to be conveniently handled by the flushing system. Generally designs are based on excluding particles greater than medium gravel size from (2 cm to 4 cm). A clear opening of 3.0 cm is recommended for design.
  - A slope across the rack should be provided to avoid accumulation of bed load on the racks. Slopes normally used vary from 0° to 20°.
  - Rectangular bars are recommended. Bar structural dimension shall be designed in accordance with Section 2.2.1/5.0 of this Standard. An appropriate contraction coefficient should be selected as explained in the following sub-section.
  - Assume 30% blockage.

  Spacing between racks is designed to prevent the entry of bedload but must also be strong enough to support superimposed loads from bedload accumulation, men and equipment. This issue is discussed further in Subsection 2.2.3 / 2.0.

1.3.3 Hydraulic Design of Trench Intake

The first step in hydraulic design is to decide the width of the trench intake bearing in mind the flow capacity required and the bathymetry of the river bed. The next step in hydraulic design is to determine the minimum trench breadth (B) that will capture the required design flow. The design approach assumes complete capture of river flow, which implies, that river flow is equal to plant flow for the design condition.

Hydraulic design is based on the following assumptions:
- Constant specific energy across racks.
- Effective head on screen is equal to base pressure (depth)
- Approach velocity is subcritical with a critical section at the entry to the structure as shown in figure 2.2.3/1.

The set of equations proposed is based on the method given by Lauterjung et al (1989).

- First calculate y_1:

$$y_1 = k.\frac{2}{3}H_0 \qquad \text{-----------} \ (1)$$

Where:

y_1 = depth at upstream edge of rack (m)
H_o = the energy head of the approaching flow (m)
k = an adjustment factor (-)

k is a function of inclination of the rack and can be determined from the following table:

Table: 2.2.1/1 Values of k as a Function of Rack Slope (α)

α	=	0°	2°	4°	6°	8°	10°	12°
k	=	1.000	0.980	0.961	0.944	0.927	0.910	0.894
α	=	14°	16°	18°	20°	22°	24°	26°
k	=	0.879	0.865	0.851	0.837	0.852	0.812	0.800

Then calculate the breadth of the collector trench from the following equations (2) to (4)

$$L = \frac{1.50\ q}{E_1.E_2C.\cos\alpha^{3/2}.\sqrt{2gy_1}} \qquad \text{--------------} \ (2)$$

Where:

L = sloped length across collector trench (m)
E_1 = blockage factor
E_2 = Effective screen area = e/m
C = contraction coefficient
α = slope of rack in degrees
y_1 = flow depth upstream from Equation 1. (m)
q = unit flow entering intake (m^3/s per m)
e = clear distance between bars (cm or m)
m = c/c spacing of bars (cm or m)

Assume E_1 = 0.3 (30%) blockage.
"C" can be calculated from the following formula (as reported by Raudkivi)
Rectangular bars:

$$C = 0.66\left(\frac{e}{m}\right)^{-0.16}.\left(\frac{m}{h}\right)^{0.13} \qquad \text{-------------} \ (3)$$

Assume h = 0.5 y_1. This formula is valid for $3.5 > \frac{h}{m} > 0.2$ and $0.15 < \frac{e}{m} < 0.30$

Finally, the required breadth (B) can be determined as below:

$$B = L\cos\alpha \qquad \text{--------------(4)}$$

1.3.4 Hydraulic Design of Collector Trench

Normally a sufficient slope on the invert of the trench is provided to ensure efficient flushing of bed-load particles that would otherwise accumulate on the invert of the trench.

A suitable scouring slope can be estimated from the following equation:

$$S_s = \frac{0.66\ d^{9/7}}{q_o^{\ 6/7}}$$

Where:

d = sediment size (m)
q_o = flow per unit width (Q/B) at outlet of trench (m^3/s per m)
S_s = design slope of trench invert.

The minimum depth of the trench at the upstream and is normally between 1.0m to 1.5 m, based on water depth plus a freeboard of 0.3 m. For final design the flow profile should be computed for the design slope and the trench bottom profile confirmed or adjusted, as required. A step-by-step procedure for calculating the flow profile that is applicable to this problem can be found in Example 124, page 342-345 of "Open-Channel Hydraulics" by Ven. T.Chow (1959). In most cases the profile will be sub critical with control from the downstream (exit) end. A suitable starting point would be to assume critical flow depth at the exit of the trench.

1.3.5 Collector Box

The trench terminates in a collector box. The collection box has two outlets, an intake to the water conductor system and a flushing pipe. The flushing pipe must be design with the capacity to flush the bed-load sediment entering from the trench, while the project flow is withdrawn via the intake. The bottom of the collection box must be designed to provide adequate submergence for the flushing pipe and intake to suppress undesirable vortices. The flushing pipe should be lower than the intake and the flushing pipe sized to handle the discharge of bed load. If the flushing pipe invert is below the outlet of the trench, the Engineer should consider steepening the trench invert. If the trench outlet invert is below the flushing pipe invert, the latter should be lowered to the elevation of the trench outlet or below. The deck of the collector box should be located above the design flood level to provide safe access to operate gates.

1.3.6 Flushing Pipe

The flushing pipe should be designed to provide a high enough velocity to entrain bed-load captured by the weir. A velocity of at least 3.0 m/s should be provided. If possible, the outlet end of the pipe should be located a minimum of 1.0m above the river bed level to provide energy to keep the outlet area free from accumulation of bed load that could block the pipeline.

1.3.7 References on Trench weirs

CBIP, (2001): *Manual on Planning and Design of Small Hydroelectric Scheme*

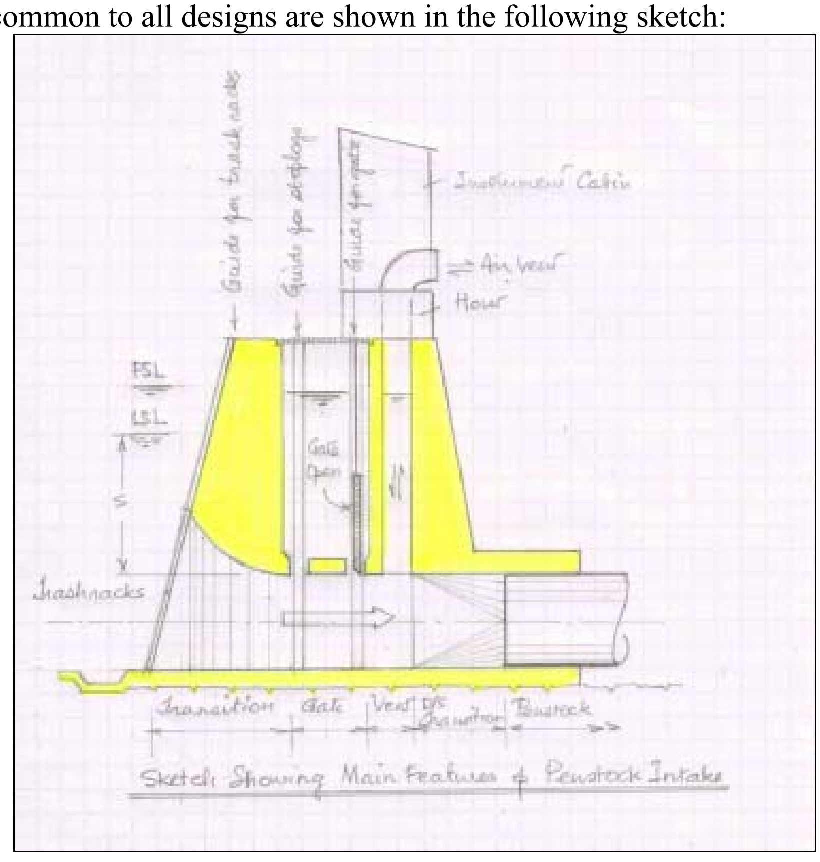
Lauterjung et al (1989): *Planning of Intake Structures*
Freidrich Vieweg and Sohn, Braunswchweig - Germany

IAHR (1993): *Hydraulic Structures Design Manual: Sedimentation: Exclusion and Removal of Sediment from Diverted Water*.
By: Arved J. Raudkivi
Publisher: Taylor & Francis, New York.

Chow (1959): *Open- Channel Hydraulics*
Publisher: McGraw-Hill Book Company, New York.

1.4 RESERVOIR, CANAL AND PENSTOCK INTAKES

The designs of reservoir, canal and penstock intakes are all based on the same principles. However, there are significant variations depending on whether an intake is at the forebay reservoir of a run-of-river plant or at storage reservoir with large draw down or is for a power tunnel, etc. Examples of a variety of layouts can be fond in IS 9761 *Hydropower Intakes – Criteria for Hydraulic Design* or *Guidelines for Design of Intakes for Hydropower Plants* (ASCE, 1995). The features common to all designs are shown in the following sketch:



The objectives of good design are:

- To prevent entry of floating debris.
- To avoid formation of air entraining vortices.
- To minimize hydraulic losses.

1.4..1 Control of floating debris

To prevent the entry of debris a trashrack is placed at the entry to the intake. For small hydro plants the trashrack overall size is determined based on an approach velocity of 0.75 m/s to 1.0m/s to facilitate manual raking. Trashracks may be designed in panels that can be lowered into place in grooves provided in the intake walls or permanently attacked to anchors in the intake face. The trashracks should to sloped at 14° from the vertical (4V:1H) to facilitate raking. The spacing between bars is determined as a function of the spacing between turbine runner blades. IS 11388 *Recommendations for Design of Trashracks for Intakes* should be consulted for information about spacing between trashracks bars, structural design and vibration problems. Also, see Section 2.2.1/5 of this Standard.

1.4.2 Control of Vortices

First of all the direction of approach velocity should be axial with respect the intake if at all possible. If flow approaches at a significant angle (greater than 45^o) from axial these will be significant risk of vortex problems. In such a situation an experienced hydraulic engineer should be consulted and for important projects hydraulic model studies may be required. For normal approach flow the submergence can be determined from the following formulae:

$$S = 0.725VD^{0.5}$$

Where:

S	= submergence to the roof of the gate section	(m)
D	= diameter of penstock and height of gate	(m)
V	= velocity at gate for design flow.	(m/s)

A recent paper by Raghavan and Ramachandran discusses the merits of various formulae for determining submergence (S).

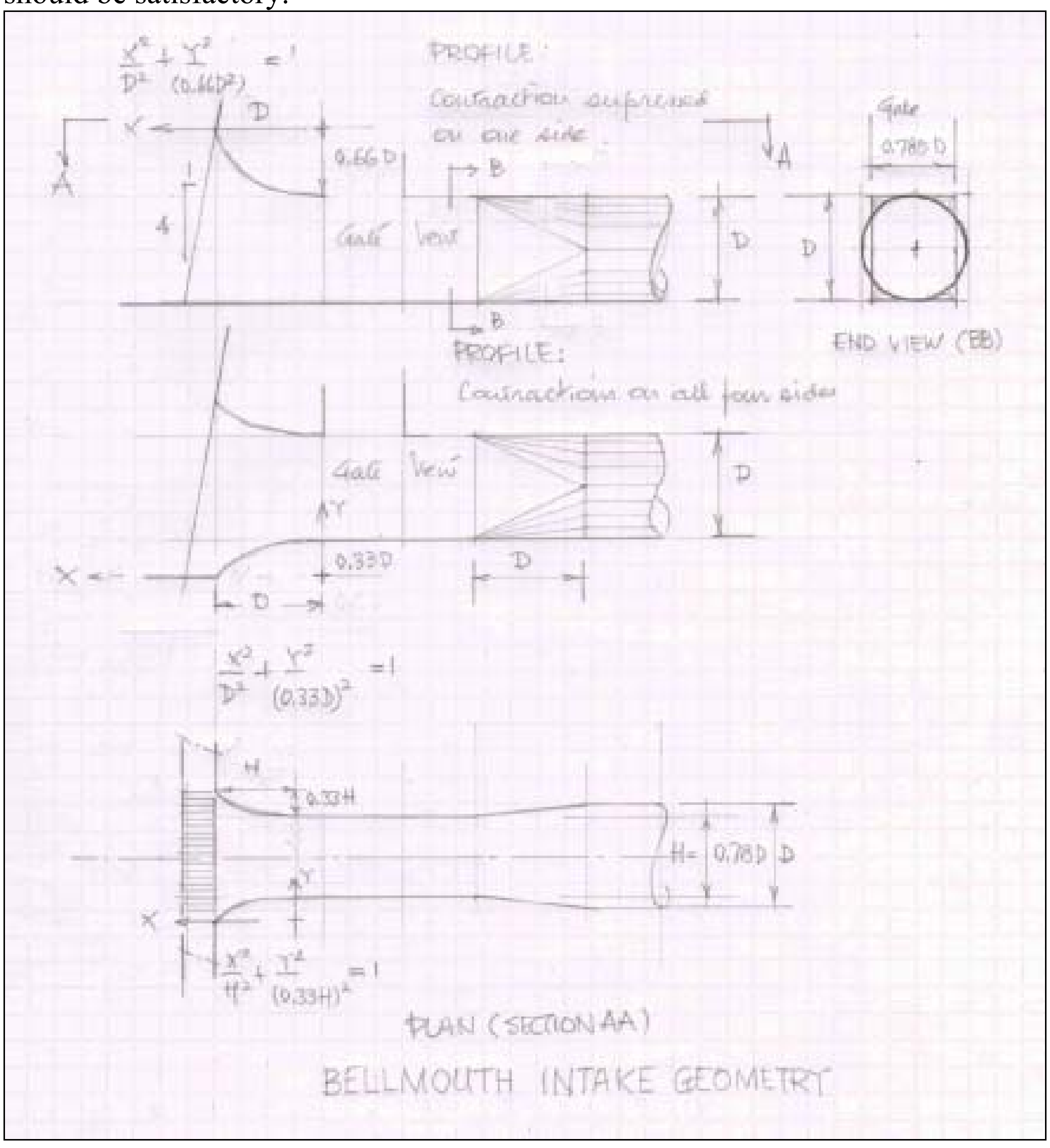
1.4.3 Minimization of Head losses

Head losses are minimized by providing a streamlined transition between the entry section and gate section. Minimum losses will be produced when a streamlined bellmouth intake is used. For a bellmouth intake the transition section is formed with quadrants of ellipses as shown in the following sketch. The bellmouth type intake is preferred when ever the additional costs are economically justified. For smaller, mainly mini hydropower stations, simpler designs are often optimal as the cost of construction of curved concrete surfaces may not be offset by the value of reduction in head losses. Details on the geometry of both types are given

- *Bellmouth Intake Geometry*

Geometries for typical run-of-river intakes are shown below:

A gate width to height of 0.785 (D): 1.00 (H) with H = D is recommended. This permits some reduction in the cost of gates without a significant sacrifice in hydraulic efficiency. There is a second transition between the gate and penstock, rectangular to circular. For a gate having H = D and W= 0.785D the flow velocity at the gate will be equal to the velocity in the penstock so no further flow acceleration is produced in this section. A length for this transition of 1.0x D should be satisfactory.

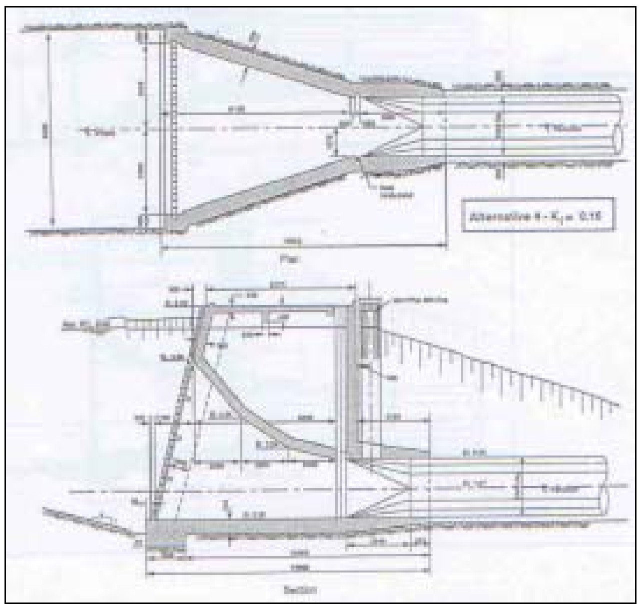


The head loss co-efficient for this arrangement in K_i =0.10

Details for layout of bell mouth transitions connecting to a sloping penstock are given in IS9761.

- *Simplified layout (Mini-Hydro):*

  For smaller/mini hydro projects intake design can be simplified by forming the transition in plane surfaces as shown below: The head loss for this design (K_i) = $0.19V^2/2g$.



1.4.4. AIR VENT

An air vent should be placed downstream of the head gate to facilitate air exchange between atmosphere and the penstock for the following conditions:

- Penstock filling when air will be expelled from the penstock as water enters.
- Penstock draining when air will enter the penstock to occupy the space previously filled by water.

The air vent (pipe) must have an adequate cross section area to effectively handle these exchanges of air. The following design rules are recommended:

Air vent area should the greater of the following values $A_V = 0.20\ A_p$

or

$$A_V = \frac{Q_T}{25.0}$$

Where:

A_V = cross-section area of air vent pipe (m^2)
A_P = cross-section area of penstock (m^2)
Q_P = turbine rated flow (ΣQ_T of more than one turbine on the penstock) (m^3/s)

The air vent should exhaust to a safe location unoccupied by power company employees on the general public.

1.4.5 PENSTOCK FILLING

A penstock should be filled slowly to avoid excessive and dangerous "blow-back". The recommended practice is to control filling rate via the head gate. The head gate should not be opened more than 50 mm until the penstock is completely full. (This is sometime referred to as "cracking" the gate.)

1.4.6 REFERENCES ON PENSTOCK INTAKES:

- Indian Standard Cited.
  IS 9761: *Hydropower Intakes – Criteria for Hydraulic Design*

1.4.7 OTHER REFERENCES

- *Guidelines for Design of Intakes for Hydroelectric Plants*
  ASCE, New York (1995)

- *Validating the Design of an Intake Structure* : By Narasimham Raghavan and M.K. Ramachandran, HRW – September 2007.

- *Layman's Guidebook*
  European Small Hydro Association
  Brussels, Belgium (June 1998)
  Available on the internet.

- *Vortices at Intakes*
  By J.L. Gordon
  Water Power & Dam Construction
  April 1970

1.5. TRASHRACKS AND SAFETY RACKS

1.5.1 Trashracks: Trashracks at penstock intakes for small hydro plants should be sloped at 4 V: 1H to facilitate manual raking and the approach velocity to the trashracks limited to 1.0 m/s or less. Use of rectangular bars is normal practice for SHP's. Support beams should be alignment with the flow direction to minimize hydraulic losses. Detailed trashrack design should be done in accordance with IS 11388.

1.5.2 Safety Racks: Safety racks are required at tunnel and inverted siphon entries to prevent animals or people who may have fallen into the canal from being pulled into these submerged water ways. A clear spacing of 200 mm between bars is recommended. Other aspects of design should be in accordance with IS 11388.

1.5.3 References on Trashracks

IS11388 – *"Recommendations for Design of Trashracks for Intakes"*.
ASCE (1995) --*"Guidelines for Design of Intakes for Hydroelectric Plants"*.

DRAWINGS:

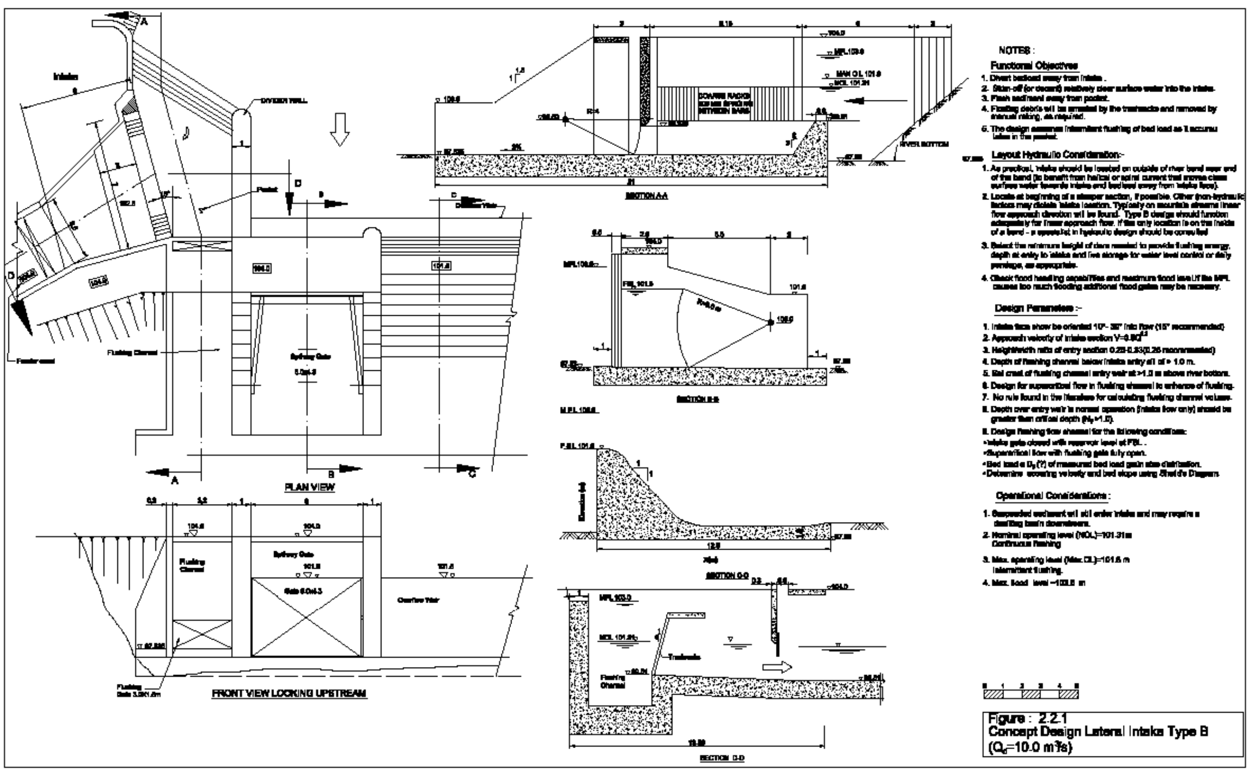


Figure : 2.2.1 Concept Design Lateral Intake Type B (Q_d=10.0 m³/s)

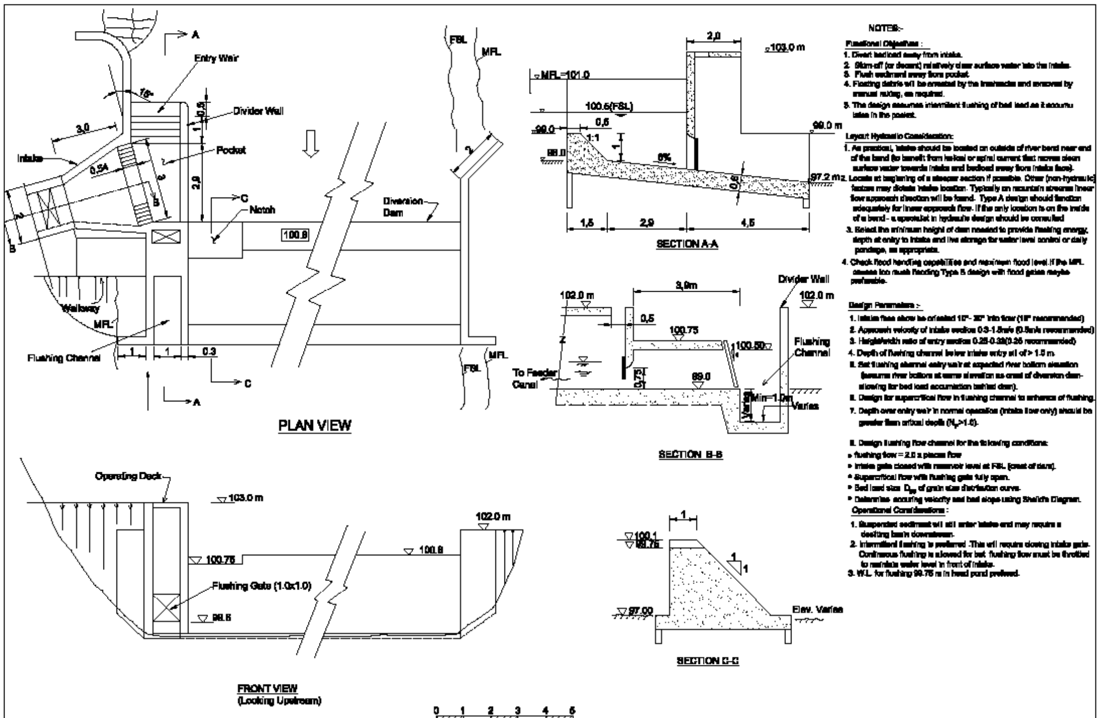


Figure 2.2.2 Concept Design for Small Lateral Intake

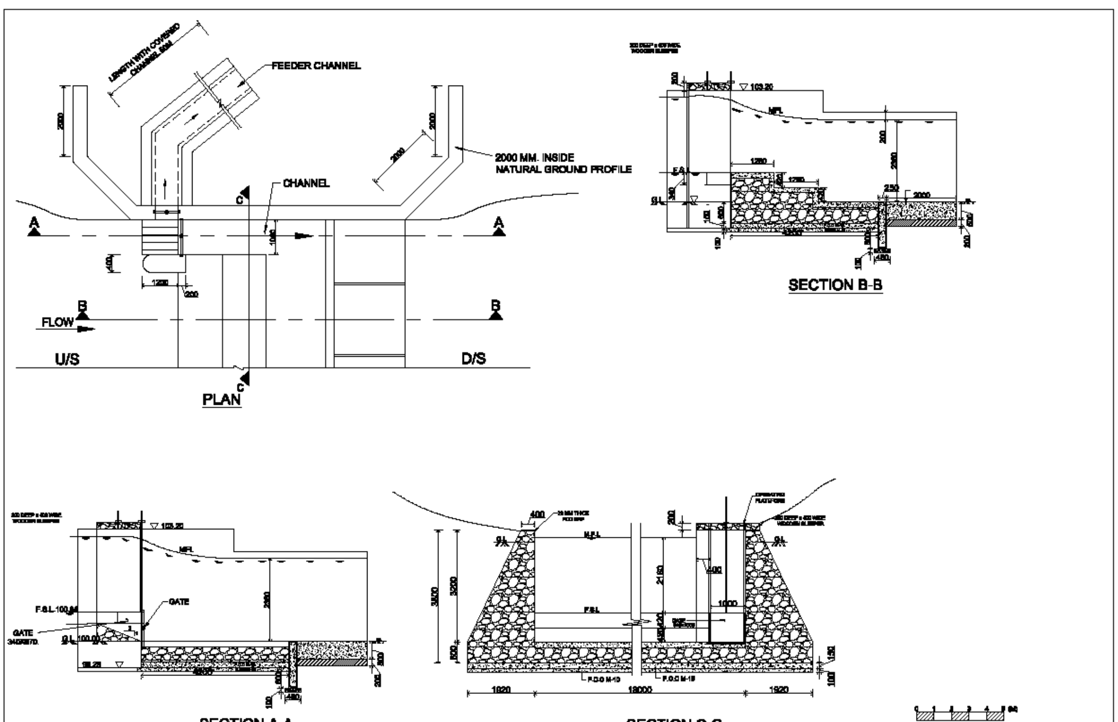


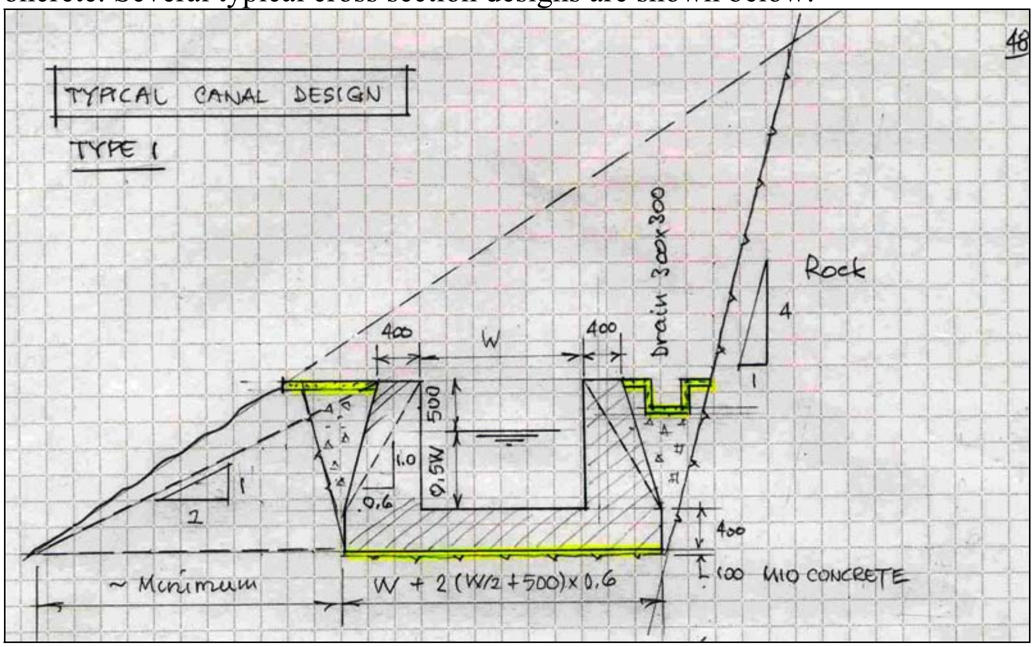
FIGURE 2.2.3
CONCEPT DESIGN SEMI PERMANENT HEAD WORKS

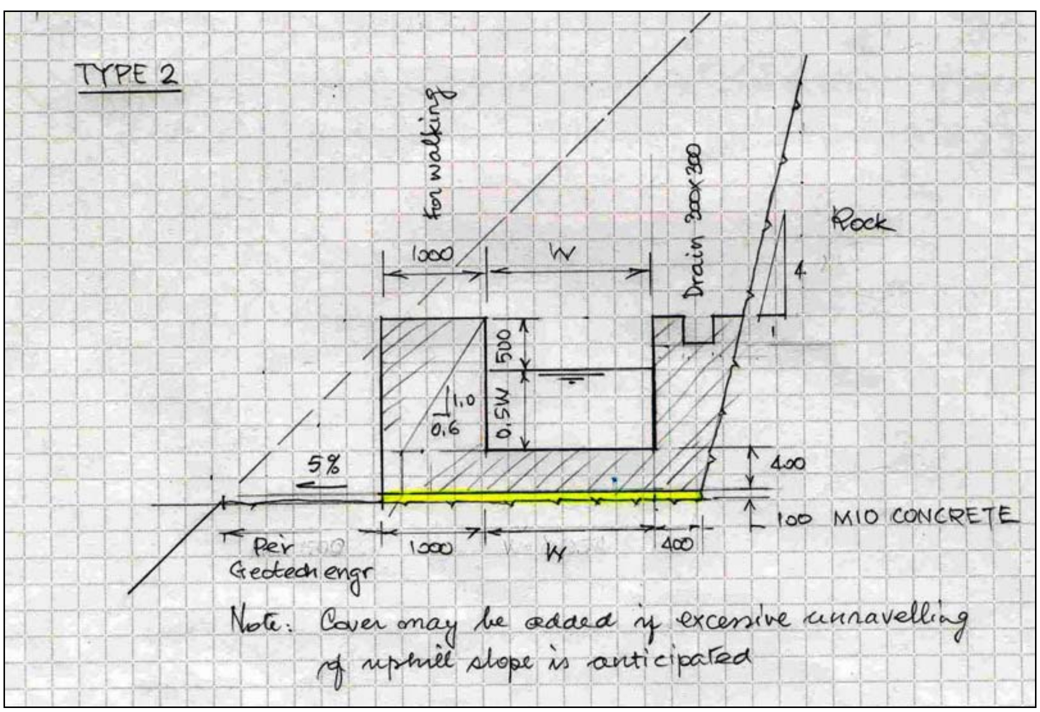
2. HYDRAULIC DESIGN OF WATERWAYS

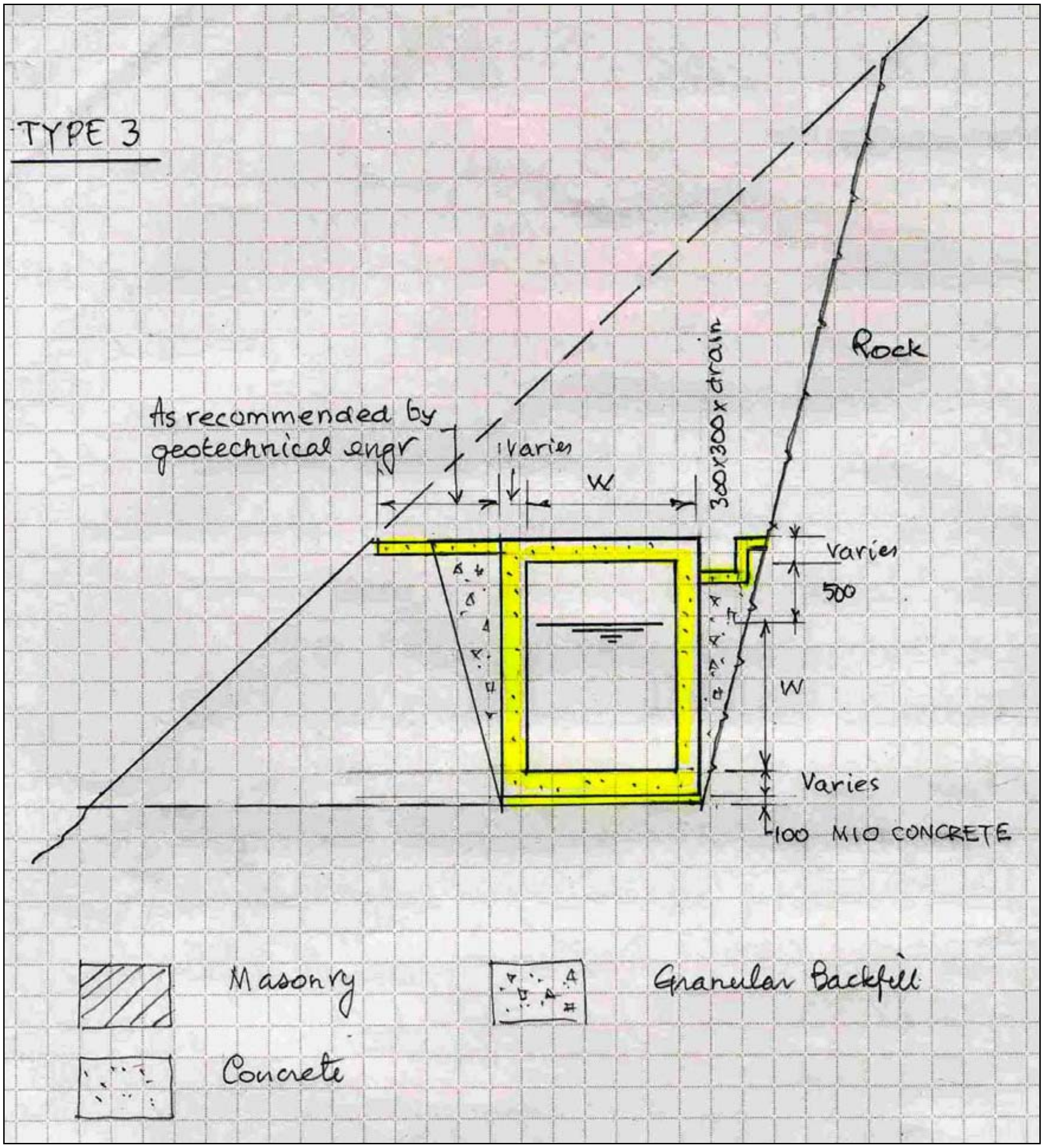
The waterways or water conduction system is the system of canals, aqueducts, tunnels, inverted siphons and pipelines connecting the head works with the forebay tank. This Section provides guidelines and norms for the hydraulic design of these structures.

2.1 CANALS

2.1.1 Canals for small hydro plants are typically constructed in masonry or reinforced concrete. Several typical cross section designs are shown below:







Lined canals in earth, if required, should be designed in accordance with Indian Standard: IS 10430.

A further division of canal types is based on function:

- Feeder canal to connect the head regulator (intake) to the desilter
- Power canal to connect the desilter to the Forebay tank.

2.1.2 Feeder Canals

2.1.2.1 Feeder canal hydraulic design shall be based on the following criteria:
Design flow (Q_d) = Turbine flow (Q_T) + Desilter flushing flow (Q_F).

2.1.2.2 Scouring velocity:

A sufficiently high velocity must be provided to prevent deposition of sediment within the canal. This (scouring) velocity can be determined from the following formulae:

$$S_C = 0.66 \frac{d^{9/7}}{q^{6/7}} \qquad n = 0.015$$

$$\therefore \qquad V_S = \frac{1}{n} . R^{2/3} \; . \; S_C^{1/2}$$

Where:

Sc	=	Scouring slope	
d	=	Target sediment size	(m)
q	=	Flow per unit width (Q/W)	(m/s/m)
R	=	hydraulic radius	(m)
Vs	=	scouring velocity	(m/s)
n	=	Manning's roughness coefficient	

2.1.2.3 Optimization:

The optimum cross section dimensions, slope and velocity should be determined by economic analysis so as to minimize the total life time costs of capital, O&M and head losses (as capitalized value). The economic parameters for this analysis should be chosen in consultation with the appropriate regional, state or central power authorities these parameters include:

- Discount rate (i)
- Escalation rate(e)
- Plant load factor
- Service life in years (n)
- Annual O+M for canal (% of capital cost)
- Value of energy losses (Rs/kWh).

Also see Section 1.7 of this Standard.

The selected design would be based on the highest of V_s *or* $V_{optimum}$.

2.1.2.4 Freeboard:

A freeboard allowance above the steady state design water level is required to contain water safely within the canal in event of power outages or floods. A minimum of 0.5 m is recommended.

2.1.3 Power Canals:

Power canal design shall be based on the following criteria

a) Design flow = total turbine flow (Q_T)

b) Power canal design should be based on optimization of dimensions, slope and velocity, as explained in the previous section.

For mini-hydro plants Q < 2.0 m^3/s optimal geometric design dimensions for Type 1 (masonry construction) can be estimated by assuming a longitudinal slope of 0.004 and a Manning's n value of 0.018. Masonry construction would normally be preferred for canals with widths (W) less than 2.0 m (flow area =

2.0 m^2). For larger canals with flow areas greater than 2.0m^2, a Type 3, box culvert design would be preferred – based on economic analysis.

c) Freeboard:

A freeboard allowance above the steady state design level is required to contain water safety within the canal in event of power outages. The waterway in most SHP's terminates in a Forebay tank. This tank is normally equipped with an escape weir to discharge surplus water or an escape weir is provided near to the forebay tank. For mini-hydro plants a minimum freeboard of 0.50 m is recommended.

The adequacy of the above minimum freeboard should be verified for the following conditions:

- Maximum flow in the power canal co-incident with sudden outage of the plant.
- Design flow plus margins for leakage losses (+0.02 to +0.05 Q_T) and above rated operation (+ 0.1Q_T).
- Characteristics of head regulator flow control.

The freeboard allowance may be reduced to 0.25 m after taking these factors into consideration.

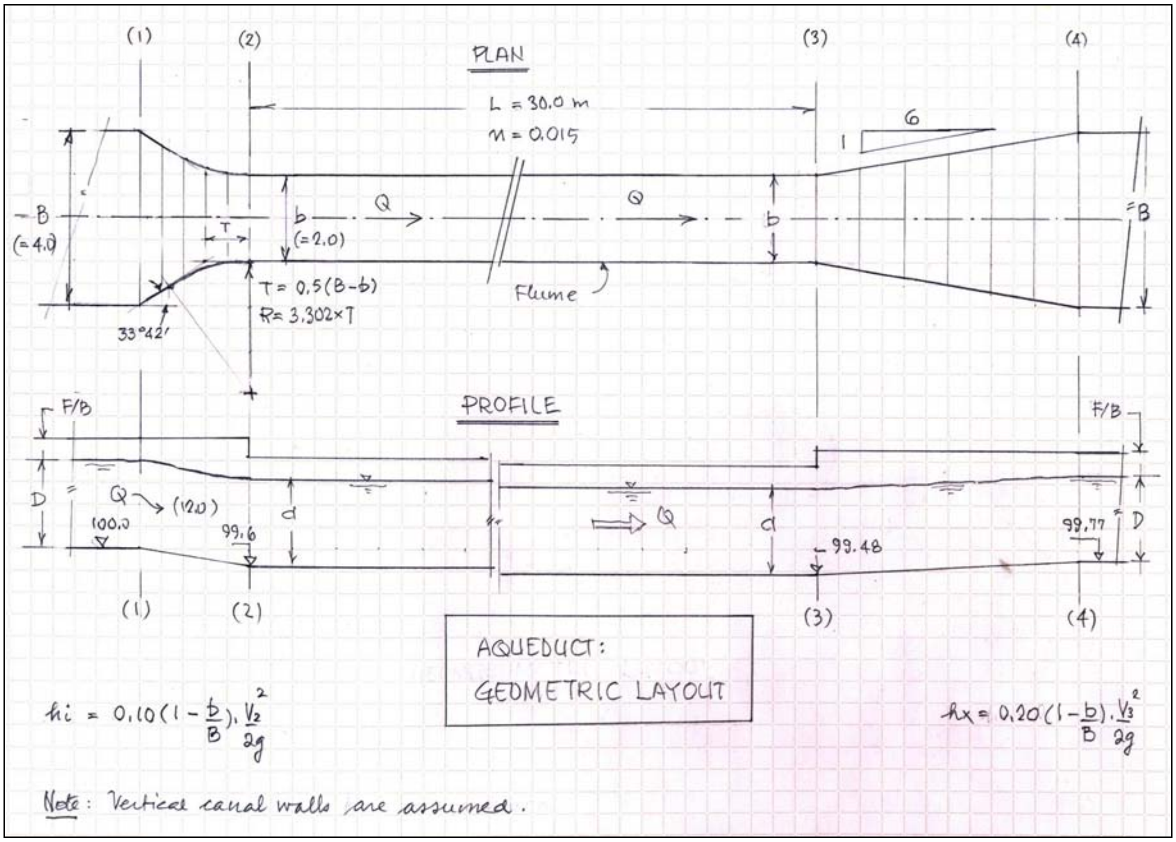
The maximum water level occurring in the forebay tank can be determined from the weir equation governing flow in the escape weir.

2.1.4 Rejection Surge

Designs which do not incorporate downstream escape weirs would be subject to the occurrence of a rejection surge in the canal on sudden turbine shutdown, giving above static water levels at the downstream end, reducing to the static level at the upstream (entry) end of the water way. Methods for evaluating water level changes due to a rejection surge are explained in Section 2.2.2 / 7.0 of this Standard.

2.2 AQUEDUCTS

Aqueducts are typically required where feeder or power canals pass over a gully or side stream valley. If the length of the aqueduct is relatively short the same channel dimensions as for the canal can be retained and there would be no change in hydraulic design. For longer aqueducts design would be based on economic analysis subject to the proviso that flow remains sub critical with $N_F \leq 0.8$ in the flume sections. The following sketch shows the principal dimension of aqueduct entry and exit transitions and flume section.



The changes in invert elevation across the entry and exit structures can be calculated by Bernouli's equation as below:

- Entry transition – consider cross – section (1) and (2);

  $$Z_1 + D + \frac{V_1^2}{2g} = Z_2 + d + \frac{V_2^2}{2g} + hL$$

  and

  $$h_L = 0.10\left(1 - \frac{b}{B}\right).\frac{V_2^2}{2g}$$

  Z_2 can be determined from the above equations, since all geometrical parameters are known.

- Flume – Sections (2) to (3)

  The slope of the flume section is determined from Manning's equation $(S) = \left(\frac{Vn}{R^{2/3}}\right)^2$. *A Manning's n = 0.018 is suggested for concrete channels.*

  Some designers increase this slope by 10% to provide a margin of safety on flow capacity of the flume.

- Exit transition – consider cross section (3) and (4):

  $$Z_3 + d + \frac{V_3^2}{2g} = Z_4 + D + \frac{V_4^2}{2g} + hL$$

and

$$h_L = 0.20\left(1 - \frac{b}{B}\right).\frac{V_3^2}{2g}$$

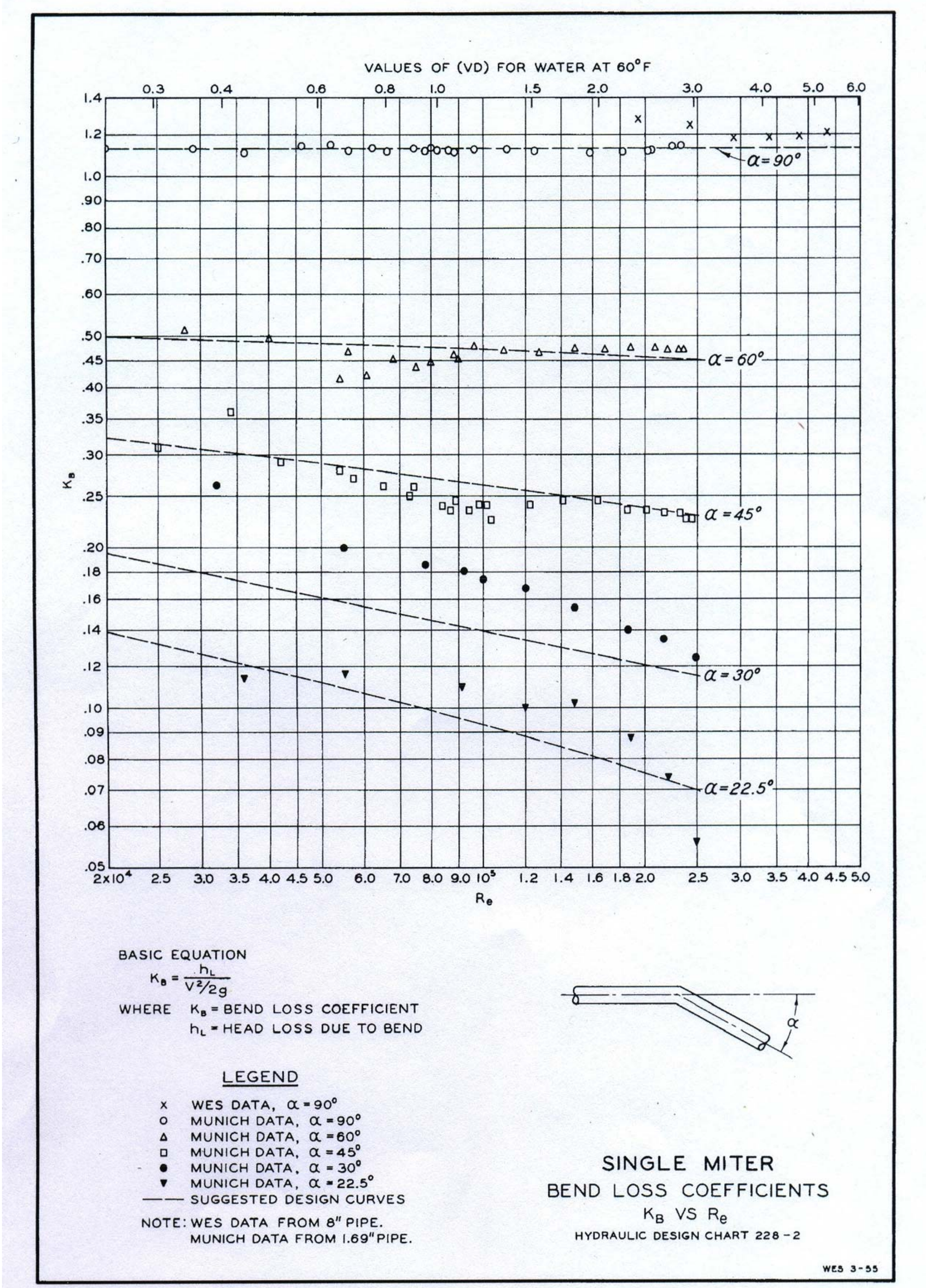
Z_4 can be determined from the above equations, since all geometrical parameters are known.

The same basic geometry can be adapted for transition between trapezoidal canals sections and rectangular flume section, using mean flow width (B) = A/D.

2.3. INVERTED SYPHONS

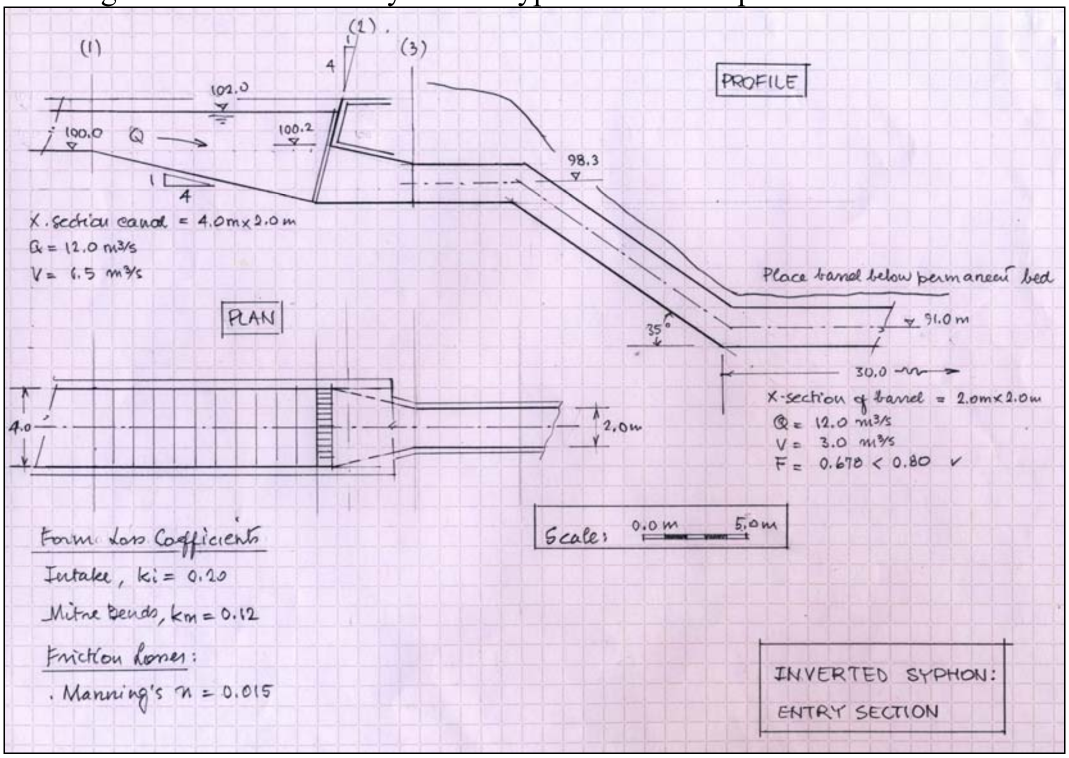
2.3.1 Inverted syphons are used where it is more economical to route the waterway underneath an obstacle. The inverted syphon is made up of the following components:

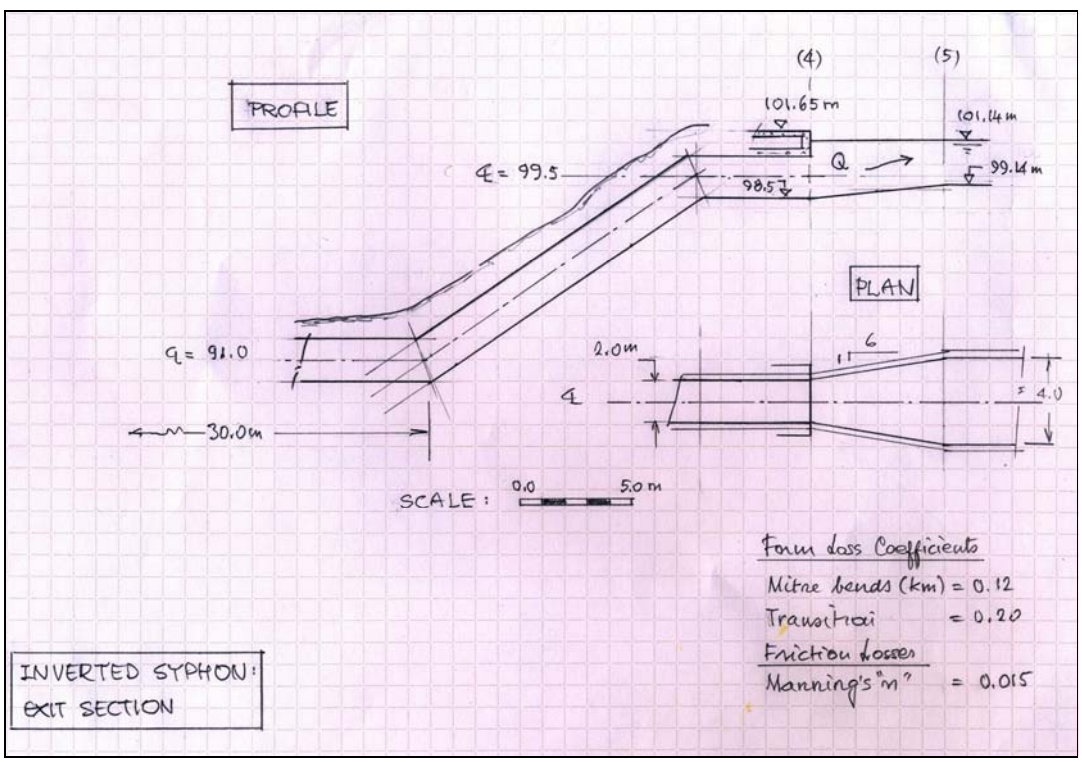
- Entry structure
- Syphon barrels
- Exit structure
- **Entry Structure:**
  Hydraulic design of the entry structure is similar to the design of reservoir, canal and penstock intakes. Follow the guidelines given in Section 2.2.2/2. of this Standard.
- **Syphon barrels:**
  The syphon barrel dimensions are normally determined by optimization studies, with the proviso that the Froude Number $\left(N_F = \frac{V}{\sqrt{gd}}\right)$ does not exceed 0.8. Invert elevations are determined by accounting for head losses from entry to exit of the structure using Bernouli's equation. For reinforced concrete channels a Manning's "n" value of 0.018 is recommended. The head loss coefficients for mitre bends can be determined from USACE HDC 228.2.


VALUES OF (VD) FOR WATER AT 60°F
K_B
R_e
α = 90°
α = 60°
α = 45°
α = 30°
α = 22.5°
BASIC EQUATION
$K_B = \frac{h_L}{V^2/2g}$
WHERE K_B = BEND LOSS COEFFICIENT
h_L = HEAD LOSS DUE TO BEND
LEGEND
x WES DATA, α = 90°
o MUNICH DATA, α = 90°
Δ MUNICH DATA, α = 60°
□ MUNICH DATA, α = 45°
● MUNICH DATA, α = 30°
▼ MUNICH DATA, α = 22.5°
—— SUGGESTED DESIGN CURVES
NOTE: WES DATA FROM 8" PIPE.
MUNICH DATA FROM 1.69" PIPE.
α
SINGLE MITER
BEND LOSS COEFFICIENTS
K_B VS R_e
HYDRAULIC DESIGN CHART 228 - 2
WES 3-55

- **Exit structure:**
  The exit structure is designed as a diverging transition to minimize head losses; the design is similar to the outlet transition from flume to canal as discussed in Subsection 2.2.2/2 of this Standard.

The following sketches show the layout of a typical inverted siphon.





2.3.2 Reference on Aqueducts and Inverted Syphons

"Hydraulic Structures"
By C.D. Smith
University of Saskatchewan
Saskatoon (SK)
Canada

2.4. LOW PRESSURE PIPELINES

Low pressure pipelines may be employed as an alternative to pressurized box culverts, aqueducts or inverted syphons. Concrete, plastic and steel pipes are suitable depending on site conditions and economics. Steel pipe is often an attractive alternative in place of concrete aqueducts in the form of pipe bridges, since relatively large diameter pipe possesses significant inherent structural strength. Steel pipe (with stiffening rings, as necessary), concrete and plastic pipe also have significant resistance against external pressure, if buried, and offer alternatives to inverted syphons of reinforced concrete construction. Generally pressurized flow is preferred. The pipe profile should be chosen so that pressure is positive through out. If there is a high point in the line that could trap air on filling an air bleeder valve should be provided. Otherwise, hydraulic design for low pressure pipelines is similar to the requirements for inverted syphons.

The choice of type of design; low pressure pipeline land pipeline material), inverted syphon or aqueduct, depends on economic and constructability considerations, in the context of a given SHP.

Manning's "n" Values for selected Pipe Materials

Material	**Manning's "n"**
Welded Steel	0.012
Polyethylene (HDPE)	0.009
Poly Vinyl Chloride (PVC)	0.009
Asbestos Cement	0.011
Cast iron	0.014
Ductile iron	0.015
Precast concrete pipe	0.013(2)

Note: (1) From Table 5.4 Layman's Guide Book – ESHA
(2) From Ven T. Chow – Open Channel Hydraulics

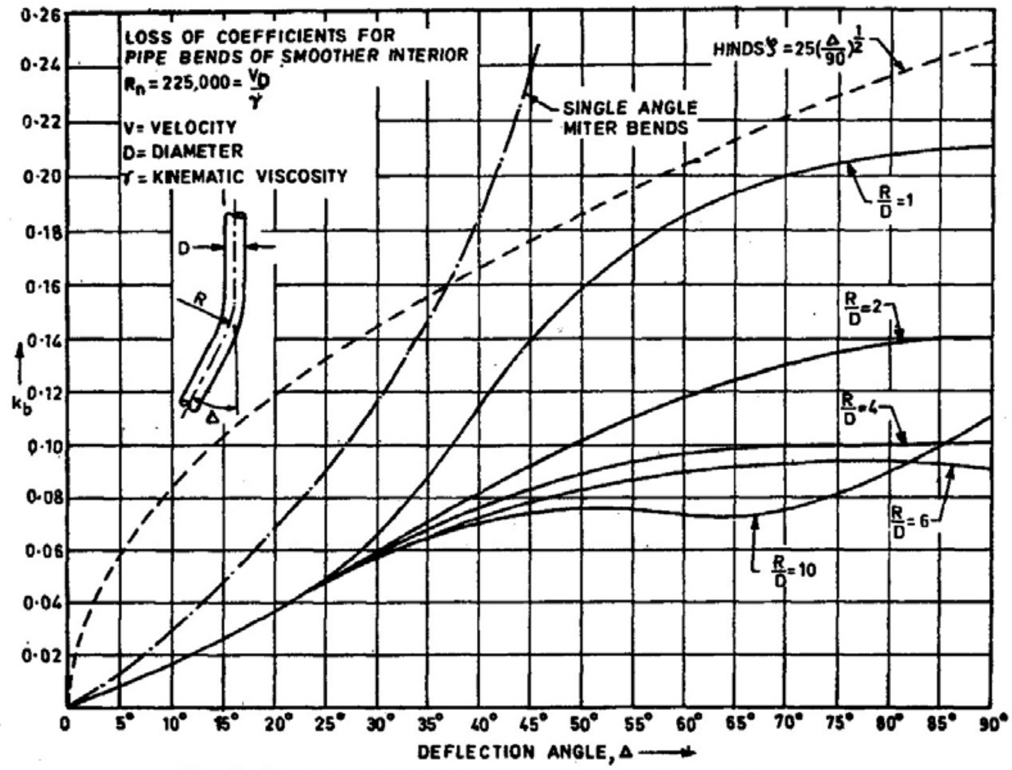


FIG. 3 LOSS COEFFICIENTS FOR PIPE BENDS OF SMOOTH INTERIOR

IS : 11625 - 1986

2.5. TUNNELS

2.5.1 Tunnels often provide an appropriate solution for water conveyance in mountainous areas. Tunnels for SHP are generally of two types.

- Unlined tunnels
- Concrete lined tunnels

On SHP tunnels are usually used as part of the water ways system and not subject to high pressures.

2.5.2 Unlined tunnels:

Unlined water tunnels can be used in areas of favourable geology where the following criteria are satisfied:

a) Rock mass is adequately water tight.

b) Rock surfaces are sound and not vulnerable to erosion (or erodible zones are suitably protected.

c) The static water pressure does not exceed the magnitude of the minor field rock stress.

Controlled perimeter blasting is recommended in order to minimize over break and produce sound rock surfaces. Additionally, this construction approach tends to produce relatively uniform surfaces and minimizes the hydraulic roughness of the completed tunnel surfaces. Design velocities of 1.5 to 2.0 m/s on the mean cross section area give optimal cross section design. It is normal practice to provide a 100mm thick reinforced concrete pavement over leveled and compacted tunnel muck in the invent of the tunnel.

IS 4880: Part 3 provides additional guidance on the hydraulic design of tunnels and on the selection of appropriate Manning's "n" values.

2.5.3 Lined Tunnels

Where geological are unfavourable it is often necessary to provide concrete linings for support of rock surfaces. IS4880: Parts 1-7 give comprehensive guidelines on the design of lined tunnels.

2.5.4 High Pressure Tunnels

Design of high pressure tunnels is not covered in this standard. For high pressure design, if required, the designer should consult an experienced geotechnical engineer or engineering geologist. For the purpose of this standard, high pressure design is defined as tunnels subject to water pressures in excess of 10m relative to the crown of the tunnels.

2.5.5 Reference on Tunnels

IS Standards:
IS 4880 "*Code of Practice for the Design of Tunnels Conveying Water*".

Other References:
" *Norwegian Hydropower Tunnelling*"
(Third volume of collected papers)
Norwegian Tunneling Society
Trondheim, Norway.
www.tunnel.no
Notably:
Development of Unlined Pressure Shafts and Tunnels in Norway, by Einar Broch.

2.6. CULVERTS AND CROSS-DRAINAGE WORKS

Small hydro projects constructed in hilly areas usually include a lengthy power canal routed along a hillside contour. Lateral inflows from streams and gullies intercepted by SHP canals often transport large sediments loads which must be prevented from entering the canal. The first line of defense is the canal upstream ditch which intercepts local lateral runoff. The flow in these chains must be periodically discharged or the drain capacity will be exceeded. Flow from these drains is usually evacuated via culverts passing underneath the canal. These culverts would normally be located where gullies or streams cross the canal alignment. The capacity of canal ditches should be decided taking into consideration the average distance between culverts. In the rare cases when distance between culverts is excessive, consideration should be given to diverting ditch flows across the canal in flumes or half round pipes to discharge over the downhill side of the canal at suitable locations. Culverts are usually required where the canal route crosses gullies or streams. Culverts at these points provide for flow separation between lateral inflows and canal inflows and often present the most economical solution for crossing small but steep valley locations. It is recommended that culverts design be based on the following hydrological criteria.

- For mini hydro projects, 1 in 10 year flood (Q_{10})
- For small hydro projects, 1 in 25 year flood (Q_{25})

Where it is practical to extract the necessary basin parameters, the procedures given in Section 1.4 should be applied. Otherwise design flows should be estimated from field measurements of cross section area and longitudinal slope at representative cross section of the gully or side stream.

A survivable design approach is further recommended with canal walls strengthened to allow local over topping without damage to the canal integrity when floods exceed the design flood values.

Detailed hydraulic design should be based on information from reliable texts or design guidelines – such as:

- "*Design of Small Bridges and Culverts*"
  Goverdhanlal

- "*Engineering and Design – Drainage and Erosion Control*".
  Engineering Manual EM 1110-3-136
  U.S. Army Corps of Engineers (1984)
  www.usace.army.mil/publications/eng-manuals

- Manufacturer's guides, notably:
  - American Concrete Pipe Association
    www.concrete-pipe.org
  - Corrugated Steel Pipe Institute
    www.cspi.ca

2.7 Power Canal Surges

2.7.1 Power canals that are not provided with escape weirs near their downstream end will be subject to canal surges on rapid load rejections or load additions. The rejection surge will typically cause the downstream water level to rise above static level and may control the design of canal freeboard. For load additions there is a risk that the level will fall to critical at the downstream end and restrict the rate at which load can be taken on by the unit.

The following formulae taken from IS 7916: 1992 can be used to estimate the magnitude of canal surges.

Maximum surge height in a power channel due to load rejection may be calculated from the empirical formulae given below:

For abrupt closure $h_{\max} = \sqrt{K^2 + 2Kh}$

For gradual closure within the period required for the first wave to travel twice the length of the channel:

$$h_{\max} = \frac{K}{2} + V.\sqrt{h/g}$$

Where:

h_{max} = *maximum surge wave height,*

K = $V^2/2g$ = *velocity head,*

V = *mean velocity of flow, and*

$$\text{h} = \text{effective depth} = \frac{area\ of\ cross\ \sec tion}{top\ width}$$

- Maximum water level resulting from a rejection surge at the downstream of a canal:
  Maximum W.L. = $Y_o + h_{max}$
- Minimum water level resulting from by a start up surge at the downstream end of a canal:
  Minimum W.L. = $Y_S - h_{max}$

  Where:

  Y_o = steady state downstream water level

  Y_S = static downstream water level.

The maximum water level profile can be approximated by a straight line joining the maximum downstream water level to the reservoir level.

2.7.2 Canal Surges on Complex Waterways:

For waterway systems comprising several different water conductor types, the above equations are not applicable. In such cases a more detailed type of analysis will be required. The U.S. National Weather Service FLDWAV computer program can be used to solved for the transient flow conditions in such cases (Helwig, 2002).

2.7.3 References

IS Standards cited:

IS 7916: 1992 *"Open Channel – Code of Practice"*.

Other References

"Application of FLDWAV(Floodwave) Computer Model to Solve for Power Canal Rejection Wave for Simple and Complex Cases".
P.C. Helwig
Canadian Society for Civil Engineering
Proceedings, Annual Conference
Montreal, Canada (2002).

3. HYDRAULIC DESIGN OF DESILTERS

3.1 BACKGROUND

Sediment transported in the flow, especially particles of hard materials such as quartz, can be harmful to turbine components. The severity of damage to equipment is a function of several variables, notably: sediment size, sediment hardness, particle shape, sediment concentration and plant head.

The control of turbine wear problems due to silt erosion requires a comprehensive design approach in which sediment properties, turbine mechanical and hydraulic design, material selection and features to facilitate equipment maintenance are all considered (Naidu, 2004). Accordingly the design parameters for desilter design should be made in consultation with the mechanical designers and turbine manufacturer.

Where the risk of damage is judged to be high a settling basin (or desilter) should be constructed in the plant waterway to remove particles, greater than a selected target size.

3.1.1 Need

The first design decision is to determine whether the sediment load in the river of interest is sufficiently high to merit construction of a desilter. There is little guidance available on this topic; however, the following limits are suggested by Naidu (2004):

Table 2.2.3/1.0 Suggested Maximum Allowable Sediment Concentration versus Plant Head.

Parameter	Low and Medium Head Turbines	High Head Turbines
Head	≤ 150 m	> 150 m
Maximum allowable sediment concentration	200 ppm	150 ppm

3.1.2 Removal Size

There are also considerable divergences of opinion on the selection of design size for sediment removal. Nozaki (1985) suggests a size range of between 0.3 mm to 0.6 mm for plant heads ranging from 100 m to 300 m. Indian practice is to design for a particles size of 0.20 m regardless of head. Some authors suggest that removal of particles smaller than 0.20 mm is not practical.

The adoption of 0.20 mm is the design (target) sediment size is recommended for Indian SHP designs.

3.1.3 Types of Desilters

There are two basic types of desilters:

Continuous flushing type
Intermittent flushing type

Guidelines for design of both types are given in this section.

3.2. DESIGN CONSIDERATIONS

3.2.1 Data Requirements (Small Hydro Plants)

It is recommended that a program of suspended sediment sampling be initiated near the intake site from an early stage during site investigations to ensure that sufficient data is available for design. The sampling program should extend through the entire rainy season and should comprise at least two readings daily. On glacier fed rivers where diurnal flow variations may exist, the schedule of sampling should be adjusted to take this phenomenon into account and the scheduled sampling times be adjusted to coincide with the hour of peak daily flow with another sample taken about twelve hours later.

While it is often assumed that sediment load is directly related to flow, this is only true on the average, in a statistical sense. In fact it is quite likely, that the peak sediment event of a year may be associated with a unique upstream event such as a major landslide into the river. Such events often account for a disproportionately large proportion of the annual sediment flow. Therefore, it would also be desirable to design the sediment measurement program to provide more detailed information about such events, basically to increase the sampling frequency to one sample per 1 or 2 hours at these times.

A five year long sediment collecting program would be ideal. Less than one monsoon season of data is considered unsatisfactory.

Some authors suggest that the vertical variation of sediment concentration and variations horizontally across the river be measured. However, on fast flowing rivers inherent turbulence should ensure uniform mixing and sampling at one representative point should be sufficient.

The data collected in a sediment sampling program should include:

- Mean daily concentration of suspended sediment (average of two readings twelve hours apart)
- Water temperature
- Flow (from a related flow gauging program)

The following additional information can then be derived from collected samples.

- A sediment rating curve (sediment concentration versus flow – where possible)
- Particle size gradation curve on combined sample
- Specific gravity of particles.

It is also recommended that a petrographic analysis be carried out to identify the component minerals of the sediment mix. It is likewise recommended that experiments be made on selected ranges of particles sizes to determine settling velocities. A further discussion on the subject of sediment sampling is given in Avery (1989)

The characteristics of the sediment on a given river as obtained from a data collection program will assist in selection of appropriate design criteria.

3.2.2 Data Requirements (Mini Hydro Plants)

On mini hydro projects where resources and time may not be available to undertake a comprehensive sampling program, selection of design parameters will depend to a great extent on engineering judgment, supplemented by observations on site and local information. The following regional formula by Garde and Kothyari (1985) can be used to support engineering decision making.

$$Vs = 530.0\, P^{0.6} .\, Fe^{1.7} . S^{0.25}\, D_d^{0.10} . \left(\frac{P_{max}}{P} \right)^{0.19}$$

Where

Vs	= mean sediment load in	(tonnes/km^2/year)
s	= average slope	(m/m)
D_d	= drainage density, as total length of streams divided by catchment area	(km/km^2)
P	= mean annual precipitation	(cm)
P_{max}	= average precipitation for wettest month	(cm)
F_e	= ground cover factor, as below:	

$$F_e = \frac{1}{\sum Ai}[0.80A_A + 0.60A_G + 0.30A_F + 0.10A_W]$$

A_A	= arable land area	
A_G	= grass land area	(all in km^2)
A_F	= forested area	
A_W	= waste land area (bare rock)	

3.2.3 Design Criteria

The principle design criteria are:

1. The target size for removal (d): d = 0.20 mm is recommended
2. Flushing flow: $Q_F = 0.2\ Q_P$ is recommended
3. Total (design) flow: $Q_T = Q_P + Q_F = 1.2\ Q_P$.
   Where Q_P is plant flow capacity in (m^3/s).

3.2.4 Siting

The following factors control site selection

1. A site along the water way of appropriate size and relatively level with respect to cross section topography
2. A site high enough above river level to provide adequate head for flushing. For preliminary layout a reference river level corresponding to the mean annual flood and minimum flushing head of 1.50 m is recommended.

In principle a desilting tank can be located anywhere along the water conductor system, upstream of the penstock intake. Sometimes it is convenient to locate the desilting basin at the downstream end of the waterway system where the desilter can also provide the functions of a forebay tank. However, a location as close to the head works is normally preferred, site topography permitting.

3.3 Hydraulic Design

A desilter is made up of the following elements:

- Inlet section
- Settling tank
- Outlet section
- Flushing system

3.3.1 Inlet Section

The purpose of the inlet section is to reduce flow velocity from the relatively high speed of the feeder canal to the low speed of the settling tank. For efficient functioning of the settling tank the velocity should be as uniform as possible without short circuits or localized high velocity areas. Where possible, introducing flow into the settling section via a distribution weir or diffuser wall is preferred. Alternatively, transition structure with walls diverging at a rate of 6:1 is recommended. A design with vanes may also be considered, as shown in Figure.

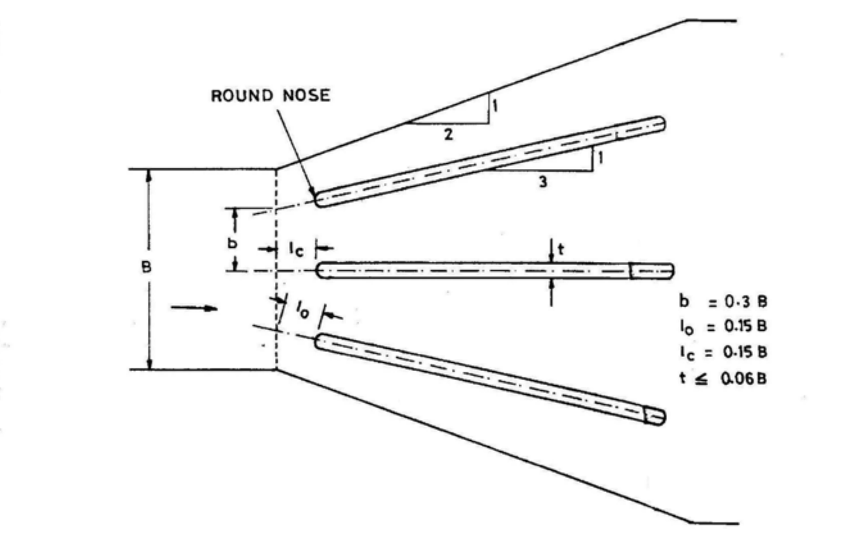


FIGURE 2.2.3.1
Recommended geometry for vanes at a basin entry for 2:1 expansion (After Atkinson 1992)

Hydraulic losses in the inlet transition can be estimated as:

$$h_L = \frac{0.3}{2g}\left(V_F^2 - V_T^2\right)$$

Where:

V_F = velocity in feeder canal (m/s)
V_T = velocity in settling basin (m/s)

3.3.2 Settling Section

The fundamental design objective is to remove all particles equal or greater than the chosen target removal size (d). The methodology recommended follows the approach given by Mosonyi:

a) Flow velocity in the tank should not entrain material that has settled out to the bottom of the tank

Thus $U \leq a\sqrt{d}$

Where, U = velocity through tank (m/s)
d = target sediment size in (mm)
a = 0.36 for d > 1.0 mm
= 0.44 for 1.0 mm > d ≥ 0.10 mm
= 0.51 for d < 0.10 mm

b) Select fall velocity (w_o) for d from Figure 2.2.3.2, assuming an appropriate water temperature.

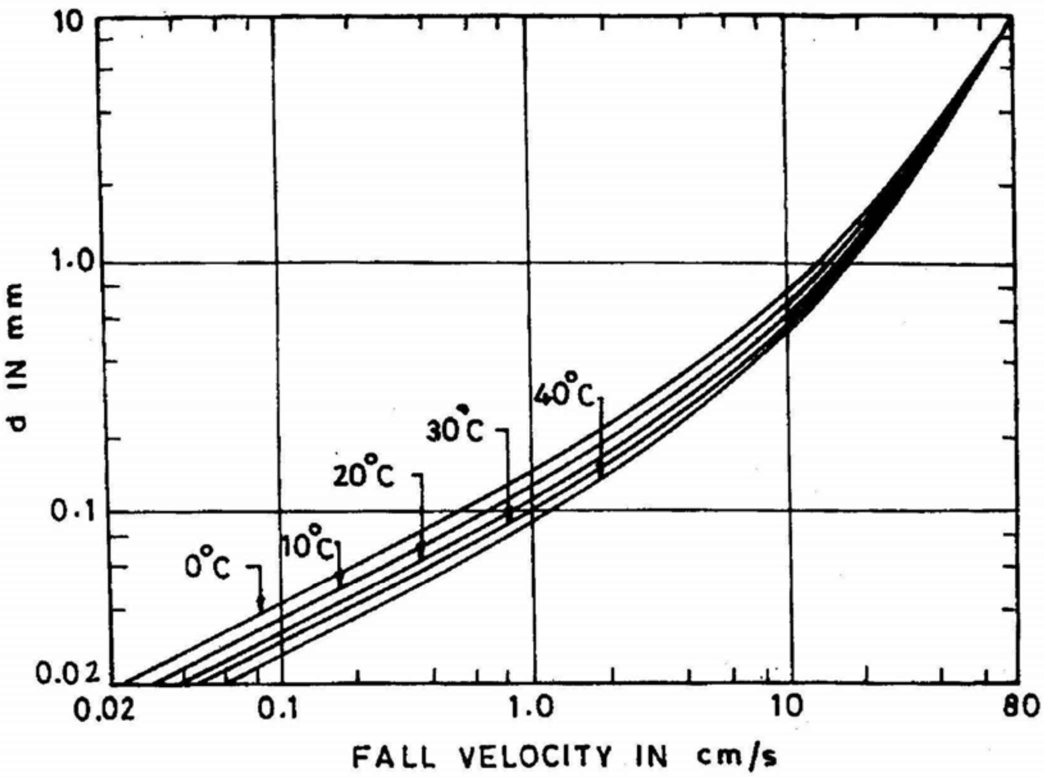


FIGURE 2.2.3.2
Fall velocity of spherical particles (relative density = 2.65) in water

c) Assume width of basin (B) and calculate depth (D) from the equation of continuity, whence:

$$D = Q/BU \qquad \text{(m)}$$

d) Adjust for effects of turbulence

$$\omega = \omega_0 - \omega' \ (m/s)$$

$$\omega' = \alpha U$$

$$\alpha = \frac{0.132}{\sqrt{D}}$$

e) Transit time $t = \dfrac{D}{\omega_0 - \alpha U} \quad (s)$

f) Length of tank

$$L = U.t \qquad \text{(m)}$$

$$L = \frac{U.D}{\omega_0 - 0.132U/\sqrt{D}} \qquad \text{(m)}$$

Vary the value of B to optimize the layout including: inlet section, settling basin and outlet section. A length to width ratio $\left(\frac{L}{B}\right)$ of 8 to 10 is preferable, but the minimum $\left(\frac{L}{B}\right)$ ratio should not be less than 4.0.

3.3.3 Outlet Section

The outlet section provides a transition between the settling tank and power canal. A transition with walls converging at 2:1 will be satisfactory.
Hydraulic losses can be estimated as:

$$h_L = 0.2\left(\frac{V_P^2}{2g} - \frac{V_T^2}{2g}\right)$$

Where: V_P = velocity in power canal (m/s)
V_T = velocity in settling basin (m/s)

3.3.4 Flushing system – Continuous Flushing Type

The recommended flushing system comprises a series of hoppers built into the base of the settling tank with side slopes of 1:1 leading to a central outlet at the bottom of the hopper. Flushing flow is withdrawn from the bottom of the hopper and controlled by a manually operated valve, one for each row of hoppers. The flushing system may be designed for either pressurized or non pressurized flow. Where head is available the non pressurized flow design is to be preferred since water passages can be made larger and therefore are easier to maintain. The usual design procedure is to assume equal flow through each hopper. Figure 2.2.3.3 shows a typical design (at end of text).

3.3.5 Flushing system – Intermittent Flushing Type

The same laws govern the design of intermittent flushing desilters, thus the main basin dimensions can be obtained using the same procedures as outlined in Sub-section 2.2.3/3.2. In place of hoppers used in a continuous flushing desilter a sufficient storage volume must be provided. Determination of this volume should be based on the incoming sediment load, trap efficiency and frequency of flushing. It is recommended that this volume be computed from the mean maximum monthly sediment load as measured or from comparable data from another plant operating in similar conditions with respect to sediment and water flows. In converting sediment flows in mass terms to volumes a relative density of 2.65 and a bulking factor of ×1.25 are should be applied. Trap efficiency can be calculated using Camp's *Sediment Removal Function* as given in Figure 2.2.3.4. The recommended flushing flow is $1.20Q_P$ and the flushing gates should be large enough so as not the throttle this flow.

3.3.6 References:

Water Power Development

Volume 2A: High – Head Power Plants
(Pages: 18-26).
By E. Mosonyi
Akadémiai Kiadó
Budapest, Hungary (1991)

Sediment Control at Intakes – A Design Guide
Edited by P. Avery
BHRA – Fluids Engineering Centre
Cranfield, England (1989)

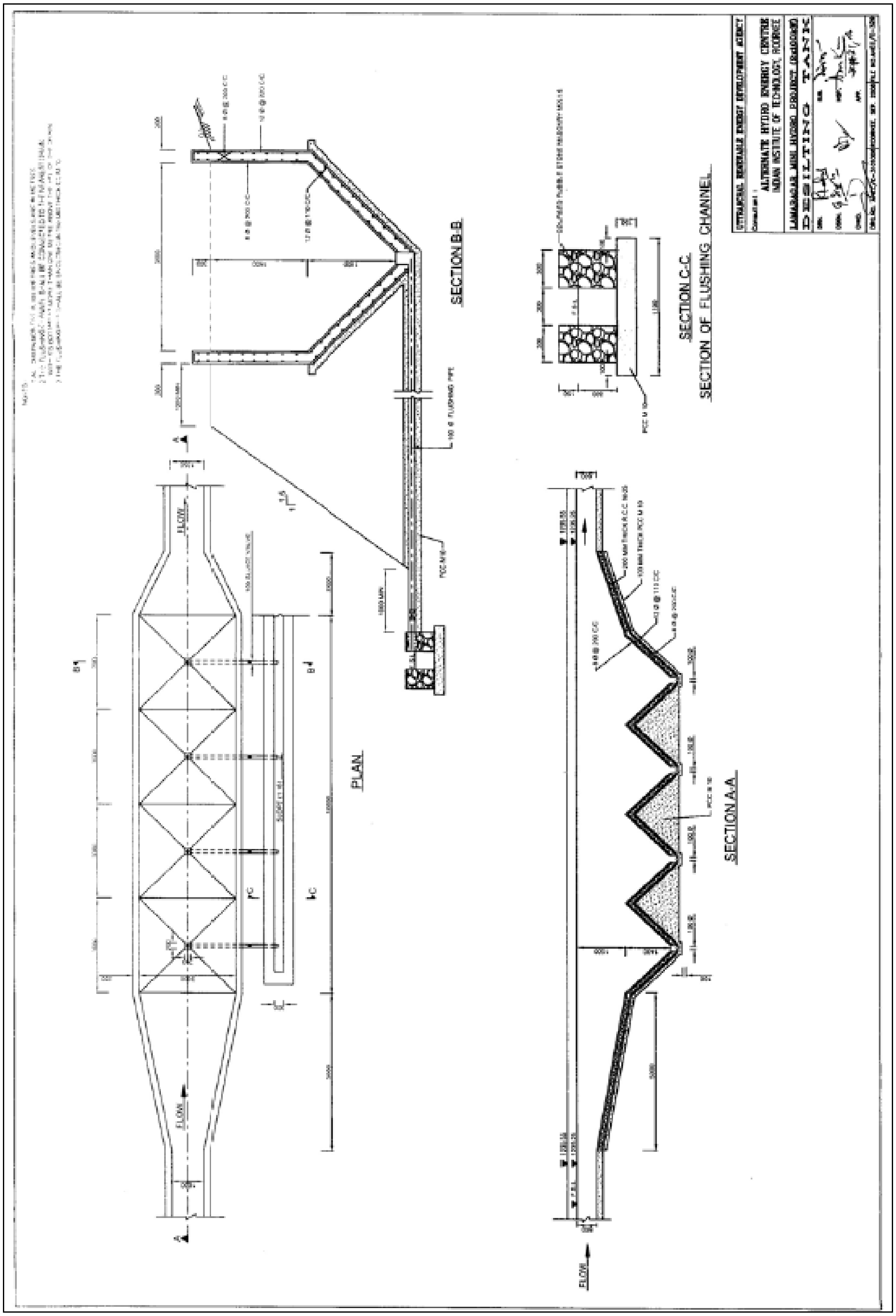
Sediment Erosion from Indian Catchments
By R.J. Garde and U.C. Kothyari
Proc. of 2nd International Workshop on Alluvial River Problems,
Roorkee, India (1985)

Silt Erosion Problems in Hydro Power Stations and *their Possible Solutions*
By B.S.K. Naidu
Published by the National Power Training Institute,
Faridabad (Haryana), 2004

Estimation of Repair Cycle of Turbine due to Abrasion by Suspended Sand and Determination of Desilting Basin Capacity
By Tsugo Nozaki –
Electric Power Civil Engineering (Japan)
Volume 218 pp 143-152 – January 1989.
(Original in Japanese)

3.3.7 Drawings

Drawings are shown on the following pages.



$Q_P = 0.798\ m^3/s$, $Q_F = 0.202\ m^3/s$, $Q_T = 1.00\ m^3/s$ & $d = 0.25\ mm$

FIGURE 3.3.3: TYPICAL HOPPER TYPE DESILTING TANK

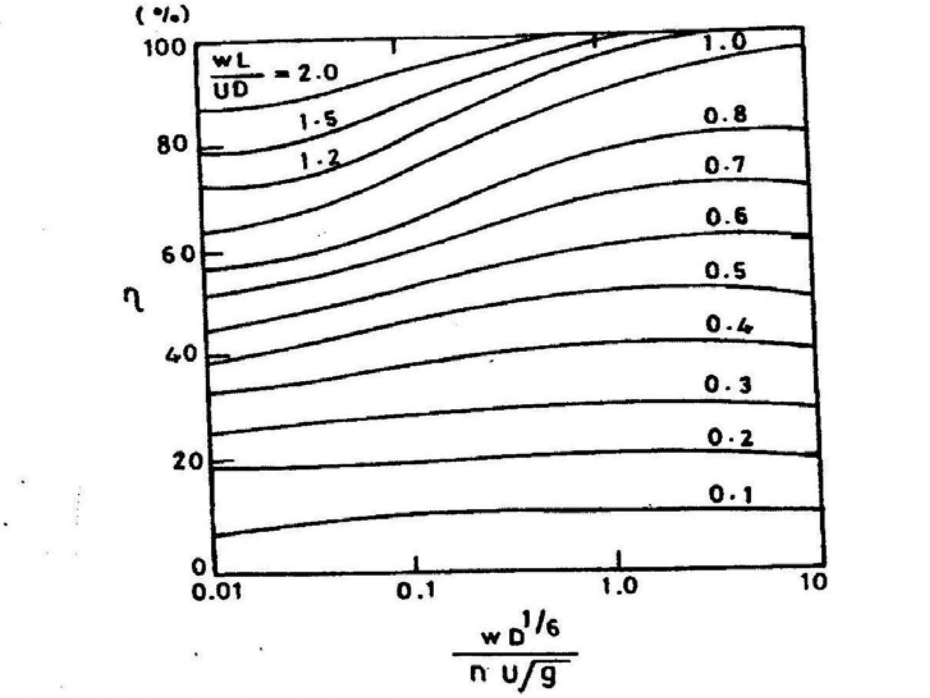


FIGURE 2.2.3.4
Camp and Dobbin's relation for efficiency of settling basin

Figure 3.3.5 & .6 shows a typical design for a flow of 15 m^3/s per chamber.

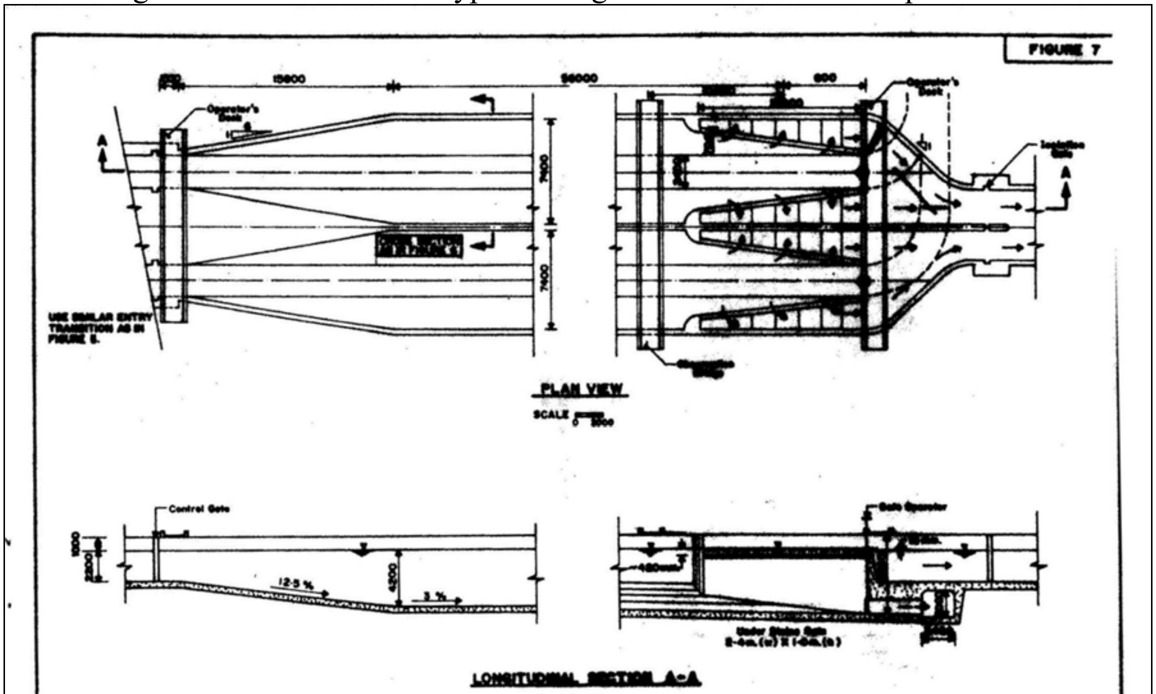
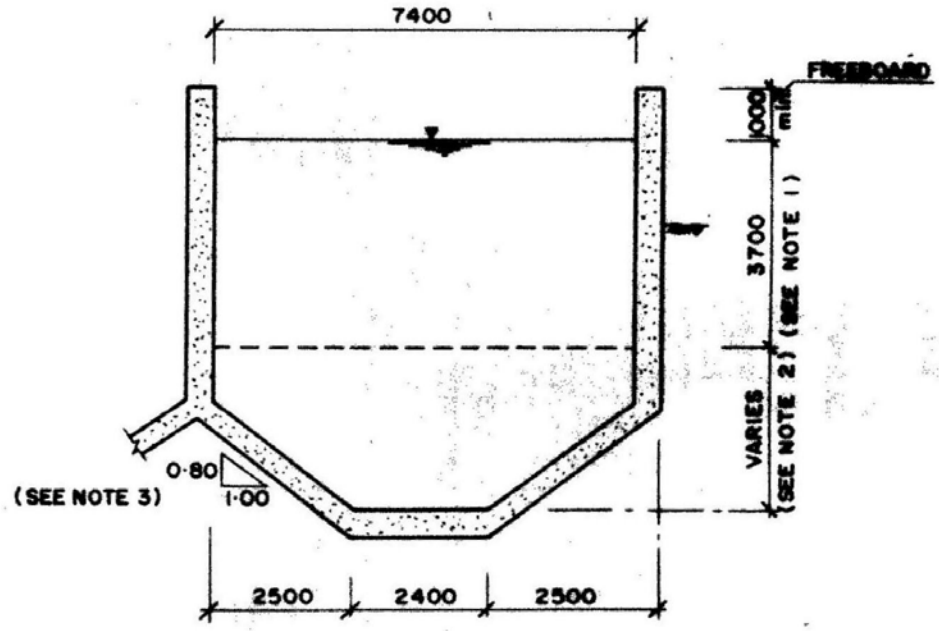


FIGURE 2.2.3.5
CONCEPTUAL

FIGURE 6



NOTES:

1. DESIGN DEPTH.
2. SEDIMENT STORAGE ZONE.
3. SIDE SLOPES SHOULD BE GREATER THAN THE ANGLE OF REPOSE OF DEPOSITED SEDIMENT, NORMALLY 35°- 40°.

FIGURE 2.2.3.6

TYPICAL CROSS-SECTION

DISCONTINUOUS DESANDER

4. HYDRAULIC DESIGN OF FOREBAY TANK

General

A forebay tank is normally located at the downstream end of the water conductor system and provides a transition between the power canal and penstock. It is usually located on a ridge on a firm foundation respecting topographical and geological conditions. Upstream from the forebay tank the waterway is characteristically open channel flow whereas downstream penstock flow is under pressure. The forebay design addressed in this sub-section is typical of designs associated with long canals where flow is controlled by the head gate and flow surplus to turbine demand is discharged over an escape weir back into the river. This section does not deal with designs having short canals where flow is controlled by the turbine (for this case the reader is referred to Sub-Section 2.2.1/1).

4.1 Function

These are two main functions:

- Provide for adjustment of turbine discharge according to load demand.
- Provide a volume of stored water to permit water level control of turbine operation.

Flow adjustment: the forebay tank and escape weir facilitate the adjustment of turbine discharge due to system load changes by diverting surplus flow over the escape weir back into the river. Normally in this mode of operation requires that canal flow be greater than plant demand flow.

Water level control: For small hydro plants connected to the grid it is convenient to match turbine output to available flow, thereby maximizing use of available water. This is achieved by means of a water level control system whereby the turbine load is adjusted to equalize available flow in the power canal with turbine flow. The forebay tank acts as a buffer to adjust for errors in turbine setting and actual inflow into the forebay tank. This requires that water levels be measured in the forebay tank and tailrace and transmitted in real time to the turbine governor which adjusts turbine output (and flow) so as to keep forebay water levels within a prescribed water level range.

For mini hydro plants equipped with load controllers, there is no feed back to the turbine, thus cost of water level gauges and data transmission systems is avoided. These plants always operate in a “water wasting” mode so that the forebay tank water level is always maintained above the escape weir crest elevation. This is not a problem during periods of high flow when river flow is much greater than plant demand. However, during low flow periods when plant flow capacity (Q_p) may be greater than river flow (Q) it would be necessary to adjust the ballast load to limit plant flow to about 90% of river flow in order to avoid draining the forebay tank.

4.2 Design Criteria - Tank

The following hydraulic design criteria are recommended:

a) The live storage volume of the forebay tank should be determined according to the response characteristics of the turbine governors. Normally a volume of $Q_p \times 120\ m^3$ (or two minutes at (Q_p= maximum plant flow) will be satisfactory for mechanical governors. For digital governors the control volume can be further reduced. In this case the engineer should contact the turbine manufacturer to define the control parameters in order to calculate the control volume needed.
b) A live storage drawdown of 1.0 m to 2.0 m below the crest of the escape weir is recommended.
c) The depth of the tank should be chosen so as to provide adequate submergence for the penstock intake in accordance with Sub-Section 2.2.1/4 of this standard. A linear alignment of power canal and penstock intake is preferred. As practical, the cross section areas of the forebay tank should be designed to avoid abrupt changes in direction which could cause undesirable vortex formation.

4.3 Escape Weir

The preferred location for the escape weir is in the rim of the forebay tank. Where this is not practical for topographic reasons the escape weir should be located at the nearest suitable site upstream of the forebay tank. For this case the effects of hydraulic transients in the power canal section between the forebay tank and escape weir should be checked to assess their impact on water level control.

A simple overflow weir is recommended with a design head that can be contained within the normal canal freeboard. Weir discharge should be routed towards a natural water course of adequate capacity or a ditch provided that is suitably protected against erosion.

4.4 Flushing Gate

A flushing gate is recommended by some designers to facilitate removal of any sediment or debris that might settle in the bottom of the forebay tank and be drawn in to the penstock.

4.5 Water Level Control

A water level control system requires that real time water level measurements in the forebay tank and tailrace canal be transmitted to the turbine governor. In the water level control mode the governor will estimate the inflow to the forebay tank and adjust the wicket gates to correct for difference between turbine and canal flows so as to maintain forebay tank levels within a prescribed range. A float type water level gauge with electronic data transmitter is recommended. The features of a float gauge are shown below:

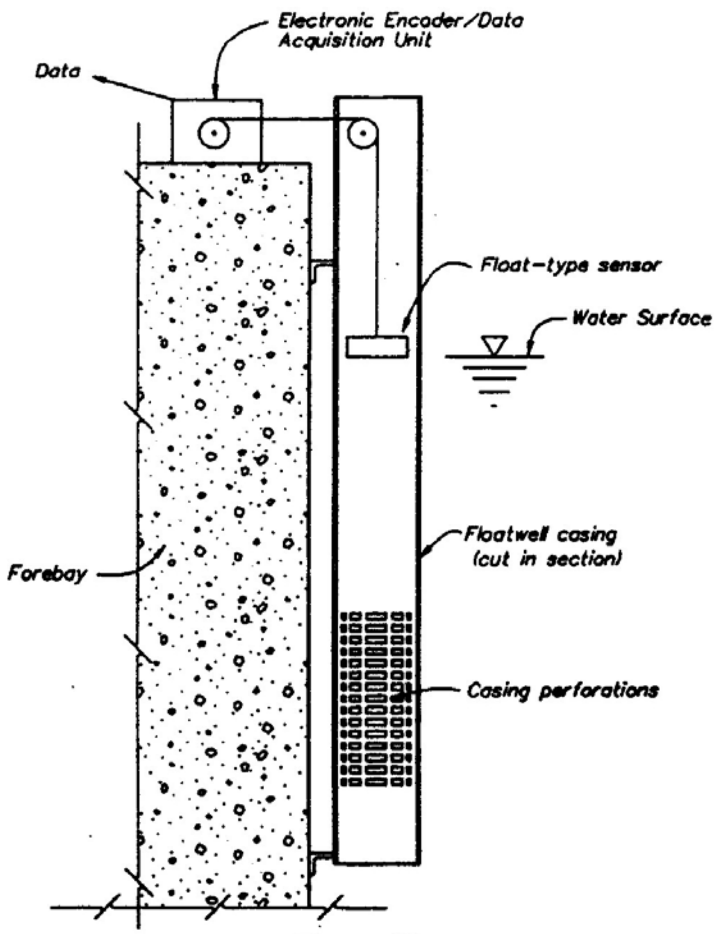


Figure 4.9
Typical Float Operated Floatwell

The precision should be +/- 3 mm or better. For additional information the reader is referred to IS 9116 (2002) "Water Stage Recorder (Flow Type) - Specification". For mini hydro plants it is recommended that a staff gauge be attached to the wall of the forebay to facilitate estimation of canal flow prior to start up of the turbines. Staff gauges or float wells should be located in areas of relatively quiet water to minimize risk of errors due to water level fluctuations.

4.6 References

IS Standards cited:
IS 9116(2002) *"Water stage Recorder (Float type) – Specification"*.

Other References:
"The Study on Introduction of Renewable Energies in Rural Areas in Myanmar" – Volume 4. Main Report: Manuals
www.lvzopac.jica.go.jp

5 CONTROL OF HYDRAULIC TRANSIENTS

5.1 BACKGROUND

The design of pressurized conduits must take into account the transient behaviour of the conduit / turbine system / power system. Water hammer pressures, turbine speed deviations and turbine/ generator runaway speeds must be kept within appropriate limits. These phenomena are interrelated in such a way that reducing the rate of wicket gate closing to control water hammer may result in excessive speed (frequency) deviations or unacceptably high turbine/ generator runaway speeds. Conversely, rapid adjustment of the wicket gate to minimize speed (frequency) deviations may result in unacceptably high water hammer pressures. Since these factors also impact the electrical system (grid) they must be controlled so that frequency and voltage deviations are maintained within strict limits.

Where plant characteristics are favourable no special means are required for dealing with the above problems. However, where situations are unfavourable some method of controlling water hammer and its related effects is required. The following criteria may be used to judge whether a surge tank or an alternative device is required for control of frequency and/or waterhammer. Note: H = gross head (m), L = length of penstock or section of penstock (m) and V = flow velocity (m/s).

- A surge tank may be required where *L/H > 4 to 8*
- *Or when* $\frac{\Sigma L_i V_i}{H} > 7 \ \ to\ 13\ (SI\ \ units)$
- A surge tank should be provided if the maximum speed rise following rejection of the maximum turbine output cannot be reduced to less than 45 % of the rated speed by other practical methods, such as increasing the generator inertia or penstock diameter or decreasing the effective wicket-gate closing time. The speed rise should be computed assuming one unit to be operating alone if there is more than one unit on the penstock. *USBR Engineering Monograph 20 provides an approximate method for estimating speed rise. (See Appendix 5 to this sub- section).*
- Experience shows that a turbine / generator will function satisfactorily if the pressure rise at the scroll case does not exceed 50% of gross turbine head for full load rejection. This situation should be checked for both high and low reservoir levels. *Sub-section 2.2.6 of this standard provides guidelines on calculating water hammer pressure in penstocks.*

The above criteria apply in particular to isolated plants where the capacity of the unit contributes more than 40% of the system capacity. Less stringent criteria may be considered for units connected to a large system where their role in frequency regulation is less important.

5.2 Methods For Control Of Hydraulic Transients

The use of a surge tank provides the most effective and reliable method for dealing with hydraulic transients; but it is also the most expensive. There are several lower cost approaches that can be applied for controlling waterhammer pressure rises and related generator speed deviations. Some approaches will provide protection against pressure rises but give little support for speed regulation. Savings in capital costs may be offset by increases in maintenance. Some alternatives are more reliable than others. The designer should weigh the advantages and disadvantages of each alternative before making a final choice as to which solution is best suited for a given project. The features of these alternatives are described below:

5.2.1 Increasing Conduit Flow Areas

Increasing the diameter (and flow area) of the penstock will improve control of hydraulic transients but this approach is rarely economic due to the increases in penstock cost.

5.2.2 Addition of Machine Inertia

Speed rise of the generator can be reduced by addition of inertia to a generator - turbine unit. This is easily achieved by the addition of a flywheel for horizontal axis machines or by the addition of mass to the generator rotor for vertical axis machines. According to Gordon and Whitman (1985) inertia of a vertical axis machine can be readily increased up to 2.5 to 3.0 times standard inertia. They also quote a rule of thumb stating that the cost of a generator would increase by 1% for each 4% of inertia added. For vertical axis generators the cost of increases in crane capacity and load bearing strength of the powerhouse structure must also be considered. Their paper provides an empirical method for assessing the feasibility of adding generator inertia for control of waterhammer and frequency regulation. A copy of this paper is provided in the appendix to this-sub section.

5.2.3 Bypass Valve

A bypass valve, as the name implies, can be used to divert flow past passed the turbine. The bypass valve is designed with a linkage to the turbine operating ring in such a way that the bypass valve opens synchronously as the wicket gates close. This allows the turbine to be closed quickly while diverting flow through the bypass valve thereby avoiding excessive waterhammer and generator speed rises. In effect the turbine wicket gate closure curve has two portions an initial fast closure rate, until the bypass valve is fully opened and a slower rate governed by closure of the bypass valve to the new operating position. A bypass valve having a capacity of 33% - 60% of turbine flow capacity is usually satisfactory. This approach provides good responses for loss off load situations, but load addition characteristics are less satisfactory as the turbine would only be able to take on load at a reduced (slow) rate. This type of bypass valve is sometimes referred to as a synchronous bypass valve as it operates in unison with the turbine wicket gates.

Figure 5.2.3 (next page) shows a schematic design of a turbine bypass valve system.

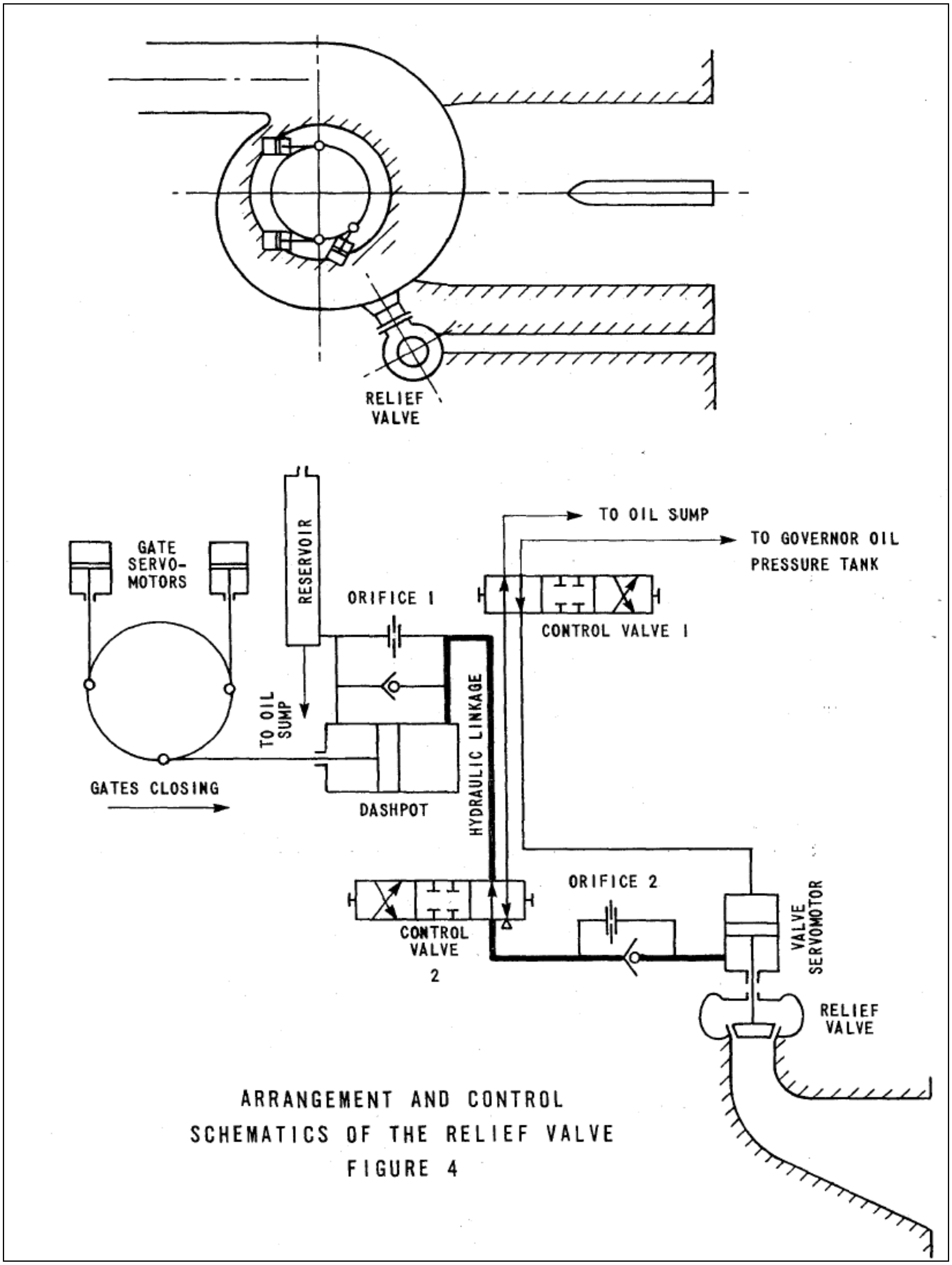


Figure 5.2.3: Schematic of Turbine Bypass Valve System.

5.2.4 Deflector Arm (Pelton Turbine)

Pelton turbines are usually provided with jet deflector arms. When a jet deflector arm is deployed it introduces a deflector bucket into the Pelton jet diverting the jet away from the runner. In this manner the hydraulic load on the runner is diverted and the generator speed rise is thereby limited; meanwhile, the needle valves can be closed slowly limiting waterhammer pressure rise. The effect is similar to the behaviour of a bypass valve. However, as for a bypass valve, the rate of load addition would be relatively slow. The operation of the deflector arms are be controlled by the turbine governor. This type of governor is called a "two part" governor since the governor must control both needle valve positions and deflector arm operation.

5.2.5 Pressure Relief Valve (PRV)

Pressure relief valves are sometimes employed to control excessive pressure rises due to waterhammer; such valves should be designed to open at a specified pressure and then re-close at a specified lower pressure. As releases may be at high pressure facilities for pressure dissipation, spray control and drainage are also required. In contrast to bypass (synchronous) valves PRV are activated by pressure rise.

5.2.6 Stand Pipe (Mini Hydro)

Pressure relief can also be provided by a stand pipe, which is essentially the riser pipe for a surge tank without the tank. The stand pipe top should be set at a height somewhat above the static water level. When surge level exceeds this elevation water starts to spill from the stand pipe; thus measures for erosion protection and drainage must also be provided. The height of the tank should be selected to accommodate small load rejection without spilling compatible with system operating requirements. (This approach is usually applied to mini-hydro plants).

5.2.7 Safety Membrane (Mini – Hydro)

For mini-hydro plants protection against excessive waterhammer pressures can be provided by safety membranes. Safety membranes, usually made of sheet aluminium and mounted on the penstock near of the power house are controlled points of weakness. When pressure on the membrane rises to a prescribed (design) level the membrane will burst to suppress further pressure rise.

Limited information is available on the design of safety membranes. It is reported that this device is widely used in China and gives reliable service. A paper by Kovalskii and Fedotov (1965), translated from Russian, gives a method for design.

5.2.8 Load Control Governors (Mini -Hydro):

Load control governors provide an effective means for control of frequency and waterhammer. A load controller switches the generator output the between the system load and a ballast load to equate the total load (system plus ballast loads) to the output of the turbine / generator unit. In effect the turbine / generator unit operates at its hydraulic (flow) capacity at all times and load changes are made without adjusting turbine flow or provoking waterhammer. This approach is usually suitable for mini-hydro plants but becomes less attractive for plants greater than about 1000 kW due to the expense of the ballast load.

5.2.9 Surge Tank

A surge tank provides a reliable solution that controls excessive waterhammer pressure rises and provides good speed regulation characteristics as well. It is the most expensive of the alternatives given in this section, but the most effective and reliable.

The main reason for considering construction of a surge tank is where a plant operates in an isolated system where both frequency and waterhammer control must be provide by the proposed hydro plant. In such a situations use of a surge tank may be required. On larger interconnected systems where the contribution to frequency control of a given unit is less important one of the other less costly alternatives (2.2.5/2.1 to 2.7) is likely to be satisfactory. Therefore, the role of the planned power plant should be carefully assessed through discussions with the power system operator and the most cost effective solution selected.

Details on the hydraulic design of surge tanks are given in the following sub-section.

5.3 HYDRAULIC DESIGN OF SURGE TANKS

5.3.1 Background

The main functions of a surge tank are:

- To reduce the magnitude of waterhammer pressures at the turbine by reflecting incident waterhammer waves at the surge tank, thus limiting the play of waterhammer to the section between surge tank and powerhouse rather than between reservoir (intake) and powerhouse.
- To improve the regulating characteristics of a hydraulic turbine. With a surge tank, the length of water column initially accelerated (or decelerated) is limited to the portion of conduit downstream of the surge tank junction to the powerhouse which is typically much shorter than the full length from intake to powerhouse.
- A surge tank provides storage for excess water on load rejection; while during load acceptance water can initially be drawn from this storage. This permits water in the upstream conduit to be accelerated without excessive drop in pressure in the penstock supplying the turbine.

There are three common types of surge tank used in hydropower plant design, the simple surge tank and two types of throttled surge tanks, differential and orifice types. The action of the simple surge tank is sluggish and requires the greatest volume. It is the most expensive and seldom adopted in preference to the other types. Modern designs usually employ either the restricted orifice, or differential type. The latter is a compromise between the simple and restricted orifice types. It decelerates flow less abruptly than the orifice type and transmits less waterhammer upstream. However the differential surge tank is more expensive than the orifice type by the cost of an internal riser. The choice depends on the extent of upstream conduit affected by waterhammer in a given case. The extent is mainly a function the ratio of orifice area to conduit area: where this ratio is large is favours the orifice type and where this ratio is small it favours the differential type. For a long conduit the length affected by waterhammer will be longer for an orifice type than for a differential type of surge tank. In such a situation the additional cost of a differential surge tank may be offset by savings in conduit steel. This issue is discussed further with reference to penstock design, as explained in Sub-section 2.2.6/7.1 of this Standard. Orifice design may incorporate geometry giving different head loss coefficients for inflow and outflow, typically the coefficient for inflow is higher than for outflow.

IS 7396 *"Criteria for Hydraulic Design Surge Tanks" Part 1: Simple, Restricted Orifice and Differential Surge Tanks"* provides detailed advice on the hydraulic design of surge tanks, including recommendations on design conditions (Clause 5.1). It is recommended that the designer follow this standard. Some additional comments and design suggestions are added for the designer's consideration.

IS 7396 provides a comprehensive methodology for the dimensioning and hydraulic design of simple, restricted orifice and differential surge tanks. The standard provides formulae for preliminary design, including tank diameters and maximum surge levels, but does not provide formulae for computing minimum surge levels (this omission is addressed in Sub-Section 2.2.5/5 Design Aids). **IS 7396** also recommends that the selected design be verified by detailed numerical calculations. The necessary formulae for such calculations are also given. Alternatively, the function of the combined turbine penstock surge tank system can be investigated using a simulation program such as WHAMO, developed by the U.S. Army Corp. of Engineers (USACE) and currently available over the internet. *(The main inconveniences of this program are that detailed turbine characteristics must be known and all data and results are given in U.S. customary units).*

IS 7396 recommends surge tank design should be based on "balanced design" – as recommended in Article 5.5.3.3, which reads: *The assumed area of the orifice should be so altered that the values of waterhammer pressure and pressure due to upsurge are nearly the same***.**

IS 7396 recommends the following factors of safety be applied to Thoma's area A_{th} and Thoma's adjusted area, As:

Types of Surge Tank	Factor of Safety
Simple	2.0
Restricted orifice or differential	1.6

With modern digital electronic governors, notably P.I.D. governors, the adjustments to flow can be made more precisely and smaller safety factors can be employed without risk of stability problems. Authors such as Chaudhry recommend factors of safety of 1.5 and 1.25 for simple and throttled surge tanks, respectively.

For mini hydro plants ($P \leq 1000$ kW) it is recommended that detailed numerical analysis be omitted and surge tank design based on graphical methods of Parmakian for simple and restricted orifice surge tanks and Creager and Justin for differential surge tanks. *(Some further work is required to confirm this recommendation, especially in the case of multi-unit plants where sequential load acceptance may produce a smaller down surge than the down surge produced by oscillation subsequent to the maximum surge on sudden load rejection. Comments requested)*.

For larger projects (1.0 MW to 25.0 MW) consideration should be given to more detailed computer simulation analyses to verify the feasibility of reducing the safety factors prescribed in IS 7396, especially where significant cost savings can be realized.

5.3.2 Other considerations

Structural design of surge tanks shall comply with applicable IS structural steel and concrete standards; the tank shall also be protected against internal corrosion by painting. Selection of paints, steel surface preparation and applicable shall be in accordance with the applicable norms.

5.3.3 REFERENCES

IS Standards cited:

IS 7396 (Part 1): *Criteria for Hydraulic Design of Surge Tanks.*

Other References:

Selecting Hydraulic Reaction Turbines
Engineering Monograph 20
U.S.B.R.

Water Hammer and Mass Oscillation (WHAMO) – Computer Program
USACERL ADP Report 98/129
Construction Engineering Research Laboratories,
U.S. Army Corps of Engineers

Generator Inertia for Isolated Hydropower Systems
J.L. Gordon and D.H. Whitman
Canadian Journal of Civil Engineering

Volume 12, Number 4 – 1985

*Applied Hydraulic Transient*s
H.M.Chaudhry,
Van Nostrand – Reinhold Co., New York (1978).

Bursting Safety Membranes
B.S.Koval'skii and V.P.Fedotov
(translated from Khimicheskoe I Neftyanoe Mashinostroenie, no. 6 - May 1965).

Hydroelectric Handbook
Creager and Justin
McGraw Hill, New York (ca. 1950).

5.4. DESIGN AIDS

Selected information is provided in the following appendices to supplement information provided in IS 7396 (Part 1). Included are methods for calculating minimum surge levels.

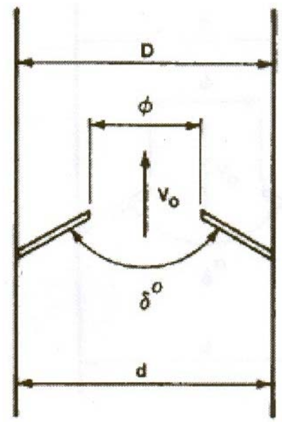
The following information is provided

- Formulae for determining loss coefficients of conical orifices
- Chapter 17 on design of restricted orifice surge tanks and design charts from
  Water Hammer Analysis by J. Parmakian.
- Paper by Gordon and Whitman on determination of generator inertia.
- Johnson's charts for estimating maximum up and down surges for differential surge tanks.
- USBR Engineering Monograph 20, Table 24

5.4.1 APPENDIX 1: EQUATIONS FOR LOSS COEFFICIENTS OF CONICAL ORIFICES

Streamlined in Direction of Flow:

Orifice geometry:



$$\alpha = \delta \;/\; 360°$$

$$\Phi = \left(\frac{\phi}{d}\right)^2$$

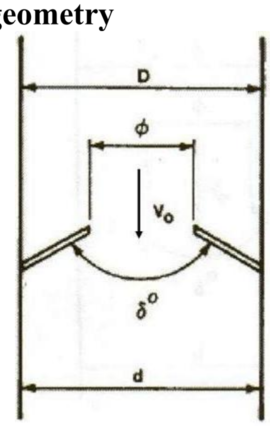
$$\psi = \left(\frac{\phi}{D}\right)^2$$

$$K = \left[\frac{1}{1-(1.5\alpha - \alpha^{3/2})\;(1-\phi^2)} - \psi\right]$$

$$H_{LO} = K.V^2 \,/\, 2g$$

5.4.2 Contrary Direction of Flow:

Orifice geometry



$$\alpha^{'} = 1-\alpha$$

(α as before)

$$\phi^{'} = \psi = \left(\frac{\phi}{D}\right)^2$$

$$\psi^{'} = \phi = \left(\frac{\phi}{d}\right)^2$$

$$K^{'} = \left[\frac{1}{1-(1.5\alpha^{'} - \alpha^{'3/2})\;(1-\phi^{'2})} - \psi^{'}\right]^2$$

$$H_{LO} = K^{'}.V^2 \,/\, 2g$$

(Source: ***Chambres d' Equilibre*** **by A. Stucky).**

Appendix 2: *Waterhammer Analysis – by J. Parmakian*

(For determination of maximum up and down surges for restricted orifice surge tanks).

CHAPTER XVII

Surge Tanks

67. Operation

A surge tank is often used at a power or pumping plant to control the pressure changes resulting from rapid changes in the flow. For example, when the turbine gates are closed at a power plant which is supplied by a long penstock, the water surface in the surge tank rises slowly above the original running level as the kinetic energy of the rejected flow is converted into potential energy. Such a conversion of energy reduces the rate of change of flow and the waterhammer in the penstock between the forebay and surge tank. Similarly, upon an opening movement of the turbine gates, energy is provided by the surge tank for the immediate demand of the turbine. This action reduces the waterhammer effects in the long penstock and assists the turbine to pick up its increased load more rapidly. At a pumping plant with a long discharge line, a surge tank can also be used to effectively control the pressure changes in the discharge line resulting from the shutdown or starting up of a pump. For example, following the sudden shutdown of a pump, the surge tank provides energy to reduce the rate of change of flow and the waterhammer in the discharge line. Upon starting a pump, most of the initial flow from the pump enters the surge tank and this action reduces the waterhammer effects in the long discharge line. In order to accomplish its mission most effectively, the surge tank dimensions and location are based on the following considerations:

(a) At a power plant where the turbine output is controlled by a governor, the surge tank must have sufficient cross-sectional area to prevent unstable action. In the event the area of the tank is too small, a load change on the turbine will cause continuous oscillations of the water level in the surge tank, possibly with increasing amplitude. This problem of surge tank instability is outside the scope of this treatment. In addition, the cross-sectional area of a surge tank at a power plant should be large enough that the magnitude of the surges will be small during normal load changes on the turbine. Otherwise, turbine speed regulation will be difficult or impossible.

(b) The surge tank should be located as close to the power or pumping plant as possible.

(c) The surge tank should be of sufficient height to prevent overflow for all conditions of operation unless an overflow spillway is provided.

(d) The bottom of the surge tank should be low enough that during its operation the tank will not drain and admit air into the turbine penstock or pump discharge line.

On high-head plants, where from other considerations it is necessary to place the surge tank at a considerable distance from the power or pumping plant, the farther the surge tank is away from the plant the less effective it will be. At such installations the waterhammer effects in the length of pipe between the plant and the surge tank should be investigated by the methods described in either Chapter XIV or Chapter XV.

68. Analysis neglecting hydraulic losses

Consider the simple surge tank installation shown in Figure 76 where the initial flow through the control gate is cut off rapidly. It is desired to find the maximum upsurge in the surge tank and the time at which this upsurge occurs. In order to present the phenomena in its most

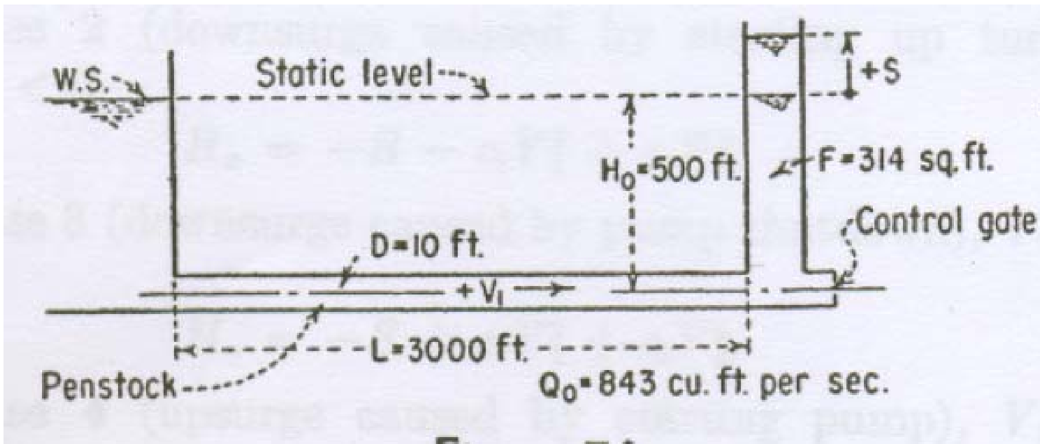


Figure 76

elementary form, the hydraulic losses and the velocity head in the pipe line are initially neglected. Moreover, the rigid water column theory of waterhammer is utilized since the effect on the upsurge of the stretching of the pipe walls and the compressibility of the water due to an increase in pressure is negligible.

Prior to the gate closure, the mass of water which is moving in the penstock is *LAw/g.* Upon gate closure the unbalanced force acting on this water column is *wAS*. From Newton's second law of motion the deceleration of the water column in the penstock is

$$-\frac{dV_1}{dt} = \frac{gS}{L} \qquad (60)$$

From the condition of continuity of flow following complete gate closure, the flow of water into the surge tank is the same as that out of the penstock, that is,

$$F\frac{dS}{dt} = AV_1 \qquad (61)$$

The simultaneous solution of Equations (60) and (61) is performed with the following boundary conditions: When t = 0.0, S = 0 and dS/dt = Q_o/F.

$$\text{Then} \quad S = \frac{Q_0}{F}\sqrt{\frac{FL}{Ag}}\sin\sqrt{\frac{Ag}{FL}}t \qquad (62)$$

$$\text{from which } S_{\max} = \frac{Q_0}{F}\sqrt{\frac{FL}{Ag}} \qquad (63)$$

and the time required to reach the maximum upsurge is

$$T = \frac{\pi}{2}\sqrt{\frac{FL}{Ag}} \qquad (64)$$

For the installation shown in Figure 76 the maximum upsurge in the surge tank above the static level due to the gate closure is computed to be 51.8 feet and the time required to reach this upsurge is 30.3 seconds. (62)

69. Analysis including hydraulic losses and throttling

Consider the surge tank system shown in Figure 77 where the positive directions of flow and surge are designated. The magnitude of the surge in the tank with the friction effects included will now be determined. The head tending to accelerate the water in the pipe line in the direction of the positive velocity V_1 is

$$H_a = -S \pm C_1 V_1^2 \pm c_2 V_2^2 \qquad (65)$$

where the signs of the last two terms depend on the direction of V_1 and V_2.

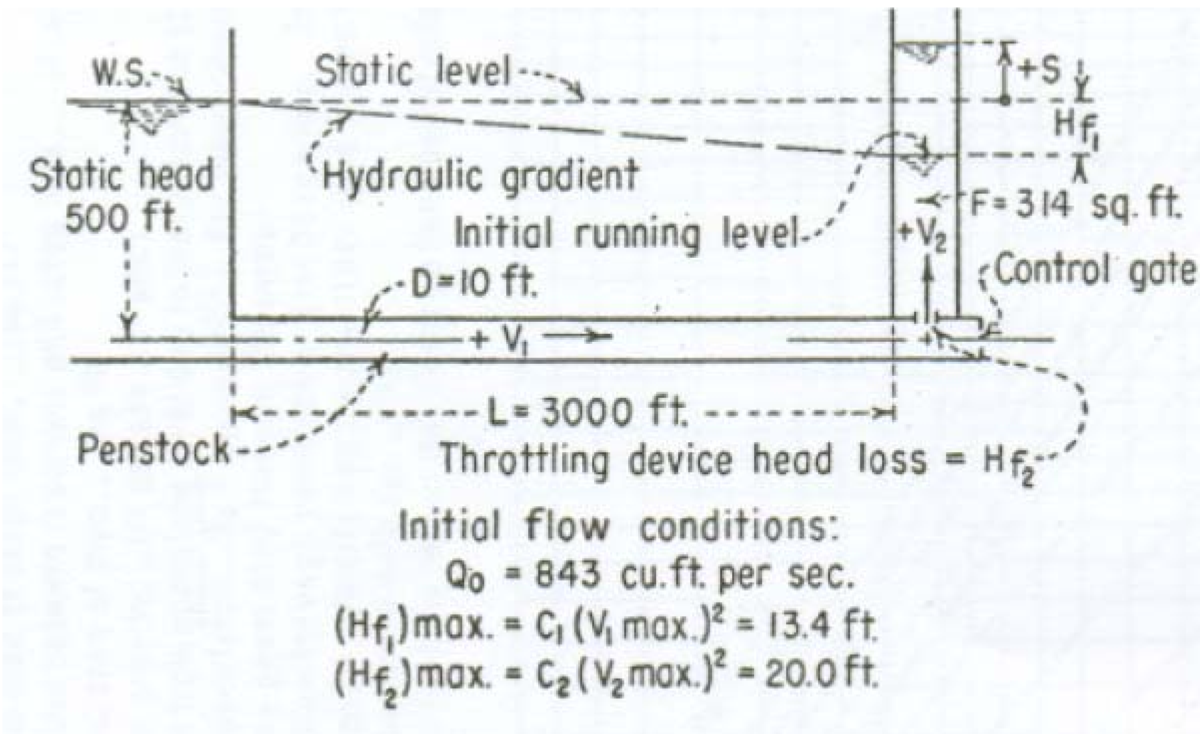


Figure 77

In this equation c_1 is a constant such that $c_1 v_1^2 = H_{f1}$ and represents the sum of the entrance loss, pipe line friction loss, and velocity head in the pipe line. Then c_2 is a constant such that $c_2 V_2^2 = H_{f2}$ and represents the throttling loss for the flow into or out of the surge tank. The following tabulation gives H_a for four possible cases:

(a) Case 1 (upsurge caused by turbine shutdown), $V_1 > 0$, $V_2 > 0$,

$$H_a = -S - c_1 V_2^2 - c_2 V_2^2 \qquad (65A)$$

(b) Case 2 (down surge caused by starting up turbine), $V_1 > 0$, $V_2 < 0$,

$$H_a = -S - c_1 V_1^2 + c_2 V_2^2 \qquad (65B)$$

(c) Case 3 (down surge caused by pump shutdown), $V_1 < 0$, $V_2 < 0$,

$$H_a = -S + c_1 V_1^2 + c_2 V_2^2 \qquad (65C)$$

(d) Case 4 (upsurge caused by starting pump), $V_1 < 0$, $V_2 > 0$,

$$H_a = -S + c_1 V_1^2 - c_2 V_2^2 \qquad (65D)$$

The mass of fluid in the pipe line being accelerated is $\omega AL/g$ and its acceleration at any time is dV_1/dt. Then from Newton's second law of motion

$$\frac{dV_1}{dt} = \frac{g}{L} H_a \qquad (66)$$

For continuity of flow

$$V_2 = \frac{AV_1 - Q}{A_2} \qquad (67)$$

and

$$\frac{dS}{dt} = \frac{AV - Q}{F} \qquad (68)$$

By substituting Equation (65) into (66) and using (67) and (68) to eliminate V_1 and V_2 a differential equation is obtained in S and t. By suitable changes in variable this equation reduces to the following form:

$$\frac{d^2 S_2}{dt_1^2} \pm \frac{1}{2}\left(\frac{dS_2}{dt_1}\right)^2 + 2b\left(\frac{dS_2}{dt_1}\right) + S_2 = 0 \qquad (69)$$

In this equation

$$b = \pm \frac{c_1 Q}{A}\sqrt{\frac{Fg}{AL}} = \pm \frac{H_{f1}}{Q}\sqrt{\frac{Fg}{(L/A)}}$$

Now S_2 is a function of S, and t_1 is a function of t. For example, Case 1 for turbine shutdown reduces to the following differential equation:

$$\frac{d^2 S}{dt^2} + \frac{Fg}{AL}\left(c_1 + c_2\frac{A}{A_2^2}\right)\left(\frac{dS}{dt}\right)^2 + 2c_1\frac{Qg}{AL}\frac{dS}{dt} + \frac{Ag}{FL}\left(s + c_1\frac{Q^2}{A^2}\right) = 0$$

(70)

The substitutions

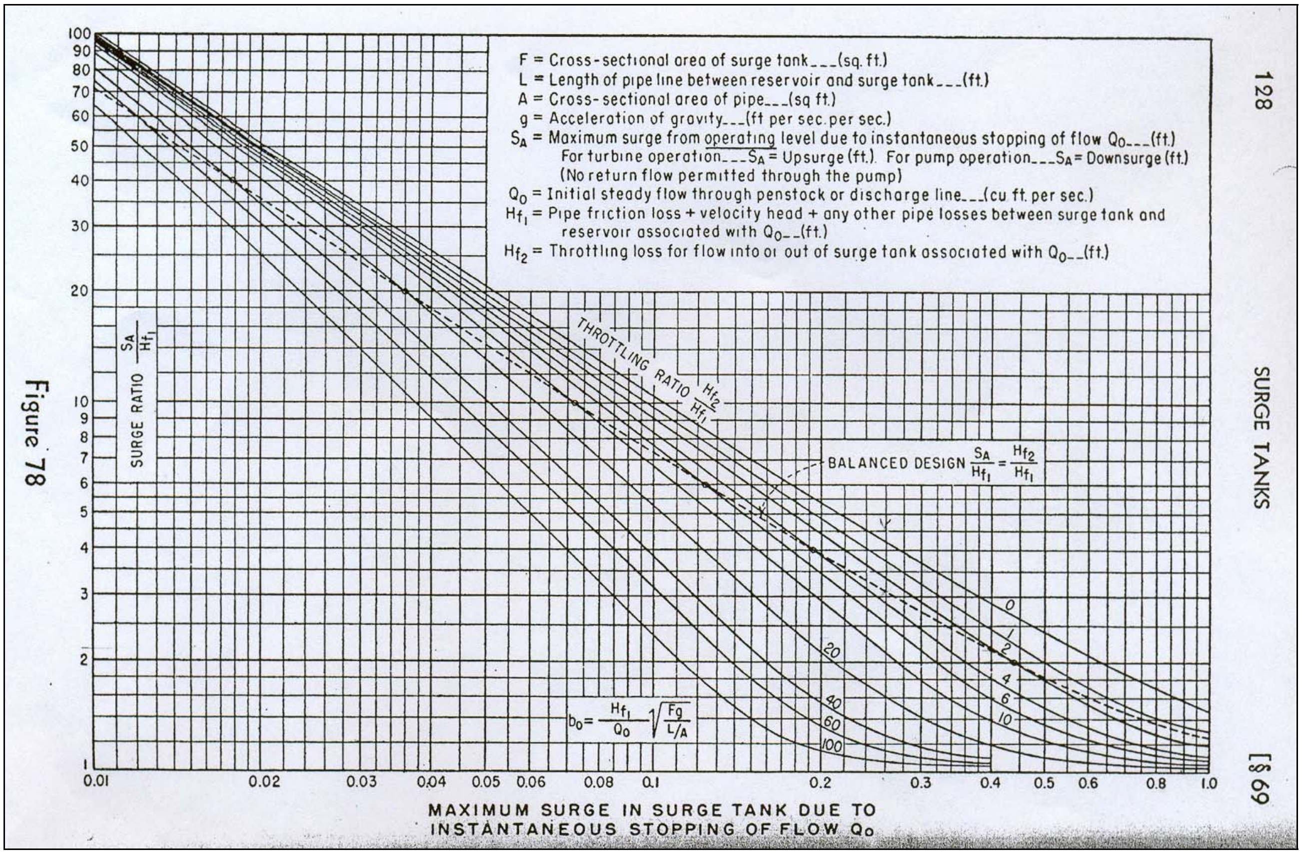
$$S = S_1 - c_1 \frac{Q^2}{A^2},$$

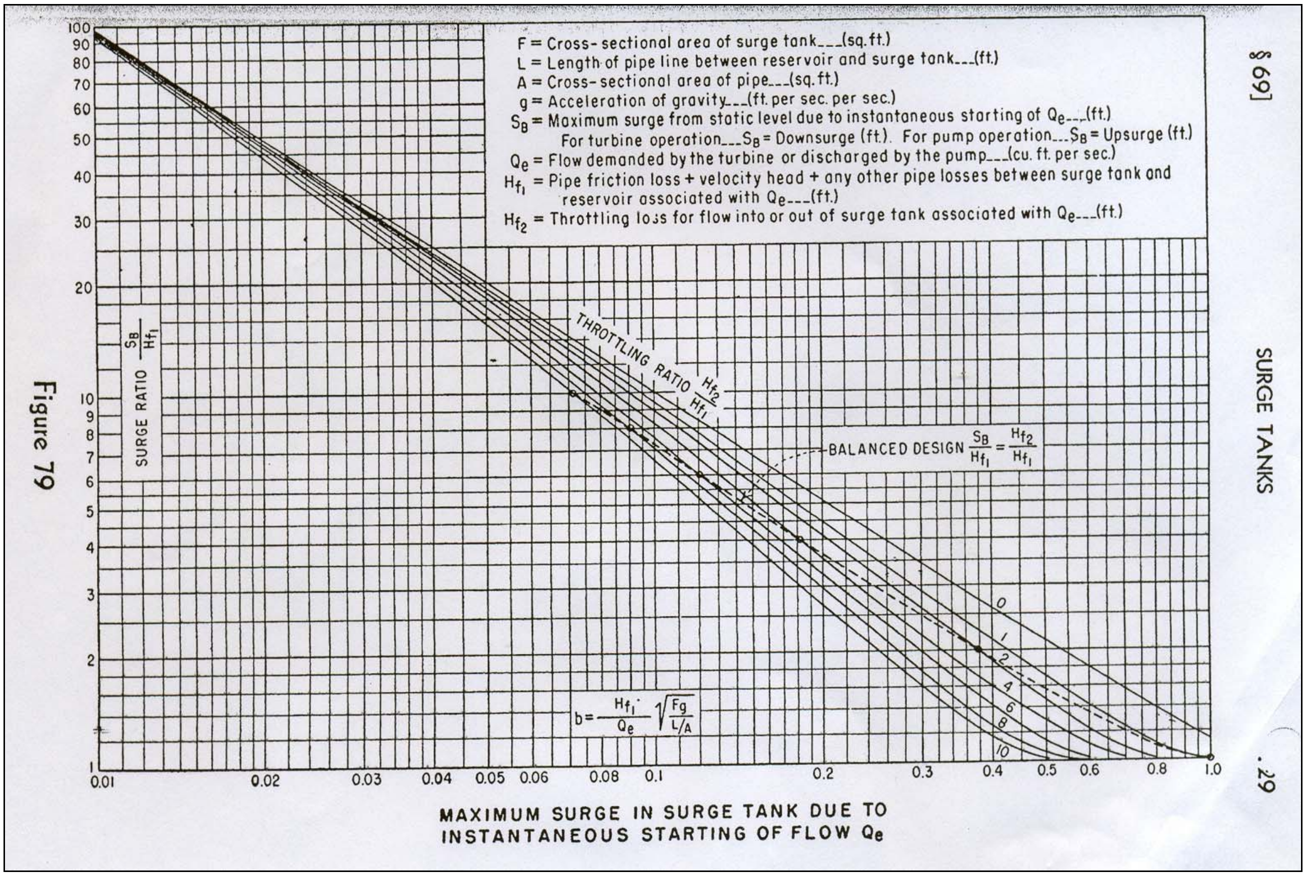
$$S_1 = \frac{ALS_2}{2Fg[c_1 + c_2 (A^2 / A_2^2)]}$$

and $$t = \sqrt{\frac{FL}{Ag}} t_1,$$

reduce Equation (70) to one form of Equation (69). The solutions[2] of Equation (69) for the four special cases of turbine and pump operation are given in Figures 78 and 79.

[2] See Reference 19.





Appendix 3: Generator inertia for isolated hydropower systems

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Abstract: Speed regulation of hydroelectric power plants of isolated systems is a complex subject, which is now becoming more important as customers install computers, stereophonic equipment, and advanced satellite dish electronic equipment in such systems. This paper presents a methodology for determining hydroelectric generator inertia, based on theoretical analysis, coupled with a review of data from over 50 hydroelectric projects with units having capacities between 2 and 300 MW. The parameters that affect generator inertia-system size, allowable frequency variation, type of load, turbine and governor, water column start time, governor time, and relief valve operation – are all discussed. A chart combining these parameters is developed, on which data from hydro projects is plotted. From an analysis of the plotted data, an empirical equation is developed for the generator inertia as a function of the aforementioned parameters.

Key words: hydroelectric power, generator inertia, speed regulation, hydro design.

Introduction

During design of a hydropower project, there is no greater interdisciplinary problem than that of selecting the required generator inertia. Its scope affects the work of electrical, mechanical, and civil engineers: electrical through the generator and controls; mechanical through the turbine, governor, and powerhouse crane; civil through sizing of the water passages, layout of the powerhouse, and support of the powerhouse crane. However, it is the civil engineer to whom this paper is directed, mainly because it is the civil engineer who is most directly affected, and has control over the major parameters that influence selection of generator inertia. First, the power-systems engineer will determine the allowable frequency deviation, and then the civil engineer, with some help from the turbine-generator engineer, will have to develop a layout and equipment configuration that will meet the frequency requirements. The options available to the designer for improving frequency regulation include use of a surge tank, locating the surge tank closer to the turbine, using larger water passages to slow down the water velocity, using faster governor times, with consequent higher waterhammer, using a relief valve on the turbine, and adding inertia to the generator. All of these alternatives add cost; hence determining the optimum configuration will require a great deal of study.

The inertia requirements for hydropower generators have received very little attention, due to the fact that on a large interconnected electric power system the governor is rarely needed to counter a frequency deviation, since the large inertia of the interconnected system keeps frequency deviations within a fraction of 1 Hz (Schleif 1971). When connected to such systems, the generator usually has a minimum inertia, often referred to as "standard inertia" (Westinghouse 1959), which has a value of

[1] $$GD^2 = 310\ 000\ (\text{MVA})^{1.25}\ N_s^{-1.875}$$

This formula was used to check the inertia ratio *J* of over 120 generators, ranging in size from 615000 kV.A down to 300 kV' A, and all except one, at a value of 0.98, were found to have inertias equal to or higher than the minimum indicated by [1].

It is only when there is a disturbance to the system that inertia comes into use. If a storm should interrupt incoming power on a transmission line, the sudden loss of generation will cause a major frequency drop, which will result in a rapid load-on at the remaining power plants on the system. This is when inertia becomes valuable, with larger inertia reducing the magnitude of the frequency excursion.

On large systems such faults due to storms are an infrequent occurrence; hence the cost of adding extra inertia usually cannot be justified. However, on smaller systems normal changes in load often become a significant proportion of the total system capacity, and the amount of inertia must be carefully assessed in order to avoid excessive frequency deviations. This simple fact was brought to the authors attention by an incident that occurred shortly after two 5.6 MW Kaplan units at a power plant in north western Canada, installed to provide power for an adjacent town, were commissioned.

Previously, the town power generation was by large, slow-speed diesels. To take advantage of the new hydropower source, the local hospital converted a water heating boiler from oil to electricity. Whenever the 4 MW boiler started up, the sudden load application caused the frequency to drop, and the power plant automatic under frequency relays initiated breaker opening to disconnect the power plant source from the town, resulting in a temporary blackout.

The problem was solved by changing the boiler controls, so that the load was added in 1 MW steps with a time delay between steps. Another solution would have been to install generators with a higher inertia, sufficient to keep the frequency deviation within about 1 or 2 Hz, thus avoiding tripping of the under frequency relays. This solution would have required implementation during construction, and would have been too costly. However, the incident does serve to illustrate the type of problems that can arise in isolated systems when the size of the load application relative to the generator capacity is not taken into account.

For this development, the inertia of the two generators was based on an approach outlined by NEMA (1958), using a formula for unit inertia with functions for unit speed, capacity of the generator, and water column start time only, as follows:

[2] $T_{ml} + T_{m2} > 100T_w(MW)^{-1}$

with the size of the unit varying from a maximum of 50 MW to a minimum of 20 MW. There is no allowance in this formula for such factors as the governor time and the magnitude of the load change, both of which have a very important bearing on the reaction of the turbine - generator unit, and hence the extent of the temporary frequency deviation.

From this incident, the authors realized that a more comprehensive approach was required, and therefore developed a preliminary version of the analysis outlined in this paper. It has been applied with success for over 20 years to generators powered by reaction turbines. For impulse units, it was initially believed that the approach was not correct, due to the different governing mode on load rejection at an impulse unit. This conclusion was reached after applying the methodology to a small, isolated power system in the high Andean mountains of South America, where several impulse unit power plants supply a city and a few small industrial loads. The methodology indicated that the system was not stable when subjected to a major load change, such as that caused by loss of generation at one of the plants due to a fault. However, the system appeared to be operating correctly. It was not until 1968, when one of the authors visited the area and enquired as to what happens when one of the power plants drops off the system, that the system operating problems became apparent. On loss of generation the whole system shuts down because under frequency relays trip out at substations. Between 1/2 and 2 h was usually required to reconnect the system. More recently, the analysis has also been used for impulse units.

The methodology developed in this paper will enable to the designer of a hydro power plant to determine the minimum requirements for generator inertia, thus avoiding the cost of excessive inertia, and will also permit comparison of the selected inertia with that at other hydro plants with similar operating criteria.

The cost of inertia

The inertia of a generator can be increased up to about 2.5–3.0 times standard inertia. With vertical shaft units, inertia is added to the generator rotor by either increasing the diameter, or the weight, or a combination of both (Gordon 1978). A general rule of thumb states that the cost of a generator increases by 1% for every 4% increase in inertia. In addition, the extra cost of the powerhouse superstructure, and perhaps the substructure required for the larger, heavier generator, must also be taken into account.

In practice, for a particular manufacturer, the cost of extra inertia is small provided the additional inertia can be fitted into the same generator frame size. There will then be a step incremental cost for the larger frame size for the next increment of inertia. However, since manufacturers work with different frame sizes, these step increment costs will occur at different points, thus smoothing out the cost increments in a competitive bidding situation.

For small horizontal units, extra inertia is usually added with flywheels and the cost increment is lower, but space requirements are significantly larger than that I for a comparable vertical-shaft unit. By the time all costs are included, it is probable that a 4% increase in inertia will add a cost equal to about 2% of the generator cost. For economy, it is therefore essential to keep generator inertia to an absolute minimum.

Measures of inertia

For the convenience of readers, formulae for inertia are given in both metric and American units. In American units the inertia is termed WR^2, in foot pound units, as weight times radius of gyration squared. In metric units, inertia is termed GD2in tonne metre units, as weight times diameter of gyration squared. The relationships between them is

[3] $WR^2\ (Ib\text{-}ft^2) = 5932GD^2\ (t.m^2)$

For a generator, the common measure of inertia is the H factor (Hovey 1960), which has a value of

[4] $H = 0.231\times 10^{-6}\ (WR^2)\ N_s^2\ (KVA)^{-1}$

in foot pound units. Alternatively it can be expressed in tonne metre units as

[5] $H = 1.37 \times 10^{-3}\ (GD^2)\ N_s^2\ (KV\ A)^{-1}$

H is the inertia constant, in kilowatt seconds per kilovolt ampere. It usually has a value ranging between 1 and 4.

Another measure of inertia is known as the unit mechanical start-up time T_m(USBR 1954). In this case the inertia value is for the entire rotating mass, including turbine runner and any flywheel. The mechanical start-up time is measured in seconds and represents the theoretical time required for the unit to reach synchronous speed when accelerated by a force equal to the full load output of the turbine. The start-up time is given by the following equations:

[6] $T_m = 0.621\times 10^{-6}\ (WR^2)\ N_s^2\ (HP)^{-1}$

[7] $T_m = 2.74 \times 10^{-3}\ (GD^2)\ N_s^2\ (KW)^{-1}$

The inertia constant and the unit start-up time are obviously related. By comparing [5] with [7], it will be seen that

[8] $T_m = 2H$

when generator rating in kV. A is equal to turbine capacity in kW, and neglecting the inertia of the turbine runner.

Use of generator inertia constant

The generator inertia constant can be used to quickly calculate an approximate value for the unit speed deviation for sudden pulse load changes (Moore 1960), assuming that there is no reaction from the turbine governor, based on the following equation:

[9] $N_2\, N_s^{-1} = 0.5\ (KW)\ tH^{-1}\ (KVA)^{-1}$

For example, assume a generator rated at 40000 kVA with an H value of 2.5, and a pulse load of 5000 kW applied for 2 s. The speed deviation will then be

$$N_2 N_s^{-1} = 0.5 \times 5000 \times 2 \times 2.5^{-1} \times 40000^{-1} = 0.05$$

In a 60-cycle system, this would mean a speed deviation of 60×0.05 = 3 Hz. If there are several generators on the system, the total kilowatt seconds of flywheel effect are simply added together. In the above example, if there had been five generators, the frequency deviation would reduce to 1% or 0.6 Hz.

As mentioned previously, this method of calculating speed deviations is approximate since it does not allow for (1) the action of the governor, or (2) the rotating inertia of the connected load, both of which will reduce the magnitude of the speed deviation. Another method of calculating the speed deviation, which allows for governor action, has been published (Gordon and Smith 1961), and nowadays there are several computer programs available that take into account the action of modern electronic governors.

Factors affecting inertia selection

There are eight basic factors that must be taken into account when determining the amount of inertia in the generator. These are (1) the size of the system, (2) the allowable frequency excursion, (3) the type of load, (4) the type of turbine, (5) the type of governor, (6) the water start time, (7) the governor time, and (8) the relief valve operation. Each of these factors is discussed as follows:

The size of the system – As mentioned previously, large systems have excellent frequency control, so that the addition of inertia for frequency regulation can be neglected except in the case of system fragmentation. However, for small systems with a total installed capacity of about 15 or 20 times the magnitude of the load change, some attention has to be given to unit inertia. Another factor is the number of generators connected to a system, and the size of the largest generator on the system. If the system has only a few generators, the largest frequency deviation will probably be caused by dropping the largest fully loaded generator, leaving the other generators to cope with a large sudden increase in load. This conclusion can be reached by using [9] to determine the approximate frequency deviation, and then using judgement to determine whether further investigation is necessary. In [9], the value used for time t should be equal to about one half of the governor response time required for the load change, for the unit on the system used to control frequency.

The allowable frequency excursion – Before the advent of electronic computers, stereo systems, television, and microwave equipment, frequency excursions of up to two or three cycles were acceptable. However, nowadays, frequency excursions of more than one-half cycle can cause problems, particularly to high-speed

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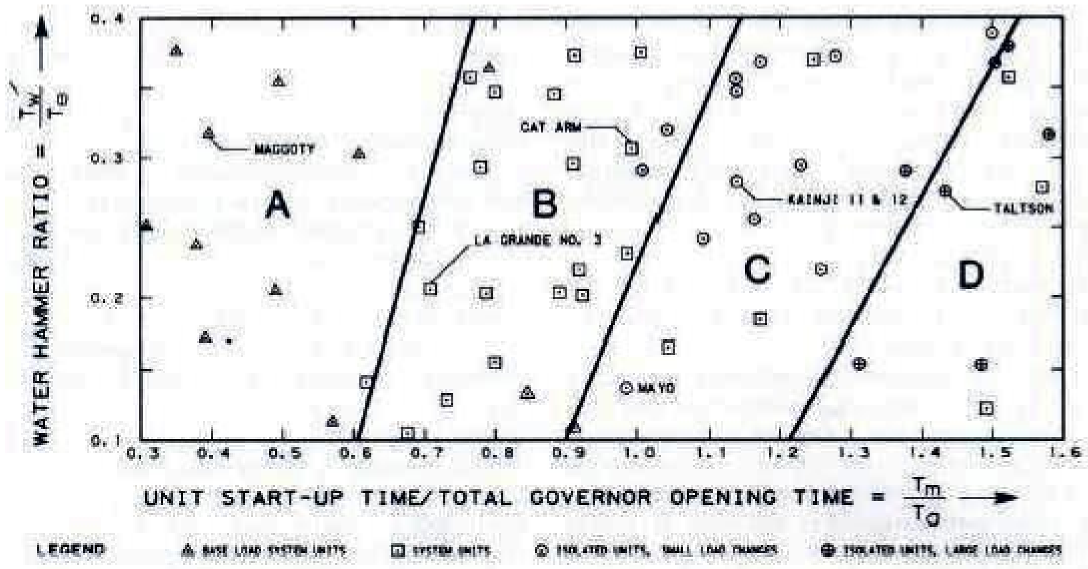

FIG. 1. Chart showing relationship between T_w, T_e, T_m, T_g, and type of development.

paper machines, which require an almost constant frequency if breaks in the paper roll are to be avoided. Again, [9] can be used to determine whether frequency excursions are likely to be within tolerable limits.

The type of load – If the load consists of a town with only small industrial establishments, sudden load changes will be small and inertia can be kept to a minimum. On the other hand, if the load consists of large electric arc furnaces, or electric-powered shovels in an open pit mine, or a deep underground mine with a high-powered shaft hoist, very large sudden load changes can be expected. With shovels, several can commence excavation of the ore body at the same instant, with the motors demanding full stalling torque, which is then removed from the system a few moments later. Large shaft hoists usually have a large power demand on starting and acceleration, followed later by generation of power on braking to decelerate and stop. These varying and pulsating types of load to not contribute towards system stability, and will require a detailed examination of unit inertia. Another factor is the rotating inertia of the load. This is usually about 10- 25% of the connected generator inertia; however, it cannot be determined with any accuracy, and can be neglected in an initial appraisal, resulting in a more conservative answer.

The type of turbine – With a reaction turbine, the governor controls flow of water through the unit, and hence power, by means of the wicket gates. **In**a Kaplan unit the blades are also moved, but at such a slow rate relative to the wicket gates that their effect can be neglected. However, in an impulse unit, the frequency excursion on load rejection can be kept within an extremely small value by rapid action of the jet deflectors. Hence an impulse unit will have a better response to pulse load changes than a reaction unit, but response to a large load increase will be about the same as that with a reaction unit.

The type of governor – Currently it is possible to purchase either mechanical governors, which measure speed and speed deviation (two elements), or electronic governors, which measure speed, rate of change of speed, and speed deviation (three elements). Electronic governors are more precise, allow use of longer water start-up times (Howe 1981), and have more adjustments, permitting a better matching of the governor response to the nature of the load change. However, as demonstrated by Ransford (1983), the response of a three-element governor to a large load change is very similar to that of a two-element governor.

The water start time – This is the theoretical time required to accelerate the water column to the velocity at full turbine load. It can be calculated from the following equation:

[10] $T_w = ((\Sigma LV)\, g^{-1} h^{-1}$

where Σ LV is the sum of the length times velocity for the water conduit upstream of the turbine, to the reservoir, or surge tank. (Note that in this particular analysis, the LV of the draft tube is not included, but is usually included in a governor stability analysis.) As the water start time increases, so does the governor time, resulting in a more sluggish response of the governor, and larger frequency deviations.

The governor time – The response time of the governor is of prime importance, since the faster the movement of the governor, the smaller will be the frequency deviation. There are two measures for the governor time, the effective time T_e, and the total time, T_g. The effective time is the time taken to move the wicket gates or needle valves through a full stroke with no cushioning at the ends of the stroke. The total time is the full stroke time including cushioning. Usually, the total time is equal to the affective time plus a few seconds. Also, the effective time T_e varies from a minimum of 2.7 times the water start time T_w for a maximum water hammer in the region of 50% to about 10 times T_w for a water hammer of about 10%.

The relief valve operation – Relief valves are usually added to a turbine to limit water hammer on long conduits during load rejection. They can be used to limit frequency deviations if operated in a water-wasting mode. In this mode the valve operation is synchronized with the wicket gate movement so that when the wicket gates open the valve closes and vice versa. However, this results in a large loss of water and hence is rarely cost-effective. Furthermore, maintenance costs for the relief valve will be excessive; hence relief valves are not recommended for limiting speed deviations.

For an isolated system, the response of the unit to a large load-on condition becomes the prime criteria in assessing unit performance. If the unit responds well to load acceptance, the response to load rejection will be equal or better. On this basis, several of the factors that affect unit performance can be neglected for the following reasons:

– Turbine type can be discarded since response to load on is similar for impulse and reaction units.
– Governor type can be discarded since response to large load changes is similar.
– The relief valve option can be discarded since its use is not recommended for speed regulation.

The problem now becomes one of developing an analysis that takes into account all of the remaining factors, namely, system size, frequency excursion, type of load, water start time, and the governor time. If the results of such an analysis are plotted, the chart could then be used to compare the relative performance of units on different systems.

Load-on speed deviation

The equation that has been developed for speed deviation during a part load change is

$$[11] \qquad N_2^2 \, N_s^{-2} = 1 - TT_m^{-1} \left[2P_2 - (P_1 + P_2)(1 - h_W)^{1.5} \right]$$

For a defined load-on this equation indicates that the speed deviation will become a function of two parameters:

- The ratio of T/T_m depends to a great extent on the type of governor (mechanical, electronic, two element or three element) and the magnitude of the load change. In order to simplify the problem, the authors have found that the ratio can be approximated, for comparison purposes, by using T_g / T_m where T_g is the total governor stroke including cushioning, with the longer time so obtained used to allow for the slower rate of response of a governor to part load changes. A chart showing the part load response rate of a typical mechanical governor has been published elsewhere (Gordon and Smith 1961). The ratio has been inverted to T_m / T_g for convenience, and to have a higher ratio correspond to a higher inertia and therefore a more stable system.

- The water hammer ratio h_w is a function of both the water column start-up time T_w and the effective governor time T_e. The Allievi water hammer charts can be used to develop this relationship as outlined by Brown (1958), wherein it will be noted that a positive water hammer of 50% will occur when the T_w/T_e ratio reaches 0.41, and a negative water hammer of 50% will be reached with a T_w / T_e ratio of only 0.36.

A chart can now be developed (Fig. 1) in which the water hammer ratio T_w / T_e is plotted as the abscissa and the inertia per unit time ratio T_m / T_g is plotted as the ordinate. Note that both of these ratios are non dimensional.

An examination of [11] will indicate that for the same load change (1) as h_w increases, speed deviation increases; in other words, as the water hammer ratio T_w / T_e increases, so does the speed deviation; (2) as T / T_m increases, speed deviation increases, and for the related inverse T_m / T_g, as this ratio increases, speed deviation will decrease. Accordingly, improved speed regulation can be expected from units that plot on the lower right of the chart.

The characteristics of over 50 hydroelectric developments have been plotted in Fig. 1, with the units divided into four categories:

- Isolated units providing power to mining operations where large electric-powered shovels or large shaft hoists are used.
- Isolated units, most of which provide power to small mining operations or towns in northern Canada.
- System units, all connected to a utility power grid, designed to provide frequency control to the interconnected system.
- Base load system units, all of which have very low inertia – governor time ratios, are energy producers, and are not designed to provide any frequency control to the power system. Based on the distribution of these units, three lines can be drawn in Fig. 1, to separate the chart into four distinct areas:

Area A – Units in this area will *not* be able to provide any frequency control, even on large systems. The units would have to be equipped with relief valves operating in the water-wasting mode and fast governor times to assist in frequency regulation.

Area B – Units in this area can be expected to assist with frequency regulation on large systems only.

Area C – Units in this area can be expected to provide good frequency regulation on isolated systems with small load changes, deteriorating to barely acceptable speed regulation as load changes increase.

Area D – Units in this area can be expected to provide good to acceptable frequency regulation on isolated systems with large load changes.

The three lines that separate these areas are based on using [11] to determine a theoretical speed drop for a large load-on. The lines between areas A–B, B–C, and C–D correspond to theoretical frequency drops of 40%, 25%, and 20% respectively, using the procedure developed by Gordon and Smith (1961), assuming an instantaneous 50% load increase. The relationship between T_m, T_g, T_p, and T_e can now be defined in one equation as follows:

[12] $T_m = kT_g(1 + T_w T_e^{-1})$

with k being an inertia factor that depends on the size of the system and the nature of the load, and has the following values:

$k < 0.55$	(Area A) No frequency regulation possible
$0.55 < k < 0.82$	(Area B) Frequency regulation on large systems only
$0.82 < k < 1.10$	(Area C) Frequency regulation on small systems with small load changes
$1.10 < k$	(Area D) Frequency regulation on small systems with large load changes

Equations [7] and [12] can now be combined to produce an equation for generator inertia as follows:

[13] $GD^2 = 3.65 \times 10^5 k\,(MW)\,T_g\,(1 + T_w T_e^{-1}) N_s^{-2}$

If the ratio of generator inertia to normal inertia is defined as J, then a value for J can be obtained by dividing [13] by [1], and assuming that MYA = 1.14 MW, to obtain

[14] $J = k\,(MW)^{-0.25} N_s^{-0.125} T_g\,(1 + T_w T_e^{-1})$

Equation [14] can now be used to determine how much extra inertia will be required in an isolated system to provide reliable frequency control. As an example, assume a 20 MW unit operating at 150 rpm providing power to a large mining operation, with a penstock layout that has a water start time of 1.1 s; the effective governor time will be about 4.0 s and total governor time will be 5.6 s. A value for J can then be calculated as

$$J = k \times 20^{-025} \times 150^{-0125} \times 5.6(1 + 1.1 \times 4.0^{-1})$$

For large load changes on a small system, k must have a minimum value of 1.1, which gives a minimum value for J = 2.0. Hence the unit must have at least 100% extra inertia in the generator, a not unreasonable figure. For the same development on a large hydro system, k = 0.55, and J = 1.0, or only normal inertia would be needed to assist in frequency control.

Typical examples

Six typical power plants are identified in Fig. 1, to illustrate use of the chart, and to indicate the types of development likely to be found in each area.

> *Area A* – Maggotty in Jamaica. A 6.3 MW unit operating at the end of a long penstock. A relief valve provides water hammer control.
>
> *Area B* – (l) Cat Arm in Newfoundland. A 136MW two-unit impulse turbined power development operating under 381 m head on a 2.9 km tunnel with no surge tank, connected to the provincial grid. (2) La Grande No.3 in Quebec. Large 12-unit, 2304 MW power plant, part of the James Bay complex, connected to the large Hydro-Quebec grid.
>
> *Area* C – (1) Kainji units 11and 12 in Nigeria, each of 110 MW. At time of unit 11- 12 installation, the Kainji development was the main source of power to the national grid. (2) Mayo in the Yukon. A small two-unit 4.4 MW power plant supplying an isolated gold mining operation.
>
> *Area D* – Taltson in the Northwest Territories. A 19 MW isolated hydro development providing power to an open pit mining operation at Pine Point, which experiences major load changes.

Conclusions

Figure 1 along with [13] and [14] can be used to determine whether generator inertial will be adequate, based on the requirements of the load and the size of the connected system. If in doubt, a more detailed analysis will be necessary using a computer program to simulate action of the governor and water conduit during a load change. Finally, a word of caution. This analysis has assumed that the length of any transmission line between the generators and the load is not excessive, or where the length in kilometres does not exceed about 12–15 times the power plant capacity in megawatts. If the transmission line is longer, a more detailed analysis will be required.

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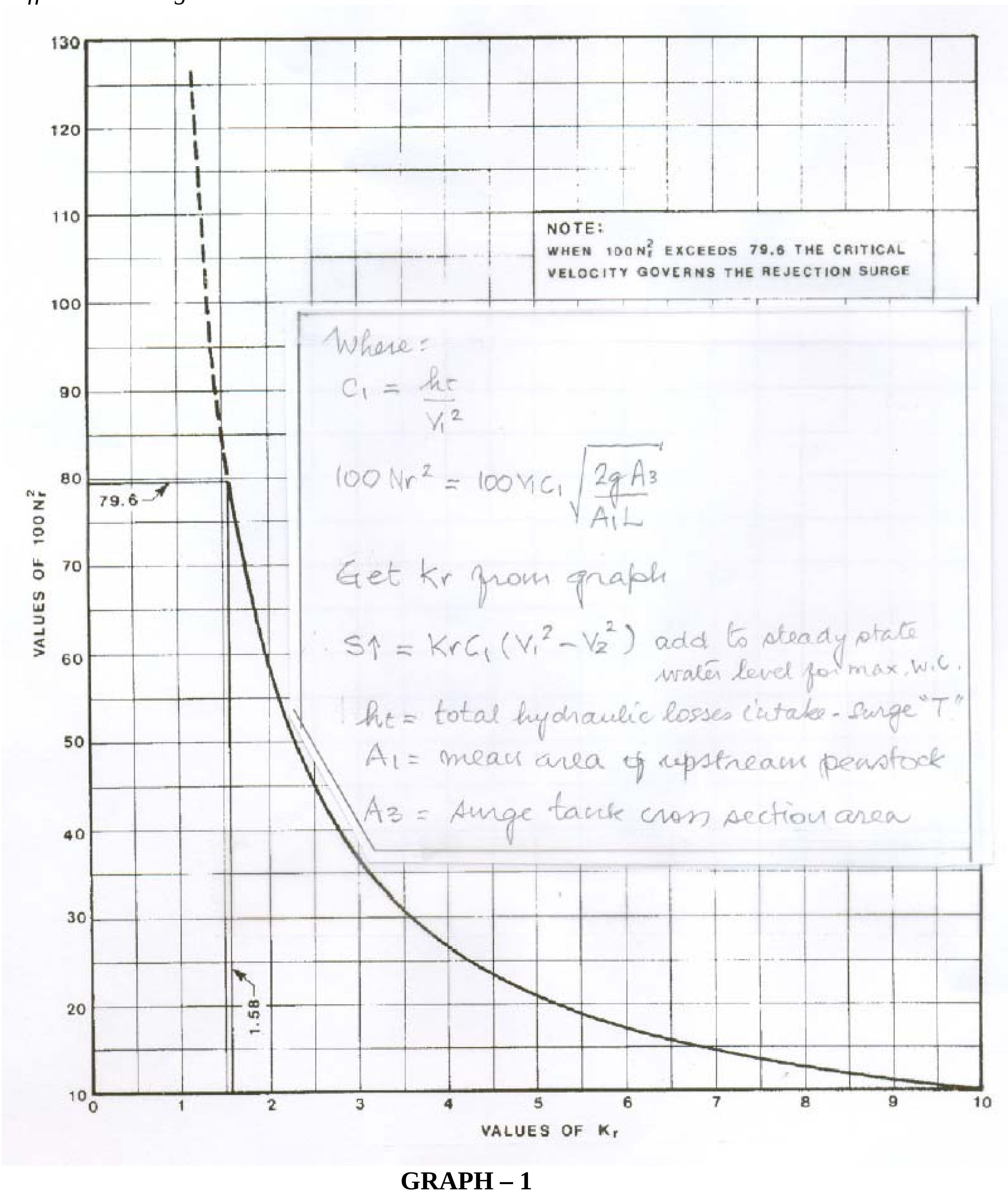
List of symbols

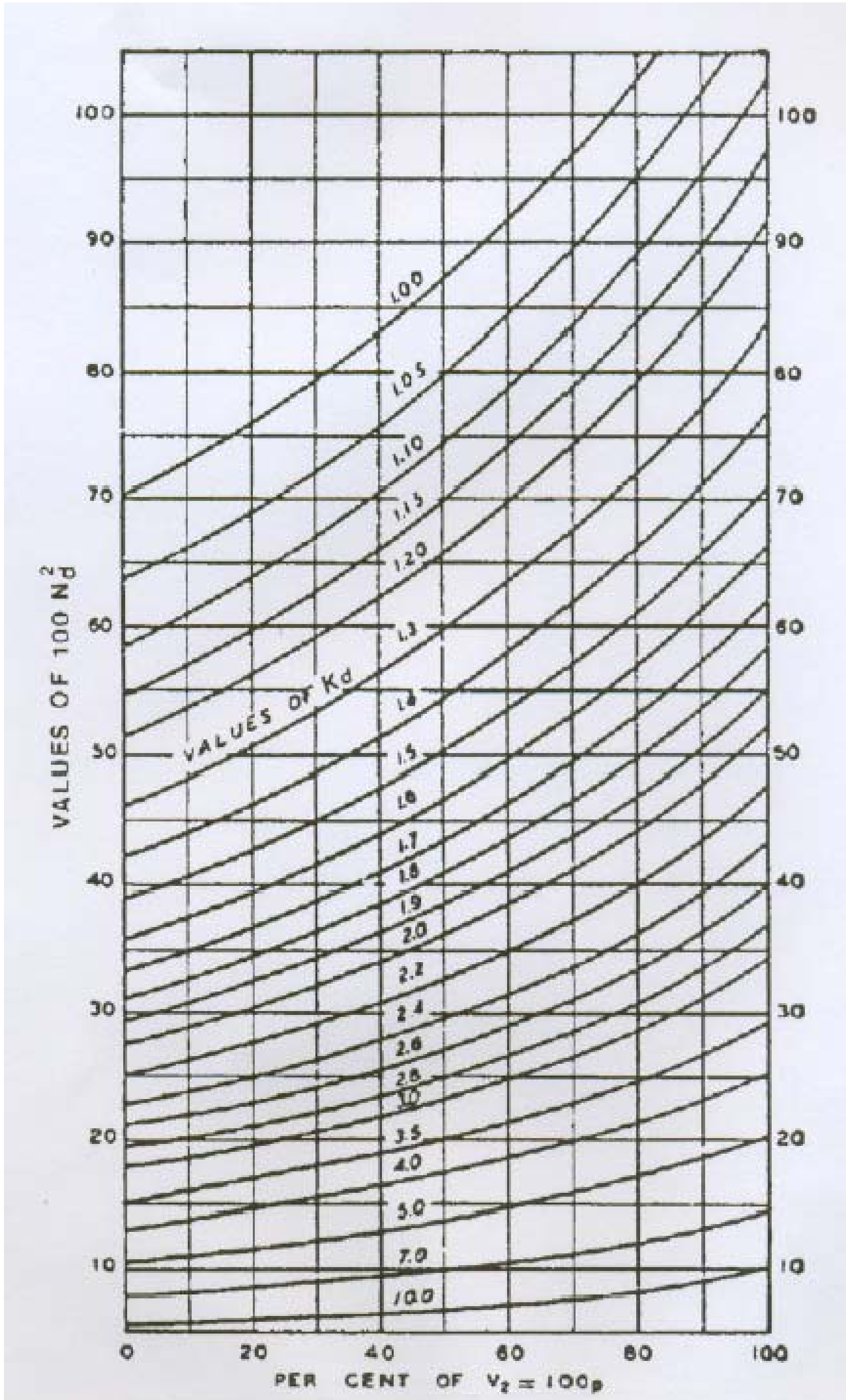
g	Acceleration due to gravity, in metres per second squared
GD^2	Generator inertia, in tonne square metres, based on diameter of rotating mass
h	Turbine rated head, in metres
h_w	Water hammer head, expressed as a fraction of h
H	Generator inertia constant, in kilowatt seconds per kilovolt ampere
HP	Turbine-rated horsepower
J	Generator inertia expressed as a fraction of normal GD^2
k	Inertia factor, depends on system and load
KVA	Generator rating, in kilovolt amperes
KW	Generator capacity, in kilowatts
$\sum LV$	The sum of water passage length times water velocity in that length, in square metres per second
MVA	Generator rating, in megavolt amperes
MW	Generator capacity, in megawatts
N_s	Synchronous speed, in revolutions per minute
N_2	Speed at end of load change, in revolutions per minute
P_1	Initial turbine output, expressed as a fraction of full load output
P_2	Final turbine output, expressed as a fraction of full load output
t	Time duration of pulse load, in seconds
T	Governor time required for a part load change, in seconds

T_e	Effective governor time, in seconds
T_g	Total governor time, in seconds
T_m	Start-up time of water column, in seconds
T_w	Start-up time of column, in seconds
WR^2	Generator inertia, in pound square feet, based on radius of rotating mass

Appendix 4: *Johnson's Charts for Estimating Maximum Up and Down Surges for Differentials Surge Tanks.*



GRAPH – 1
DIFFERENTIAL SURGE TANK
JOHNSON'S CHART FOR LOAD REJECTION CONDITION



GRAPH – 2
For Estimating Maximum Down-Surge

Calculate:

$$C_2 = \frac{ht}{V_2^2} \qquad \text{and} \qquad 100\,Nd^2 = 100V_2 .C_2 \sqrt{\frac{2g\,A_3}{A_1 L}}$$

ΔV_2 as a percent of V_2 gives the value of K_d from Graph 2; whence
$S\downarrow = KdC_2\,(V_2^2 - V_3^2)$. Subtract S ↓ from steady state W.L. to get min. W.L. in tank.

Where:

h_t = total hydraulic losses from intake to surge tank "T"
V_2 = average initial flow in u/s pipeline
V3 = average final flow in u/s pipeline

APPENDIX 5: Table 24 (USBR Monograph 20) for estimation of generator runaway speed.

BASIS FOR DETERMINING WHEN SURGE TANKS WILL BE REQUIRED ON TURBINE PENSTOCK INSTALLATION FOR SPEED REGULATION

1. Surge tanks shall be provided where the resulting reduction in waterhammer pressures will provide a more ecomical turbine penstock installation.

2. A surge tank may also be provided for the condition where the computed percent of speed rise, based on the rejection of the entire rated load for a unit operating independently, cannot be reduced to about 45 percent by other practical methods.

3. For a unit which is one of several units on a common penstock header system, the permissible percent of speed rise will be computed on the basis of one unit operating alone.

4. If the turbine is equipped with a watersaving type of pressure regulator, the critical regulation occurs during load additions when the pressure regulator does not act. Therefore, the permissible percent of speed rise shall be computed on the basis that the pressure regulator is inoperative.

5. For preliminary studies, speed rise may be computed by the following illustrated method. Final design may require a more accurate step-by-step process.

METHOD FOR COMPUTING SPEED RISE

Notation:

T_f = Servomotor minimum closing time, sec.
P_r = Turbine full-gate capacity at h_r, hp
h_r = Rated head, ft
n = Rotational speed; design, r/min
n_s = Design specific speed
WR^2 = Flywheel effect of revolving parts, lb-ft²
L = Equivalent length of water conduit, ft
A = Equivalent area of water conduit, ft²
g = Gravitational constant (acceleration), ft/s²
$Q_r = \frac{550 \cdot P_r}{62.4 \cdot h_r \cdot 0.875}$ = Turbine full-gate discharge, ft³/s
$V_r = \frac{Q_r}{A}$ = Conduit water velocity for full-gate at h_r, ft/s

To obtain the speed rise for full-load rejection, determine the following values:

(a) $T_k = 0.25 + T_f$, full closing time of servomotor(s)
(b) $T_m = \frac{WR^2 n^2}{1\,600\,000 P_r}$, mechanical startup time.
(c) $\frac{T_k}{T_m}$
(d) $n_{sr} = \frac{n(P_r)^{1/2}}{(h_r)^{5/4}}$ At rated condition, hp units
(e) Determine S_R from chart A using n_s and $\frac{T_k}{T_m}$
where:
S_R is speed rise in percent of rotational speed n, for full-gate load rejection to zero, excluding effects of waterhammer
(f) $T_w = \frac{\Sigma L V_r}{g h_r}$ Water startup time
(g) $K = \frac{T_w}{T_f}$
(h) $S'_R = S_R(1+K)$, speed rise in percent of rotational speed n, for full-gate load rejection to zero, including effects of waterhammer

Example:

Given: T_f = 5 sec., P_r = 40 000 hp, h_r = 80 ft, n_{sr} = 94.7, WR^2 = 53 240 000 lb-ft², V_r = 13.779 ft/s, L = 340 ft

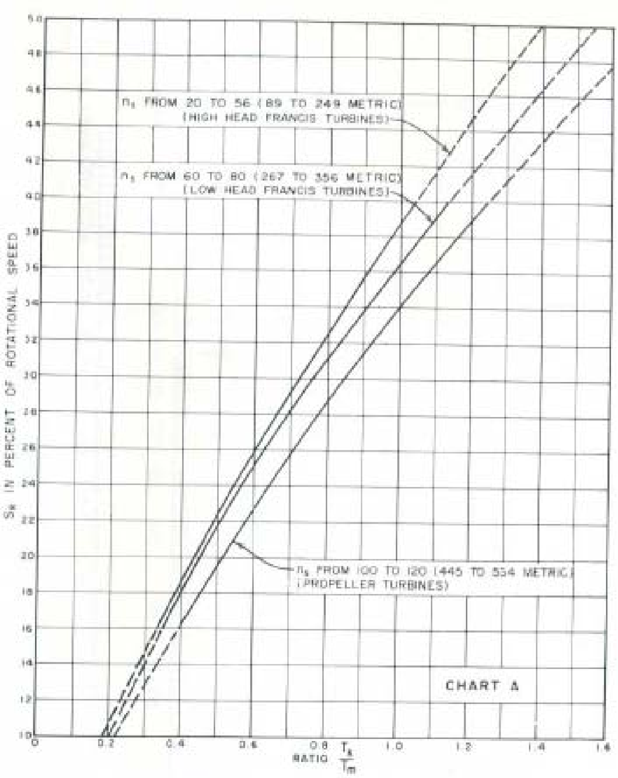
(a) $T_k = 0.25 + 5 = 5.25$
(b) $T_m = \frac{(53\,240\,000)(94.7)^2}{(1\,600\,000)(40\,000)} = 7.46$
(c) $\frac{T_k}{T_m} = \frac{5.25}{7.46} = 0.704$
(d) $n_{sr} = \frac{(94.7)(40\,000)^{1/2}}{(80)^{5/4}} = 79.2$
(e) S_R = 28.1 percent from chart A.
(f) $T_w = \frac{(340)(13.779)}{(32.174)(80)} = 1.820$
(g) $K = \frac{T_w}{T_f} = \frac{1.820}{5} = 0.364$
(h) $S'_R = (28.1)(1+0.364) = 38.9$ percent

References:

Memorandum by John Parmakian dated March 25, 1949, titled "A Proposed Basis for Determining When Surge Tanks Will be Required for Speed Regulation at Turbine Penstock Installations."
A.S.M.E. paper No. 48-A-37 "Speed Regulation Computations for Hydraulic Turbines," Robert Lowy, 1949.

Note:

The above example is for one set of variables. Calculations should also be made at other conditions to establish speed rise magnitude; i.e., for heads of h_r, H_{min}, and H_{max}.



HYDRAULIC TURBINE SPEED REGULATION UTILIZATION OF SURGE TANKS

9-17-75
OCT. 1949 – JOHN PARMAKIAN

106-D-68

FIGURE 24.—*Hydraulic turbine speed regulation—utilization of surge tank.*

6 HYDRAULIC DESIGN OF PENSTOCKS

6.1 BACKGROUND

The design of penstocks must take into account the related issues of waterhammer and speed control. Experience shows that pipeline / turbine systems meeting the following conditions do not require additional protective devices:

- Where L/H < 5 to 8, or
- Where $\frac{\Sigma L_i V_i}{H_n} < 7 \ to \ 13$
- Where maximum speed rise of the generator on full load reject < 45%.
- When pressure rise at the scroll case on full load rejection < 50%.

(*Taken from Sub-section 2.2.5/6.0 of this Standard)*.

While the focus of this sub-section is on the design of simple penstocks, a method for determining the design pressures in a pressure conduit incorporating a surge tank will also be given.

A step by step methodology is proposed, as below:

- Preliminary design and optimization.
- Determination of waterhammer pressure extremes.
- Assessment of the adequacy of machine rotating inertia.
- Confirmation of acceptable effective wicket gate closing time, additional machine inertia and if other protective devices are required.

6.2 PRELIMINARY DESIGN AND OPTIMIZATION:

For mini-hydro and small hydro plants the optimum penstock diameter can be determined using Sarkaria's (1958) formula:

$$D = 3.55\left(\frac{Q^2}{2gH}\right)^{\frac{1}{4}}$$

Where: Q = flow (m^3/s)
H = rated net head on turbine (m)

The reliability of these results will be enhanced if Sarkaria's formula is first calibrated against recent and comparable designs.

For larger SHP it is recommended that a more detailed optimization analysis be undertaken. In this analysis the optimum diameter should be determined as the diameter for which the capital cost of the penstock plus capitalized value of hydraulic losses would be a minimum. These economic calculations should be in accordance with Sub-section 1.7 of this standard.

Where water carries a significant sediment load steel pipes are preferred. In such cases Mosonyi also recommends that water velocities should not exceed 3-5 m/s. For preliminary designs the following normal water hammer pressure rises may be assumed, as a ratio of max head (h) divided by static head (H_0):

a) For high head plants with impulse turbines (Pelton)

$$\frac{h}{H_0} = 0.10 \ \ to \ \ 0.25 \qquad (use \ \ 0.15)$$

b) For medium head plants with reaction turbines (Francis)

$$\frac{h}{H_0} = 0.2 \ to \ 0.5 \qquad (use \ 0.3)$$

c) For low head plants with reaction turbines (Kaplan, Fixed Propeller)

$$\frac{h}{H_0} = 0.3 \ \ to \ \ 0.5 \qquad (use \ 0.4)$$

Note that Z^2 used in Allievi's method is the same as $(H_0 + h)/H_0$.

d) Finally a preliminary pressure design must be performed to determine penstock shell
thickness (see Section 2.2.3/9).

The following table gives Manning's values and other material properties for various pipe materials.

Table .6.1: Materials used in pressure pipes

Material	Young's modulus of elasticity (E) (N/m^2) $.10^9$	Coefficient of linear expansion (a) (m/m °C) $.10^6$	Ultimate tensile strength (N/m^2) $.10^6$	Poisson's Ratio μ	Manning's n
Welded steel	206	12	400	0.3	0.012
Polyethylene	0.55	140	5	n.a.	0.009
Polyvinyl Chloride (PVC)	2.75	54	13	n.a.	0.009
Asbestos cement	n.a.	8.1	n.a.	n.a.	0.011
Cast iron	78.5	10	140	0.3	0.014
Ductile iron	16.7	11	340	0.3	0.015

At this stage of design it is opportune to consider the use of pipes of various materials, taking into consideration, cost, constructability and service aspects. Today there is a wide choice of materials for penstocks. For the larger heads and diameters, fabricated welded steel is probably is best option. Nevertheless spiral machine-welded steel pipes should be considered, due to their lower price, if they are available in the required sizes. For high heads, steel or ductile iron pipes are preferred, but at medium and low heads steel becomes less competitive, because the internal and external corrosion protection layers do not decrease with the wall thickness and because there is a minimum wall thickness for pipe handling.

For smaller diameters, there is a choice between manufactured steel pipe, supplied with spigot and socket joints and rubber "O" gaskets, which eliminates field welding or with welded-on flanges, bolted on site plain spun or pre-stressed concrete, ductile iron spigot and socket pipes with gaskets; cement-asbestos; glass-reinforced plastic (GRP); PVC or polyethylene (PE) plastic pipes. Plastic pipe is a very attractive solution for medium for medium heads – a PVC pipe of 0.4 m diameter can be used up to a maximum head of 200 meters – because it is often cheaper, lighter and more easily handled than steel and does not need protection against corrosion. PVC pipes are easy to install because of the spigot and socket joints provided with "O" ring gaskets. PVC pipes are usually installed underground with a minimum cover of one meter. Due to their low resistance to UV radiation they cannot be used on the surface unless painted coated or wrapped. The minimum radius of curvature of a PVC pipe is relatively large – 100 times the pipe diameter – and its coefficient of thermal expansion is five times higher that for steel. They are also rather brittle and unsuited to rocky ground.

Pipes of PE – high molecular weight polyethylene – can be laid on top of the ground and can accommodate bends of 20-40 times the pipe diameter – for sharper bends, special factory fittings are required – PE pipe floats on water and can be dragged by cable in long sections but must be joined in the field by fusion welding, requiring a special machine. PE pipes can withstand pipeline freeze-up without damage, but for the time being, may be not available in sizes over 300 mm diameter.

Appropriate values for Manning's n, friction factor of and loss coefficients for trashrack and other form losses can be found in Section 2.2.1/4 and 5 and Section 2.2.2 /5; while *Appendix 1* gives formulae for calculation of waterhammer wave speeds.

6.3 ESTIMATION OF WATERHAMMER PRESSURE RISES / DROPS

Allievi has developed a graphical method which can be used to determine waterhammer pressure changes. His charts are based on linear gate operation, that is the effective area of valve or wicket gate varies uniformly with time. His graphs give pressure rise or drop ratios (Z^2) as a function of two parameters ρ and θ , as defined below:

- Pipeline parameter : $\rho = \frac{aV_0}{2gH_0}$
- Valve operation parameter: $\theta = \frac{aT}{2L}$

Where:

a = waterhammer wave velocity (m/s)

V_0 = initial water velocity for valve closure, or final steady state velocity for valve opening (m/s)

T = effective opening time of the valve a wicket gates (s)

L = effective length of conduit (m)

For conduits with variable diameters:

Effective area $(A_e) = \dfrac{\Sigma L_i A_i}{\Sigma L_i}$

and $\quad V_0 = \dfrac{Q}{A_e}$

For conduits with varying diameter / thickness ratios:

$$a = \frac{\Sigma L_i a_i}{\Sigma L_i}$$

Formulae for computation of waterhammer wave velocity (a) are given in Appendix 1. Allievi's charts for determination of waterhammer pressure rises are given in Appendix 2 of this section.

Effective opening or closing time for wickets gates is usually taken as 2.(T_{75} – T_{25}), see Figure 2.2.6.1.

6.3.1 Determination of Waterhammer Pressure Rise

Due to Valve / Wicket Gate Closure (Normal Operations): In this analysis forebay maximum operating should be assumed.

Data required:

$$A_e = \frac{\Sigma L_i A_i}{\Sigma L_i} \qquad \rightarrow V_0 = \frac{Q}{A_e} \quad (initial \ \ flow)$$

$$a = \frac{\Sigma L_i a_i}{\Sigma L_i}$$

Solution:

- Assume Z^2, per sub-section 2.2.6 / 2.0.
- $H_o = H_s - \Sigma h_2$
- $\rho = \dfrac{aV_0}{2gH_0}$

Where:

H_0 = initial stead state head
H_s = static head
$\Sigma h_l = head \ \ losses$

Knowing ρ, Z^2 find θ from the appropriate Allievi chart.

Whence $T_e = \dfrac{2L\theta}{a}$

(Allievi's charts are given in Appendix 2 of this Sub-section)

6.3.2 Determination of Pressure Drop due to Valve or Wicket Gate Opening (Normal Operations):

For minimum pressure the low forebay operating level should be assured. This calculation is only required once T_e has been determined. Data required same as in (a) above, *except that V_0 is the final steady state velocity.*

Solution:

$$H_0 = H_s$$

$$\rho = \frac{aV_0}{2gH_0}$$

$$\theta = \frac{aT}{2L}$$

Knowing ρ and θ find Z^2 from the appropriate Allievi chart.

Minimum head at valve (H) = H_0Z^2

Note that governor opening and closing times can be set at different values, if required, to avoid vacuum conditions in the line, as explained in Sub-section 2.2.6/5.

6.4 PARAMETERS FOR FINAL DESIGN:

Water hammer pressure extremes shall be determined for normal and emergency operating conditions.

Normal Conditions: Governor and needle valves / wicket gates operating as designed.

- *For maximum waterhammer pressure rises* full load rejection shall be assumed coincident with maximum forebay water levels.
- *For maximum pressure drops* partial or full load addition under governor control is to be assumed, consistent with electrical system characteristics. Minimum forebay operating W.L. is to be assumed for these computations.

Emergency Conditions: Emergency waterhammer is produced under the following conditions:

- Load rejection
- Governor cushioning stroke inoperative
- Part gate closure in $\frac{2L}{a}$ seconds at maximum rate of gate movement.

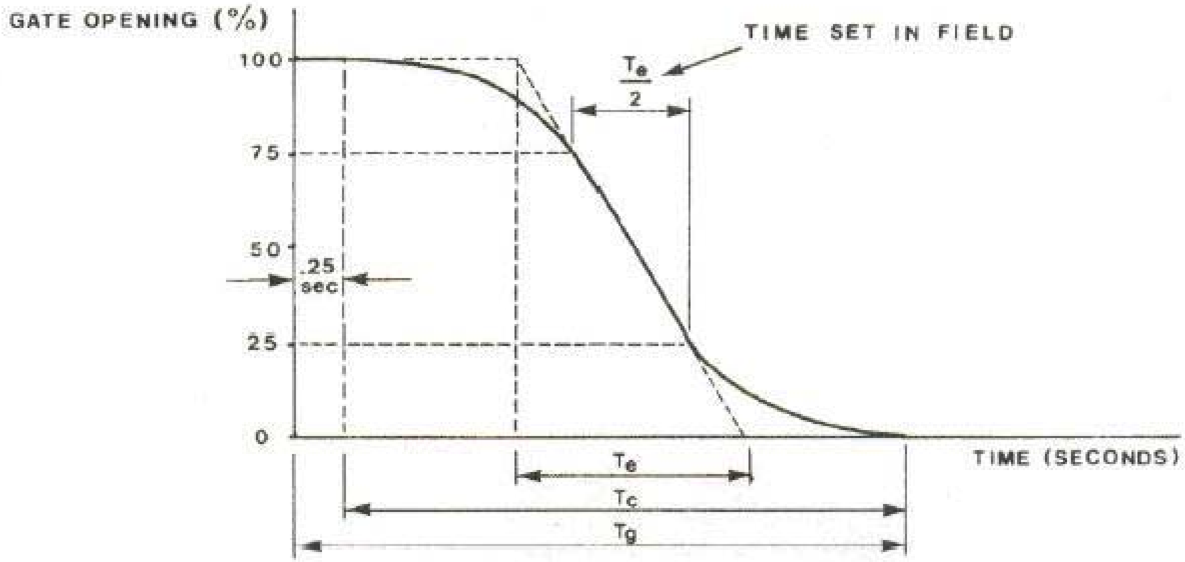
The waterhammer pressure rise (h) can be calculated by Michaud's formula:

$$h = \frac{2LV_{cr}}{gT_e} \ and \ \ V_{cr} \cong V_0$$

Figure 2.2.6.2 (a) shows the results of the analyses for simple penstocks. For pipelines with surge tanks the surge and water hammer pressures are combined as shown in Figure 2.2.6.2 (b).

This approach is approximate but on the conservative side. Diagrams for both normal and emergency water hammer have to be developed. It should be emphasized that waterhammer excess pressure vary linearly with respect to the length of the pipelines. In cases where effective velocities and wave velocities have been used in Allievi's procedure, the result must be adjusted to give the correct, distribution of water hammer pressure.

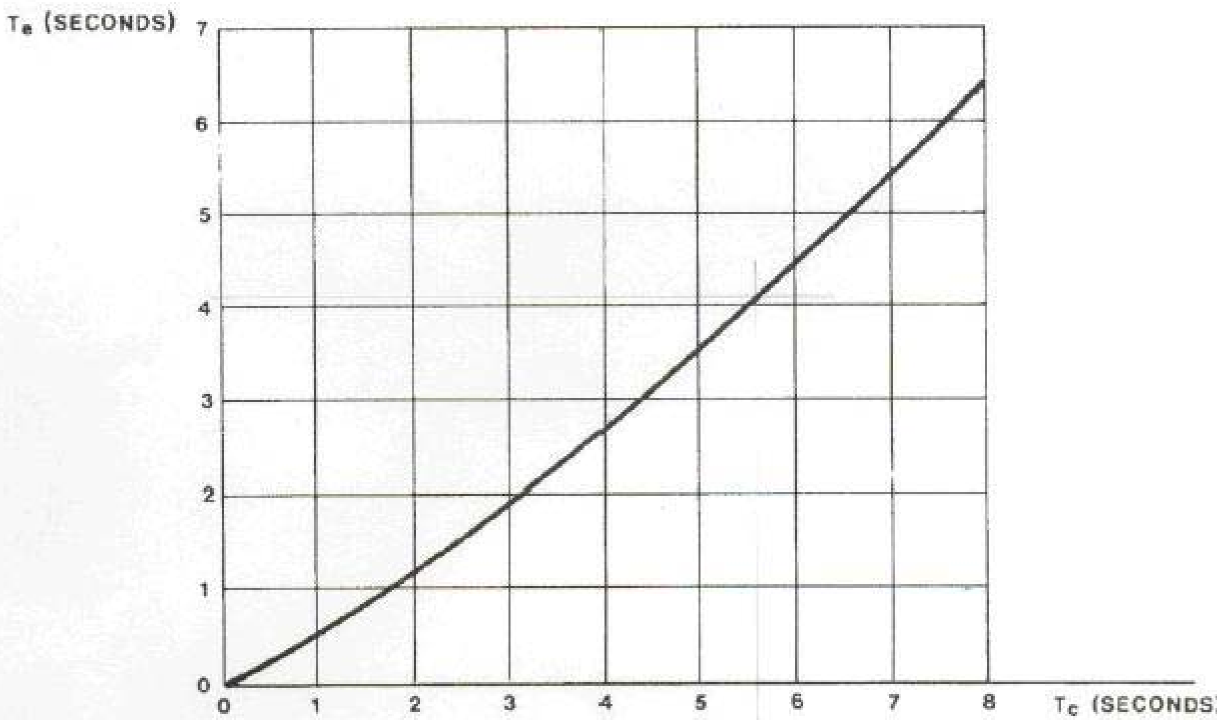
For larger projects (>10 MW) a more detailed waterhammer should be considered, especially where surge tanks or other pressure control devices are incorporated penstock/pipeline. Use of a simulation model such as WHAMO is recommended. Alternatively, traditional graphical or numerical methods can be used as explained in reputed authors such as Parmakian and Chaudhry.



NOTE:

a) For $T_e > 6$ Seconds $T_c = T_e + 1.6$ Seconds

b) For $T_e < 6$ Seconds get T_c from the graph below



6.5 LAYOUT

6. 5.1 Route Selection

The following points should be considered in choosing the layout and routing of a penstock.

- The shortest practical route is preferred.
- Sharp bends should be avoided. Generally the radius of curvature of a bend (R) = 3 to 5 times (d) the diameter of the penstock.

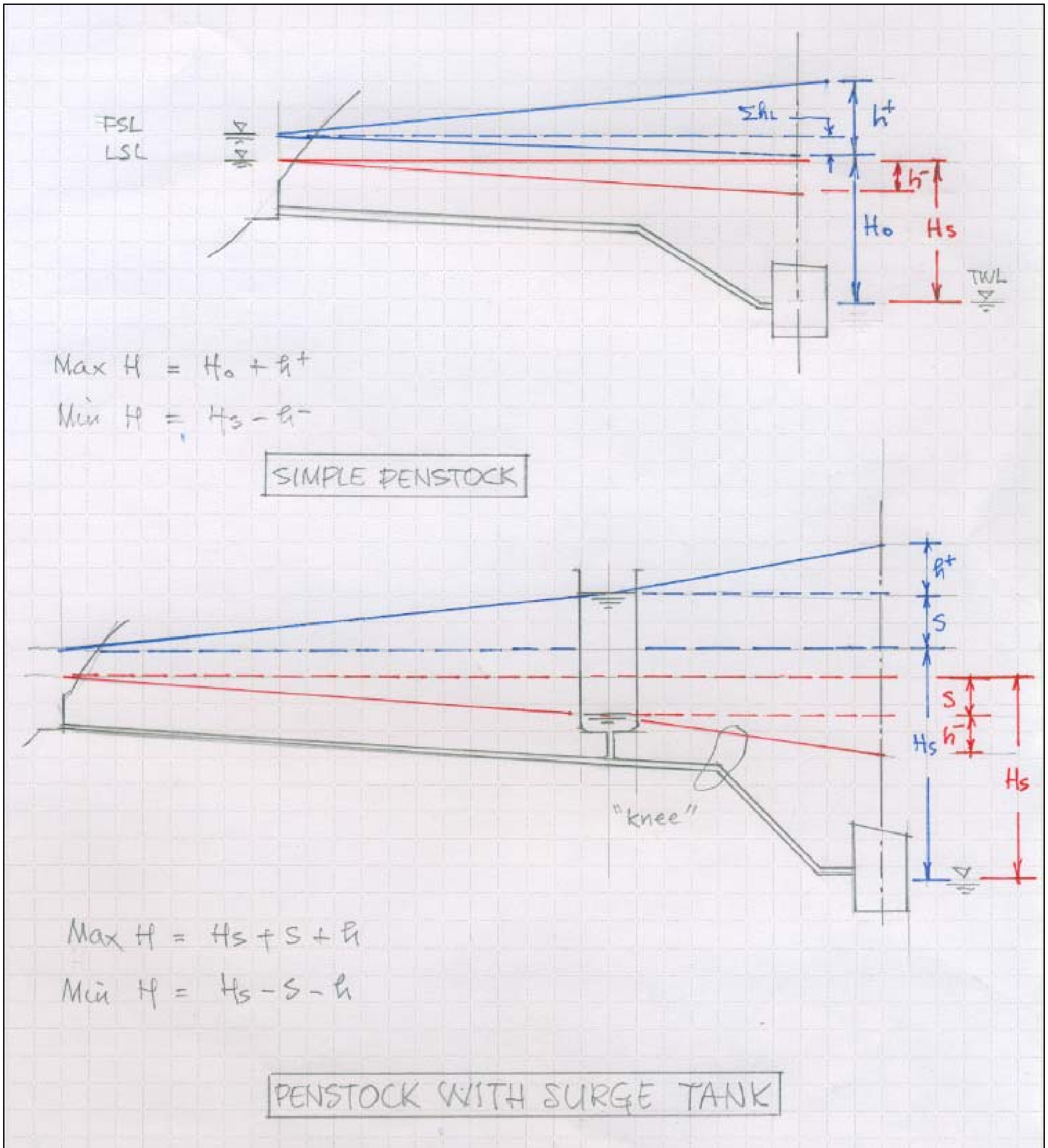


FIGURE 2.2.6.2

- A route following a ridge-line is preferred to avoid drainage problems. Where this is not possible care must be taken design an effective drainage system to divert surface runoff away from the penstock and powerhouse.
- Choose an alignment that will ensure that penstock is always under positive pressure. Vulnerable points are typically as "knees in the vertical alignments. The minimum pressure gradient line should be at least one (1) penstock diameter above the elevation of the bend with reference to the top of the bend.
- A horizontal section of 5 times the diameter of the penstock should be provided upstream of the scroll case entry – to ensure uniform distribution of flow velocity to the turbine.

6.5.2 Number of Penstocks

Depending of the size of the plants and the number of units proposed. The decision number of penstocks is based on consideration of economics and practicality.
The following points should be addressed:

Number of Penstocks

- Decided on the basis economic analysis of the merits and demerits and of different feasible alternatives.
- Long penstock (high head penstock), single in the upper stretch and branching in the lower stretch.
- Short penstock – one to each unit.
- A single big size penstock with manifold distributor to each unit - less costly when compared to multiple penstocks but hydraulic losses at the manifold may be significant
- Civil works and number of accessories increase as numbers of penstocks increase.

6.5.3 Buried versus surface design:

The pros and cons of each type of penstock design are summarized in the following notes:

Surface Penstock:
Advantages:

- Easily accessible for inspection.
- Installation often less expensive
- Easily accessible for maintenance and repairs
- Safety against sliding may be ensured by properly designed anchorages.

Disadvantages:

- Prone to rusting and corrosion being exposed.

- Repeated painting of outer surface is needed.
- Supporting and anchoring on steep hill slope is difficult and costly.
- Susceptible to damage by landslides and rockfalls.
- Expansion joints necessary.
- Chances of water conveyed being frozen in severe cold climates.

Buried Penstocks:

Advantages:

- Protection against effect of temperature.
- Protection against freezing of water.
- Less visual impact.
- Protection against animals, earthquake shocks.
- No expansion joints are needed
- Continuous support helps in reducing steel plates thickness

Disadvantages:

- Less accessible for inspection – difficult to locate leaks.
- Installation costly.
- Tendency of sliding of pipes on steep slopes.
- Need special coating against the corrosive action of ground water.
- Maintenance and repairs difficult.

6.6 UNUSUAL CIRCUMSTANCES

6.6.1 Penstock / Surge Tank Layouts with Long power Tunnels.

The waterhammer wave initiated by action of the wicket gates is not perfectly reflected at the surge tank but a components is transmitted into the power tunnel upstream of the surge tank "tee" If the power tunnel is relatively long this will result in a waterhammer over pressure extending some distance upstream to a critical point C as shown in Figure 2.2.6.3.

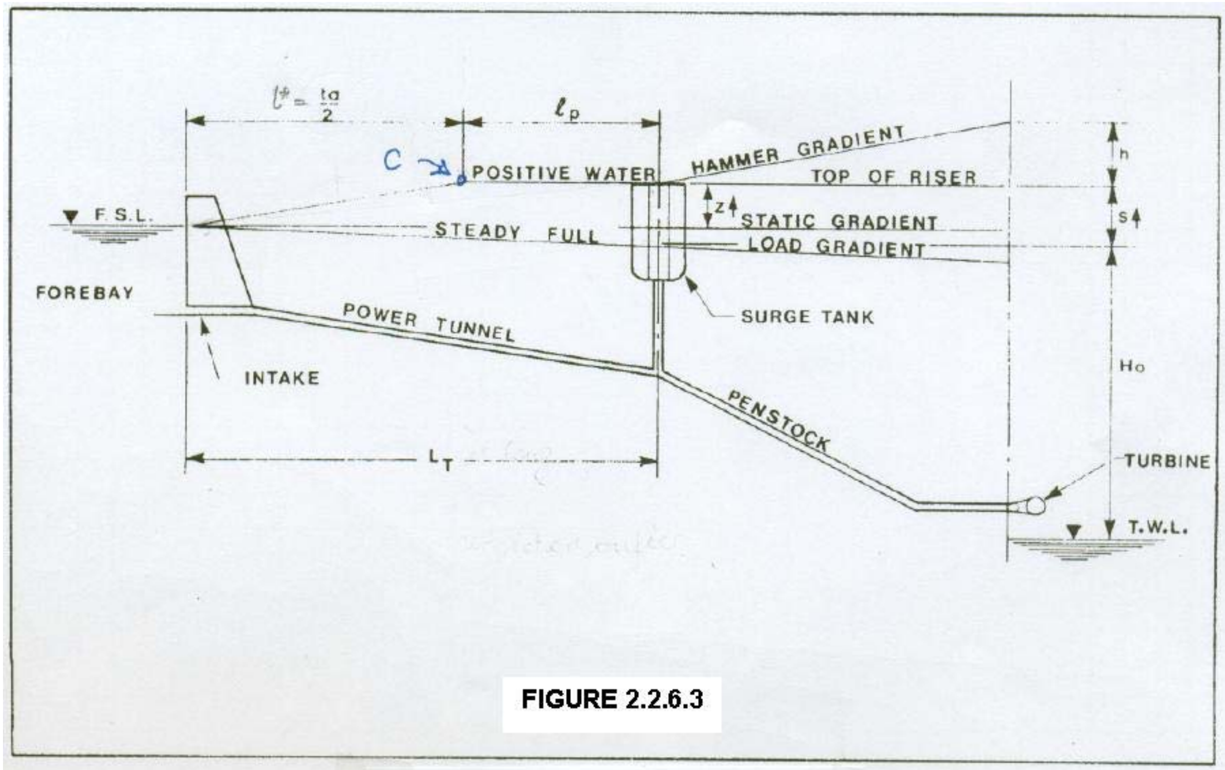


FIGURE 2.2.6.3

The extents of this zone can be estimated by estimated as $l* = \frac{ta}{2}$,

For restricted orifice surge tanks:
t ≈ T_c (Where T_c is wicket gate closure time)

For differential surge tanks:

$$t \approx \frac{Y_r R}{V_z A} + \frac{t'}{2}$$

Where:

Yr	= max. rise in surge tank W.L. above static
R	= area of internal uses (assume 0.9A)
A	= area of power tunnel
V_2	= initial velocity in power tunnel
t'	= T_c)

This phenomenon does not affect simple surge tanks. If a preliminary assessment indicates the presence of persisting water hammer in the power tunnel, a detailed analysis of conduits, surge tank and turbine is recommended using WHAMO or an equivalent computer program.

6.6.2 Choking

For low specific speed Francis turbines (N_S<270) located on long penstocks there is a risk that maximum waterhammer could be caused by choking of flow by the turbine runner under runaway conditions. This problem is a characteristic of low specific speed Francis turbines where turbine flow decreases as runaway speed increases (unlike higher specific speed Francis runners where turbine flow increases with runaway speed). For low inertia machines full runaway can be reached in a few seconds (similar or less than the normal wicket gate closure time). In such cases the maximum waterhammer produced by "choking" could be greater than from normal wicket gate operation. This phenomenon has been investigated by Ramos who produced the following chart from which the effects of turbine overspeed on waterhammer can be estimated.

The variables shown in this chart are defined below:

- Q_{Rw} = flow at full runaway (m^3/s)
- Qo = turbine rated discharge (m^3/s)
- T_W = water starting time (s)
- T_m = mechanical staring time (s)
- T_C = effective wicket gate closure time (s)
- T_E = time for one round trip for first elastic wave (s)
- H_O = gross head (m)
- ΔH_M = waterhammer pressure increase (m)

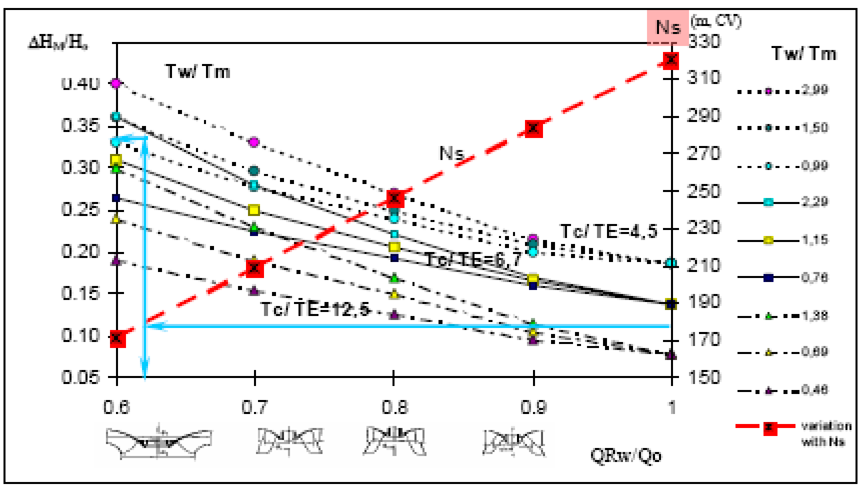


Fig. 7.6 – Maximum overpressure induced by both the overspeed and gate effects of low specific speed reaction turbines on upstream penstock (RAMOS, 1995).

As indicated in the figure (follow the blue arrows), the calculation begins with the turbine N_S value. Moving horizontally the N_S dashed line is reached and Q_{Rw}/Q_o can be found. Knowing the T_W/T_M value (the relative water and turbine inertia time constants) and selecting the relative wicket gate closure time T_C/T_E the relative maximum waterhammer pressure ($\Delta H_M/H_O$) can be determined. This graph gives an approximate prediction of the maximum waterhammer pressure due sudden load rejection in Francis units of a small power plant.

If the above calculation indicates the likelihood of a problem due to the effects of turbine overspeed then the turbine manufacturer should be consulted and a detailed analysis of the power tunnel / surge tank / penstock / turbine system using WHAMO (or an equivalent computer program) should be undertaken in collaboration with the turbine manufacturer. The objective of theses calculations would be to verify the severity of the problem and to evaluate corrective measures. The turbine and generator manufacturers should also be required to guarantee unit inertia (WD^2) and T_C (effective wicket gate closure time) at the design stage.

6.7 References

Water Power Development
Volume 2A: High Head Power Plants
E. Mosonyi
Akadémiai Kiadó
Budapest, Hungary (1991)

Waterhammer Analysis
J. Parmakian
Dover Publications, New York (1963)

Applied Hydraulic Transients
H.M.Chaudhry
Van Nostrand Rienhold Co., new York (1978).

Guidelines for Design of Small Hydropower Plants
H. Ramos
Western regional Energy Agency & Dept. of Development,
Belfast, North Ireland (2000)

Water Hammer and Mass Oscillation (WHAMO) – Computer Program
USACERL ADP Report 98/129
Construction Engineering Research Laboratories,
U.S. Army Corps of Engineers

6.8 APPENDICES:

6.8.1 Appendix 1: *Formulae for Determination of Waterhammer Wave Celerity*

General Equation for wave celerity (a)

$$a = \sqrt{\frac{k/\rho}{1+\left[\frac{k}{E}\cdot\frac{D}{e}\right].c_1}}$$

Where:

a = wave celerity (m/s)
K = modulus of deformation of water (GPa)
ρ = mass density of water (kg/m^3)
E = young's modulus pipe shell (GPa)
D, e = pipe diameter & thickness (mm)
C_1= factor for pipe restraint
μ = Poisson's ratio

Restraint conditions for thin walled elastic pipes $\left(\frac{D}{e} \geq 100\right)$

- Pipe anchored at upstream end only $C_1 = 1 - \frac{\mu}{2}$
- Pipe anchored against longitudinal movements $C_1 = 1 - \mu^2$
- Pipe with expansion joints throughout its length $C_1 = 1$

Restraint conditions for thick walled elastic pipes $\left(\frac{D}{e} < 100\right)$

- Pipe anchored at upper end only
  $C_1 = \frac{2e}{D}(1+\mu) + \frac{D}{D+e}.\left(1 - \frac{\mu}{2}\right)$
- Pipe anchored against longitudinal movement
  $C_1 = \frac{2e}{D}.(1+\mu) + \frac{D(1-\mu^2)}{D+e}$
- Pipe with expansion joints throughout its length
  $C_1 = \frac{2e}{D}(1+\mu) + \frac{D}{D+e}$

Circular tunnel:

$$a = \sqrt{\frac{K/\rho}{1 + (2K/E_R)(1+\mu)}}$$

Where E_R = Modulus of rigidity of rock
μ = Poisson's ratio

Selected Material properties are listed in Table 2.2.6/1

Source: *Fluid Transients*
By Streeter and Wylie.

6.8.2 Appendix 2: *Allievi's Charts*

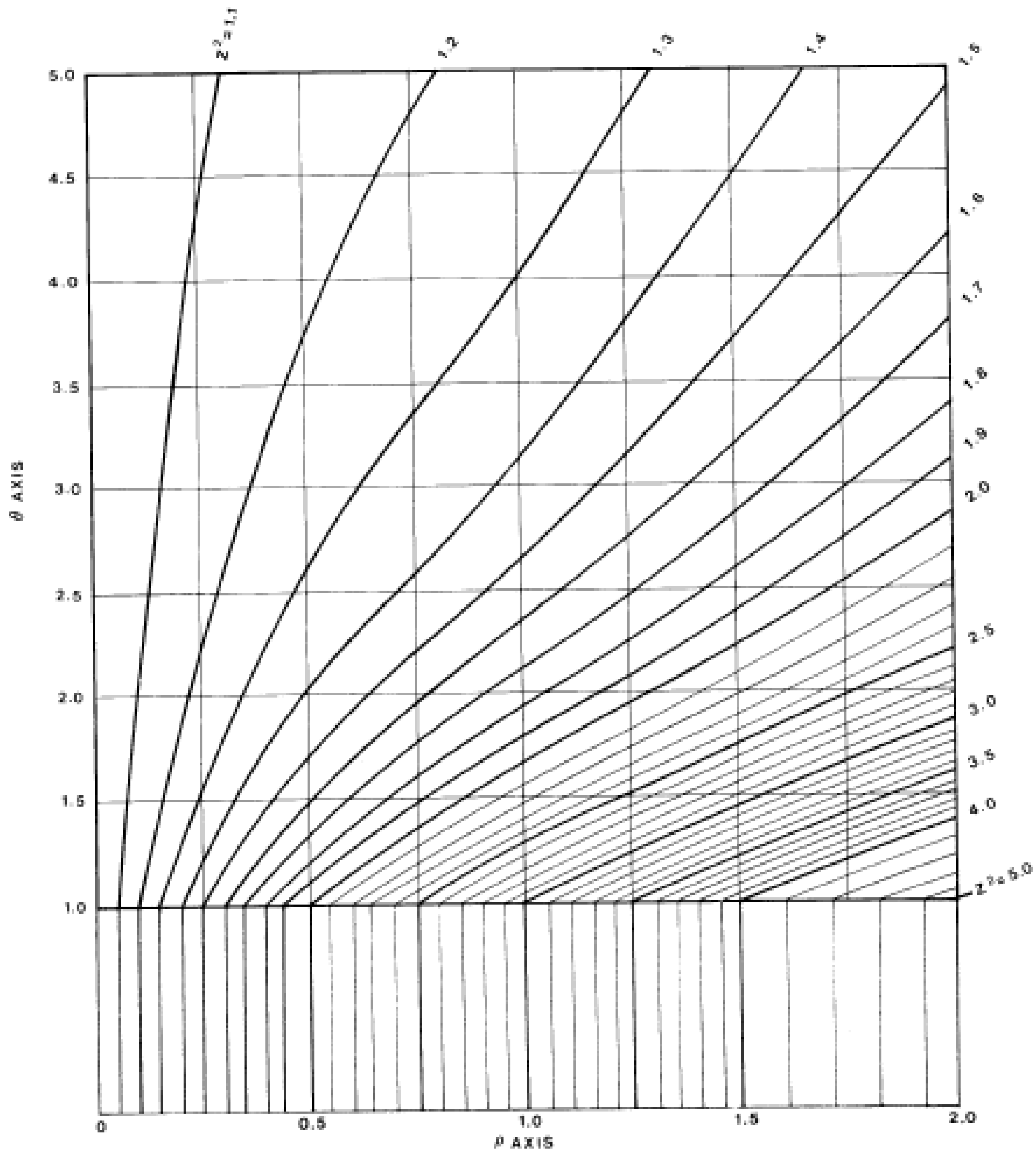


FIGURE 8

ALLIEVI CHART - WATER HAMMER
MAXIMUM RISE IN PRESSURE : Z^2 FOR UNIFORM GATE MOTION
AND SIMPLE CONDUITS (FOR SMALL VALUES OF ρ AND θ)

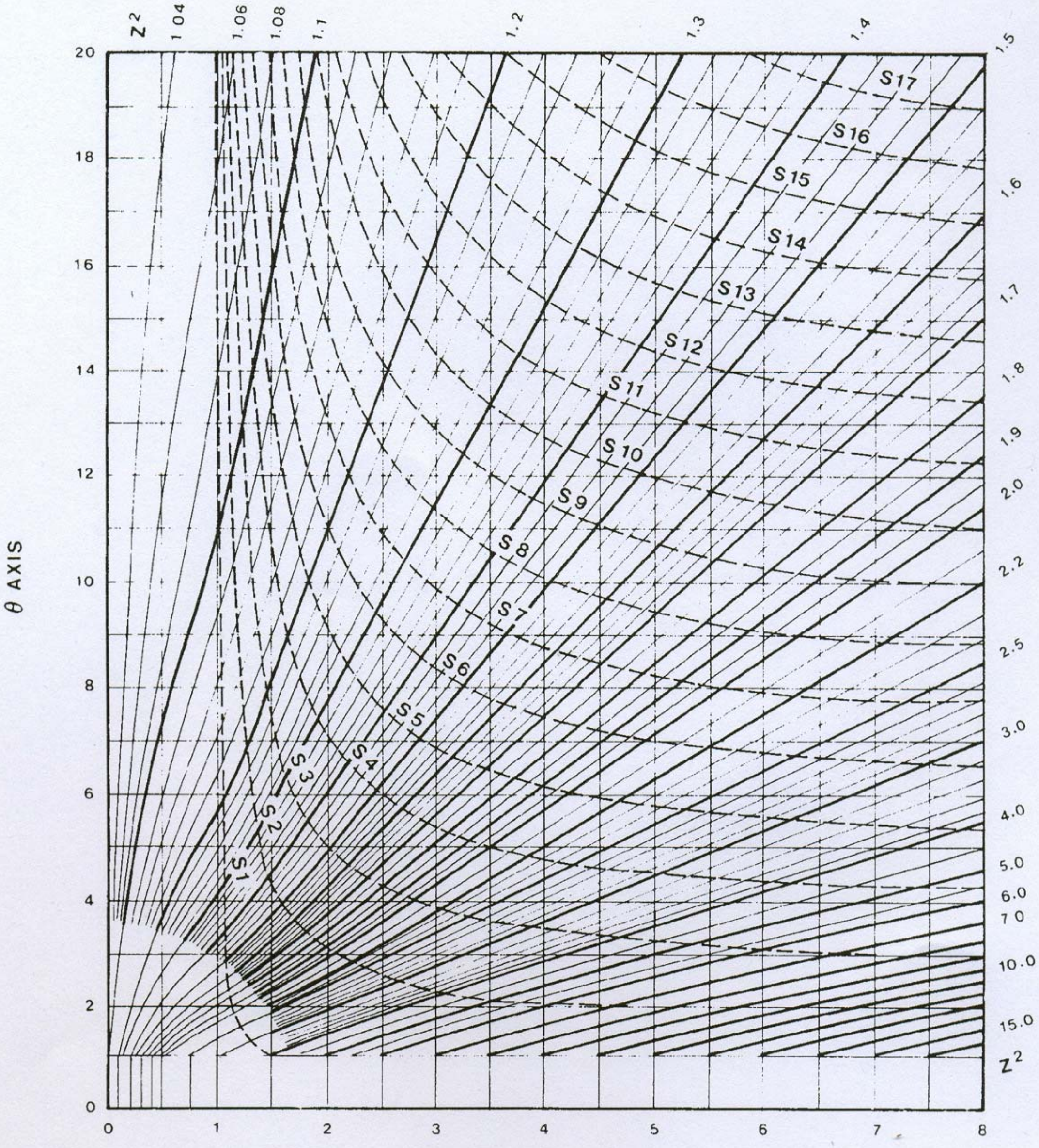


FIGURE 9

ALLIEVI CHART – WATER HAMMER
MAXIMUM RISE IN PRESSURE: Z^2 FOR UNIFORM GATE MOTION AND SIMPLE CONDUITS (FOR INTERMEDIATE VALUES OF ρ AND θ)

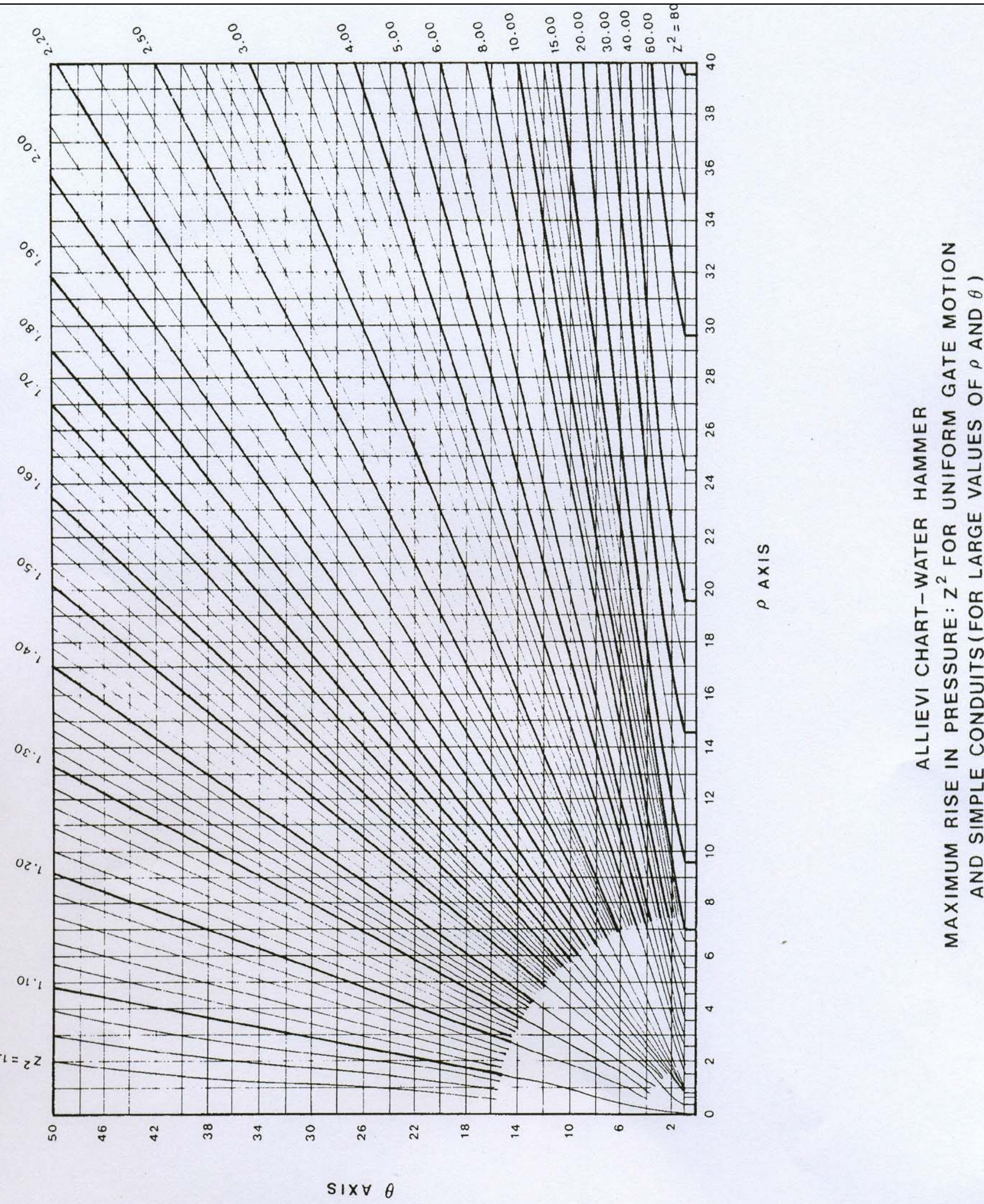

ALLIEVI CHART–WATER HAMMER
MAXIMUM RISE IN PRESSURE: Z^2 FOR UNIFORM GATE MOTION
AND SIMPLE CONDUITS (FOR LARGE VALUES OF ρ AND θ)
ρ AXIS
θ AXIS
$Z^2 = 1.04$
1.10
1.20
1.30
1.40
1.50
1.60
1.70
1.80
1.90
2.00
2.20
2.50
3.00
4.00
5.00
6.00
8.00
10.00
15.00
20.00
30.00
40.00
60.00
$Z^2 = 80$
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38 40
2 6 10 14 18 22 26 30 34 38 42 46 50

FIGURE 10

- 33 -

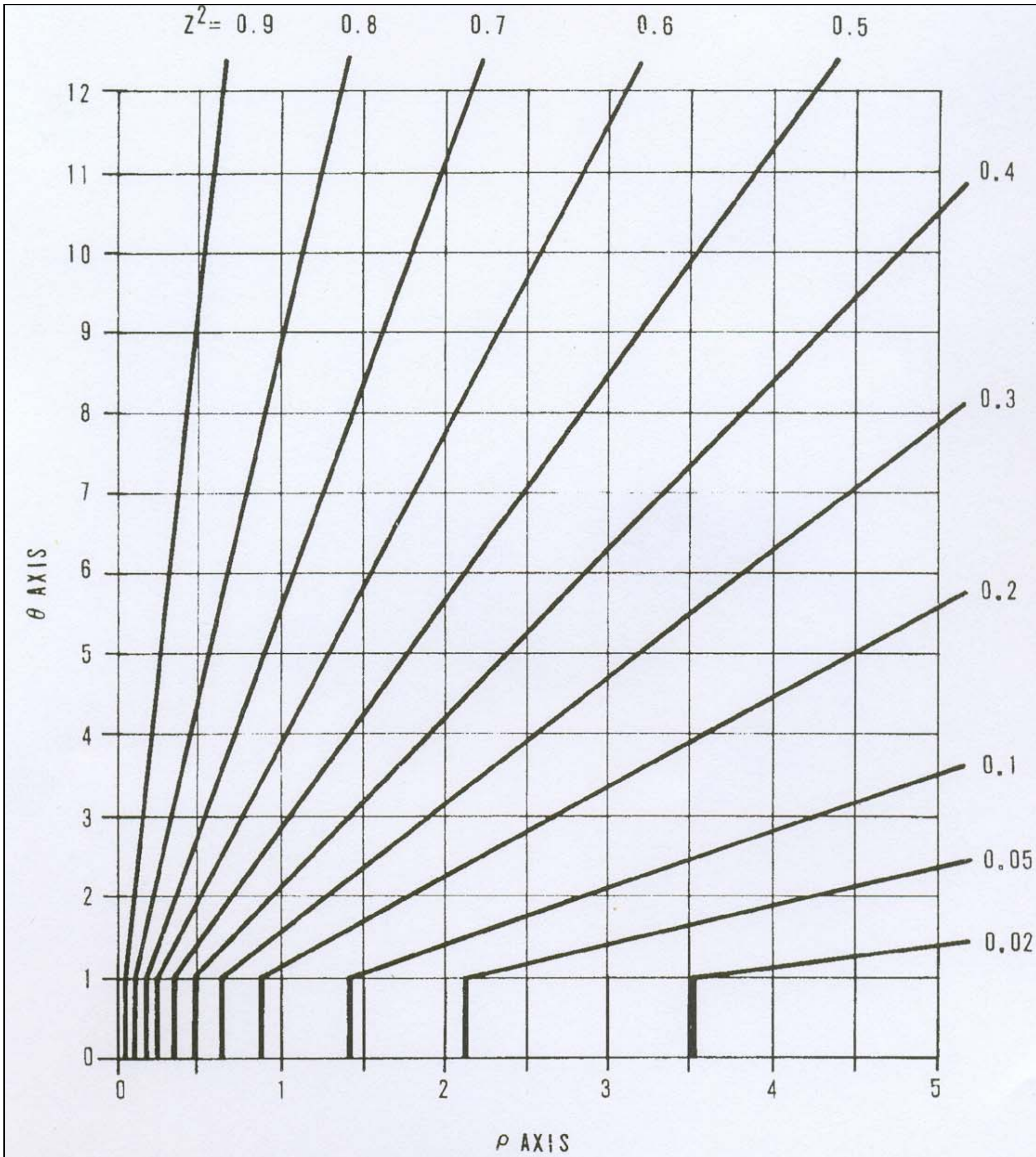


FIGURE 11

ALLIEVI CHART - WATER HAMMER
MAXIMUM DROP IN PRESSURE FOR UNIFORM GATE MOTION AND SIMPLE CONDUITS (FOR SMALL VALUES OF ρ AND θ)

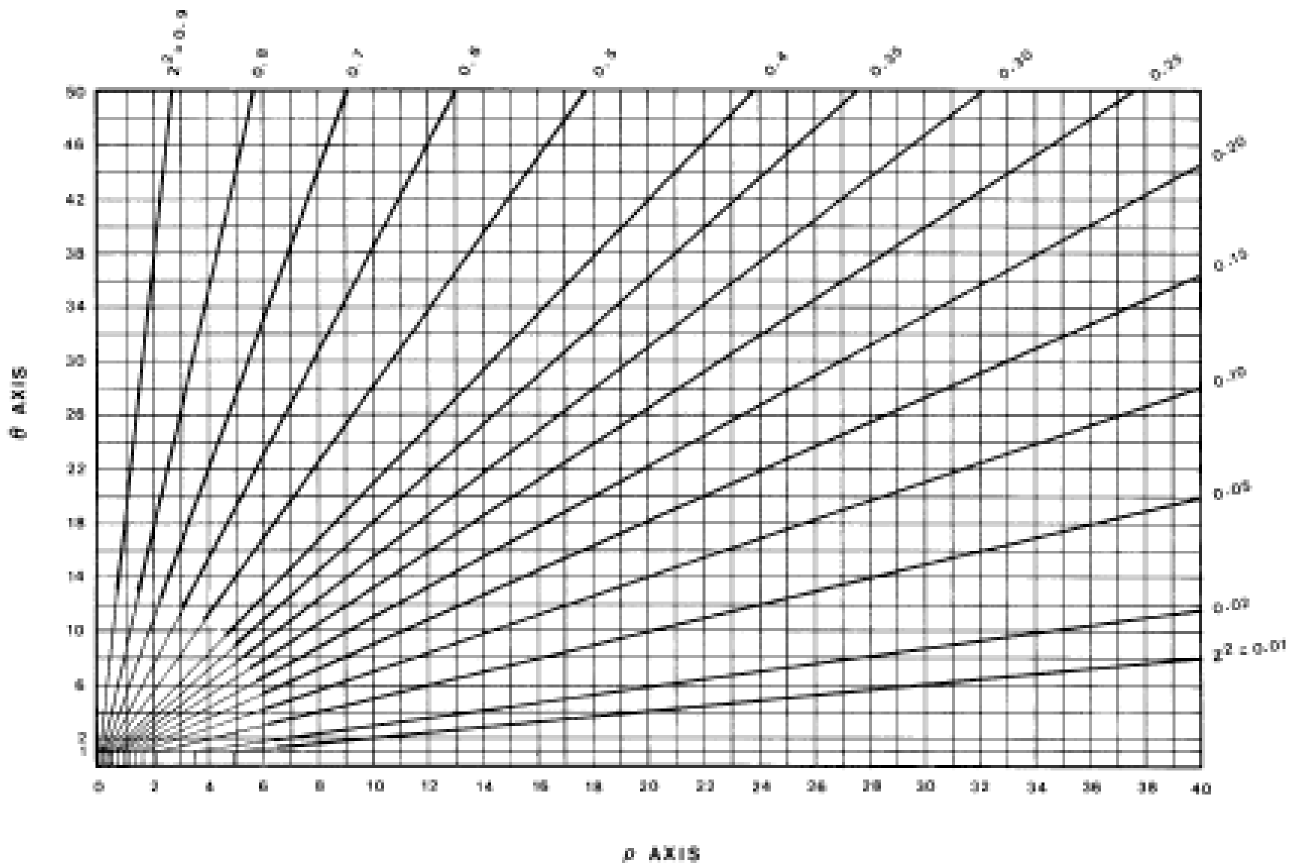


FIGURE 12

ALLIEVI CHART — WATER HAMMER
MAXIMUM DROP IN PRESSURE FOR UNIFORM GATE MOTION
AND SIMPLE CONDUITS (FOR LARGE VALUES OF ρ AND θ)

7 TAILRACE CANAL

7.1 BACKGROUND

After passing though the turbine flow is returned to the river via the tailrace canal. The main objectives of hydraulic design of the tailrace channel are:

- Determine the head-discharge relationship at the powerhouse, from which turbine runner centre line and main floor elevations can be determined.
- Establish optimal canal layout and cross-section dimensions.
- Determine appropriate canal lining and/or erosion protection

7.2 DATA REQUIREMENTS

It is of utmost importance that sufficient data be collected to establish a reliable head-discharge relationship in the river opposite the outfall of the tailrace canal. Ideally a relationships established by flow and water level measurement is preferred. Such a site specific measurement program is usually impractical. It is normally more practical to carry out a cross-section survey of the river downstream of the powerhouse. The choice and of cross section locations should be representative of the river channel. Cross-sections should be extended above the visible high water level to accommodate at least the 1 in 100 year flood; also the water level at each cross-section should be recorded during the survey. If possible the reach surveyed should start above a control section. However, river flow at most locations along a river will be close to the normal depth of flow; accordingly, the initial section for backwater calculation should be located at a uniform section of the river and the length of reach sufficient that errors in estimating the starting water level at this section will be sufficiently attenuated before reaching the location of the tailrace outfall. Several flow measurements should be made for use in estimating Manning's "n", with one at the time of river survey. This data is needed to establish the tailrace head-discharge curve.

7.3 LAYOUT

The powerhouse–tailrace setting is usually determined by practical engineering judgment, taking into account: access, space requirements, foundation conditions and the like. Where site conditions are suitable it is recommended that the tailrace canal be oriented to discharge at an angle of 30°-45° to the centre line of the receiving river. This will help in keeping the tailrace channel clear of bed load deposits that could cause back water effects at the powerhouse or require expensive maintenance dredging to control waters levels.

7.4 HYDRAULIC DESIGN

7.4.1 Head discharge Curve:

Determine the head-discharge curve for the river at the tailrace outfall and at the plant. This will require backwater computation, as outlined in most standard text on open channel hydraulics. Suitable values for Manning's "n" coefficient are given in Appendix 1 to this section.

The following water levels are of particular interest:

a) TWL for minimum plant / river flow combination to establish the turbine centre line elevation.
b) TWL corresponding to the 1 in 100 year flood (Q_{100}) to establish plant main floor elevation.

7.4.2 Design of Tailrace Canal

The design of the tailrace canals differs from power canal design in that water tightness is of less importance; accordingly an earthen canal with rock/riprap lining is often satisfactory. The dimensions of the canal should be based on economic optimization, as recommended in Section 2.2.2/2 for power canals. For short tailrace canals ($L \leq 50$m) detailed optimization analysis may be omitted and design based on a velocity of 1.5 – 2.0 m/s with the design flow (Q_P).

7.4.3 Erosion Protection

Detailed design of the canal section, slope and erosion protection for earthen canals should be done using the “Tractive Force Method”, (Ven T. Chow pp. 168). The design should be based on the maximum tailrace flow (Q_p), but the erosion protection should be extended up to the 1 in 100 year level.

7.4.4 Reference

Open Channel Hydraulics
Ven T. Chow
Mc Graw Hill Book Company
New York (1959)

7.5 Appendix: Manning's "n" values.

Table 3.1
Manning's 'n' Values

Type of Channel and Description	Minimum	Normal	Maximum
A. *Natural Streams*			
1. **Main Channels**			
a. Clean, straight, full, no rifts or deep pools	0.025	0.030	0.033
b. Same as above, but more stones and weeds	0.030	0.035	0.040
c. Clean, winding, some pools and shoals	0.033	0.040	0.045
d. Same as above, but some weeds and stones	0.035	0.045	0.050
e. Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
f. Same as "d" but more stones	0.045	0.050	0.060
g. Sluggish reaches, weedy. deep pools	0.050	0.070	0.080
h. Very weedy reaches, deep pools, or floodways with heavy stands of timber and brush	0.070	0.100	0.150
2. **Flood Plains**			
a. Pasture no brush			
1. Short grass	0.025	0.030	0.035
2. High grass	0.030	0.035	0.050
b. Cultivated areas			
1. No crop	0.020	0.030	0.040
2. Mature row crops	0.025	0.035	0.045
3. Mature field crops	0.030	0.040	0.050
c. Brush			
1. Scattered brush, heavy weeds	0.035	0.050	0.070
2. Light brush and trees, in winter	0.035	0.050	0.060
3. Light brush and trees, in summer	0.040	0.060	0.080
4. Medium to dense brush, in winter	0.045	0.070	0.110
5. Medium to dense brush, in summer	0.070	0.100	0.160
d. Trees			
1. Cleared land with tree stumps, no sprouts	0.030	0.040	0.050
2. Same as above, but heavy sprouts	0.050	0.060	0.080
3. Heavy stand of timber, few down trees, little undergrowth, flow below branches	0.080	0.100	0.120
4. Same as above, but with flow into branches	0.100	0.120	0.160
5. Dense willows, summer, straight	0.110	0.150	0.200
3. **Mountain Streams, no vegetation in channel, banks usually steep, with trees and brush on banks submerged**			
a. Bottom: gravels, cobbles, and few boulders	0.030	0.040	0.050
b. Bottom: cobbles with large boulders	0.040	0.050	0.070

Table 3.1 (Continued)
Manning's 'n' Values

Type of Channel and Description	Minimum	Normal	Maximum
B. *Lined or Built-Up Channels*			
1. **Concrete**			
a. Trowel finish	0.011	0.013	0.015
b. Float Finish	0.013	0.015	0.016
c. Finished, with gravel bottom	0.015	0.017	0.020
d. Unfinished	0.014	0.017	0.020
e. Gunite, good section	0.016	0.019	0.023
f. Gunite, wavy section	0.018	0.022	0.025
g. On good excavated rock	0.017	0.020	
h. On irregular excavated rock	0.022	0.027	
2. **Concrete bottom float finished with sides of**:			
a. Dressed stone in mortar	0.015	0.017	0.020
b. Random stone in mortar	0.017	0.020	0.024
c. Cement rubble masonry, plastered	0.016	0.020	0.024
d. Cement rubble masonry	0.020	0.025	0.030
e. Dry rubble on riprap	0.020	0.030	0.035
3. **Gravel bottom with sides of**:			
a. Formed concrete	0.017	0.020	0.025
b. Random stone in mortar	0.020	0.023	0.026
c. Dry rubble or riprap	0.023	0.033	0.036
4. **Brick**			
a. Glazed	0.011	0.013	0.015
b. In cement mortar	0.012	0.015	0.018
5. **Metal**			
a. Smooth steel surfaces	0.011	0.012	0.014
b. Corrugated metal	0.021	0.025	0.030
6. **Asphalt**			
a. Smooth	0.013	0.013	
b. Rough	0.016	0.016	
7. **Vegetal lining**	0.030		0.500

Table 3.1 (Continued)
Manning's 'n' Values

Type of Channel and Description	Minimum	Normal	Maximum
C. *Excavated or Dredged Channels*			
1. **Earth, straight and uniform**			
a. Clean, recently completed	0.016	0.018	0.020
b. Clean, after weathering	0.018	0.022	0.025
c. Gravel, uniform section, clean	0.022	0.025	0.030
d. With short grass, few weeds	0.022	0.027	0.033
2. **Earth, winding and sluggish**			
a. No vegetation	0.023	0.025	0.030
b. Grass, some weeds	0.025	0.030	0.033
c. Dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
d. Earth bottom and rubble side	0.028	0.030	0.035
e. Stony bottom and weedy banks	0.025	0.035	0.040
f. Cobble bottom and clean sides	0.030	0.040	0.050
3. **Dragline-excavated or dredged**			
a. No vegetation	0.025	0.028	0.033
b. Light brush on banks	0.035	0.050	0.060
4. **Rock cuts**			
a. Smooth and uniform	0.025	0.035	0.040
b. Jagged and irregular	0.035	0.040	0.050
5. **Channels not maintained, weeds and brush**			
a. Clean bottom, brush on sides	0.040	0.050	0.080
b. Same as above, highest stage of flow	0.045	0.070	0.110
c. Dense weeds, high as flow depth	0.050	0.080	0.120
d. Dense brush, high stage	0.080	0.100	0.140

Limerinos (1970) related n values to hydraulic radius and bed particle size based on samples from 11 stream channels having bed materials ranging from small gravel to medium size boulders. The Limerinos equation is as follows:

$$n = \frac{(0.0926)R^{1/6}}{1.16 + 2.0\log\left(\dfrac{R}{d_{84}}\right)} \qquad (3\text{-}2)$$

where: R = Hydraulic radius, in feet (data range was 1.0 to 6.0 feet)
d_{84} = Particle diameter, in feet, that equals or exceeds that of 84 percent of the particles (data range was 1.5 mm to 250 mm)

8 TEMPORARY RIVER DIVERSION DURING CONSTRUCTION

8.1 BACKGROUND

Temporary river diversion is normally required to facilitate construction of dams and other works located in the river bed. Depending on the magnitude of river flows the design and construction of diversion works can be difficult and expensive. Typically, diversion works comprise, 20% to 25% of head works capital costs, but costs vary greatly depending on site features and hydrology. Construction of head works and related temporary river diversion works are weather dependent and constitute a key activity in any project construction schedule. The design engineer needs to be aware of the importance of this activity and to ensure that his estimate includes an adequate allowance for the costs of temporary river diversion and that his construction plan allows for the challenges of in-river construction.

The following factors influence the design of temporary river diversion works:

- Duration of construction of in-river structures.
- Vulnerability to overtopping (concrete dams versus embankment dams).
- Stream flow characteristics.
- Magnitude and duration of floods during construction period.

The climate in most of India is characterized by two distinct seasons, a wet season with high flows, and a dry season with low flows. The dry season provides the best conditions for construction of in-river works as the flows to be handled are much smaller than during the wet season. Accordingly, it would be advantageous to schedule construction of in-river structures and related temporary diversion works for the dry season. For SHP's it may be possible to complete all vulnerable works within a single dry season, or at least to advance the work to a stage where the incomplete works are safe from wet season floods.

8.2 SELECTION OF DIVERSION FLOOD:

The design of temporary river diversion works involves evaluation of risk versus cost of diversion works. Risk could include:

- Damage to the works.
- Downstream damages.
- Cost of delays.
- (Sometimes) dangers to public health and safety.

These risks are mainly attributed to site hydrology and the frequency of occurrence of large floods. Of course there are other risks related to design and construction of the cofferdams and other water diversion structures, which must be controlled by competent design and attention to quality control of construction.

On large projects the design flood is sometimes determined from cost – benefit analysis of the issue. However, it is more common to apply design criteria based on precedents.

For SHP the following criteria are recommended:

- For concrete or gabion dams that can resist limited overtopping - the 1 in 10 year flood (Q_{10})
- For embankment dams that would be destroyed or severely damaged - the 1 in 20 year flood (Q_{20}).

Where the works can reasonably be completed within a single dry season, flood frequencies should be computed for dry season floods, otherwise annual floods should be used.

8.3 METHODS OF CONSTRUCTION

8.3.1 In Situ:

For very small rivers construction of the head works may be possible without diversion. In such cases, stone barriers, gabion dams or crib dams could be built in running water and the impermeable element, added later. Impermeable barriers could be timber planking, plastic sheeting or a barrier of sand-cement bags. The in-situ work assumes water depths generally less than 0.5 m or “knee depth”. Flow depths much greater than this would be too dangerous, especially if flow velocities were high (>1.0 m/s).

8.3.2 Simple Diversion techniques

For small to medium sized rivers, water could be diverted via flumes, culverts or ditches as shown in the following photographs from the *Design of Small Dams (USBR – 1987).*

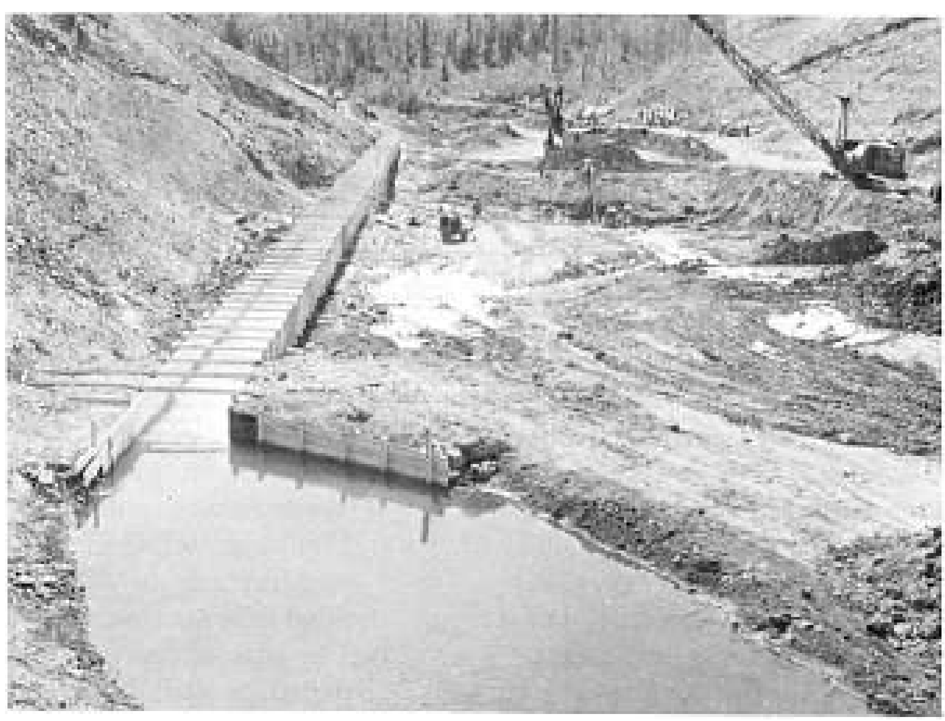


Figure 11-1: ***Diversion by flume.***

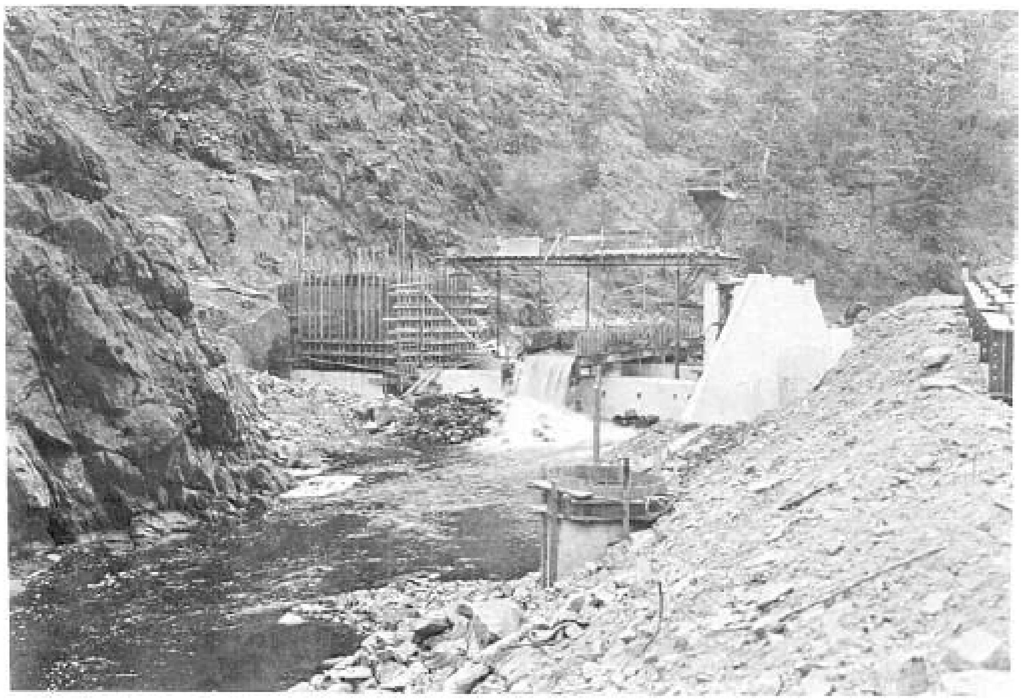


Figure 11-2.—Temporary diversion flume used during construction of a concrete dam. Horsetooth Feeder Canal Tunnel No. 1. CBT 245-704-330.

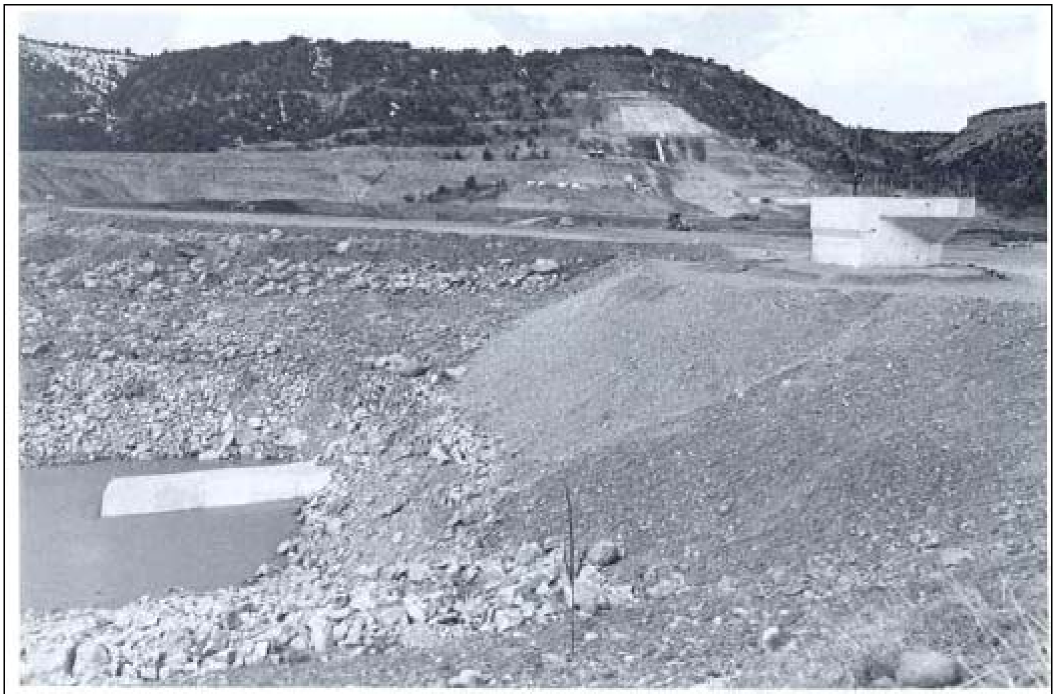


Figure 11-3.—Diversion through an auxiliary stream-level conduit. Intake structure (under construction) is at upper right. Ridgway Dam. P894-427-5891 NA.

Figure 11-8.—Temporary diversion channel through an earthfill dam. Bonny Dam. 414-289C.

8.3.3 Staged Diversion

On large rivers a staged diversion approach is sometimes employed. For example: in Stage I, portion of the river bed is un-watered behind cofferdams to permit construction of the spillway. While in Stage II the dam and / or powerhouse is constructed while diverting water though the spillway. There are many variation of this approach depending on site topography and features of the head works.

Continued on next page.

8.3.4 Diversion by tunnel

In confined, narrow valleys diversion via tunnel is often the preferred approach.

Figure 11-5.—Diversion adit and upstream cofferdam at Seminoe Dam.

8.3.5 Cofferdams

A cofferdam is a temporary dam or barrier to divert a stream or enclose an area during construction. The most common type of cofferdam is a rock embankment or berm built across a river by end dumping with an upstream zone of impermeable fill for water proofing (also placed by dumping). If large size rocks are used for the embankment an intermediate filter zone may be required. For small cofferdams polyethylene sheets can also be used. Alternatively, cofferdams may be composed of crib or sheet pile cells.

8.4 RESPONSIBILITIES

8.4.1 Contractor's Responsibilities:

It is general practice to require the contractor to assume responsibility for the diversion of the stream during the construction of the dame and appurtenant structures. This requirement should be defined by appropriate paragraphs in the specifications that describe the contractor's responsibilities and define the provisions incorporated in the design to facilitate construction. Usually, the specifications should not prescribe the capacity of the diversion works or the details of the diversion method to be used, but hydrographs prepared from available stream-flow records should be included. In addition, the specifications usually require that the contractor's diversion plan be subject to the owner's approval.

8.4.2 Designer's Responsibilities

For difficult diversion situations, it may prove economical for the owner to assume the responsibility for the diversion plan. One reason for this is that contractors tend to increase bid prices for river diversion if the specifications contain many restrictions and there is a large amount of risk involved. Where a dam is to be constructed in a narrow gorge, a definite scheme of cofferdams and tunnels might be specified, because the loss of life and property damage might be heavy if a cofferdam were to fail.

Another point to consider is that the orderly sequence of constructing various stages of the entire project often depends on the use of a particular diversion scheme.

8.5 REFERENCE

Design of Small Dams
U.S. Bureau of Reclamation
Denver, Colorado (1987)

9 ENVIRONMENTAL MITIGATION WORKS

9.1 INTRODUCTION

9.1.1 Background

This section deals with the hydraulic design of environmental mitigation works that may be required by the responsible authority. The extent of mitigation works will vary greatly between projects as a function of the setting and features of a given project. On some projects no mitigation works may be needed; whereas, on others expensive works may be required.

9.1.2 Scope

The guidelines cover the most common types of mitigation works:

- Supply of reserve flows
- Fishways
- Compensation channels

9.2 RESERVE / RIPARIAN FLOW RELEASES

The responsible authority may require minimum flows be maintained in the river channel downstream of a diversion dam.

These minimum flows could be required for the following reasons:

1. For use of people living beside the river in the impacted area (between diversion dam and powerhouse). In this case the hydropower plant developer should investigate the relative economics of providing piped water to the affected households.

2. For meeting the biological needs of biota living in the river reach between diversion dam and powerhouse, in this case the requirements may vary from month to month.

During high flow periods excess flows released at the head works of run-of-river plants will usually suffice to meet reserve flow requirements. On the other hand during low flow periods where plant demand is equal on greater than inflow, reserve flows must be provided by releases from the reservoir. While such releases may be provided by opening a spillway gate or sediment flushing gate – it is recommended that a pipe sized for this purpose with a control valve be installed.

9.3 FISHWAY

9.3.1 Background

Fishways are required on rivers where one or several important fish species need to migrate upstream as part of their life cycle requirements. A fishway provides a means for fish to bypass a diversion dam which in other circumstances would be a barrier to fish migration.

The main types of fishways are:

- Vertical slot fishways

- Pool and weir fishways
- Denil fishways

The vertical slot type is recommended as its function is relatively stable over a wide range water level variation; additionally its construction is relatively simple. The following guidelines describe its features and recommend appropriate design parameters.

9.3.2 Biological Design Criteria

Biological design criteria must be defined in consultation with an experienced fisheries biologist as below:

1. Target specie, mean size (length) and range of sizes (at 5% and 95% exceedence limits). While design is based on a specific target species, other fish having similar or better swimming abilities, will also use the fishway.
2. Period of upstream migration and any thresholds of flow, temperature (or other) that would control upstream migration.
3. Rate of Migration: mean and peak daily rate of migration in numbers per hour.
4. Swimming ability: burst swimming speed. If this is not known for the target the target species, estimates can be obtained from the following graphs:

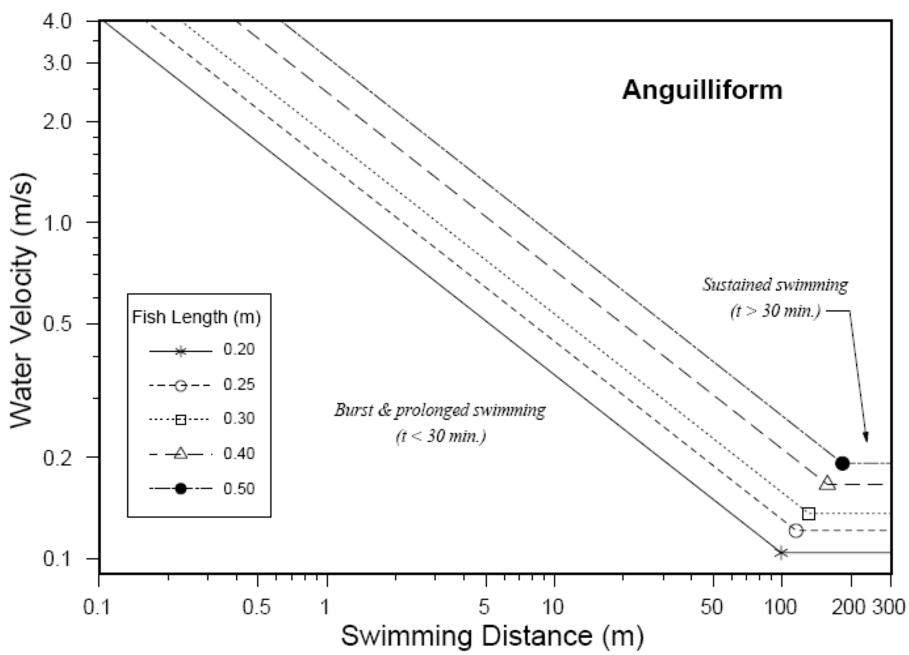


Figure 6.4 Swimming distance curves for several fish lengths (Anguilliform mode).

(From Katopodis – 1992).

Note: Anguilliform fish have long slender bodies and are weak to moderate swimmers

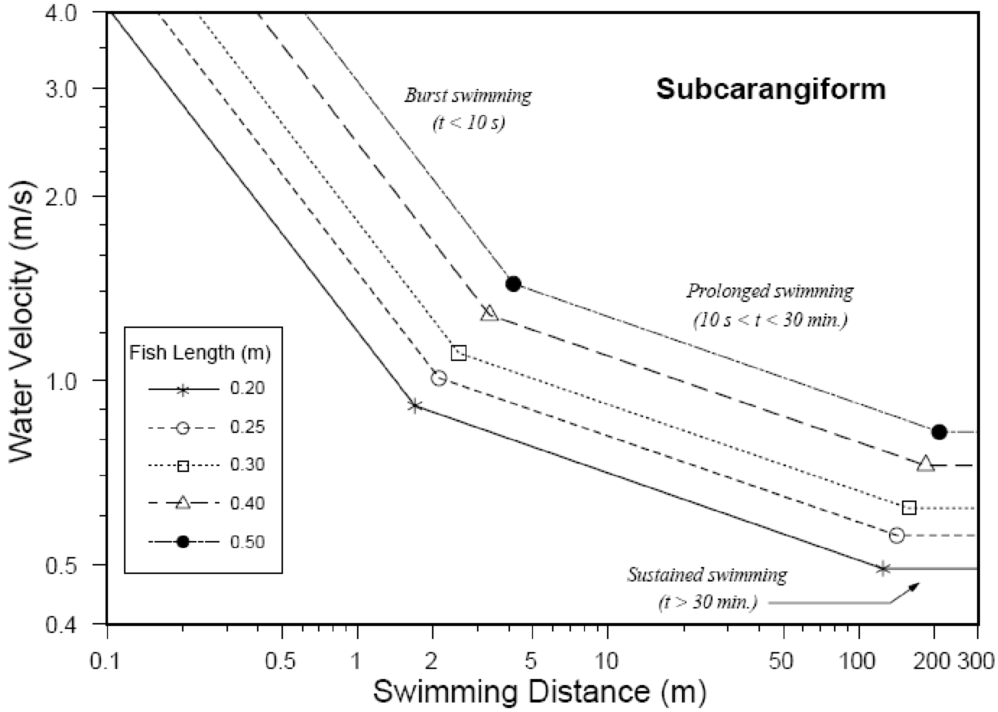


Figure 6.5 Swimming distance curves for several fish lengths (Subcarangiform mode).

(From Katopodis – 1992).

Note: Subcarangiform fish have torpedo shaped bodies and are strong swimmers, examples – trout and salmon.

9.3.3 Principles of Design

Fishways are usually planned according to the following principles:

1. The layout should be designed so that there is a significant velocity in the area approaching the fishway. In their upstream migration fish use the current as a direction guide, so if the entrance to the ladder is located in a dead area the fish may not find it. The velocity should be about 1.0 m/s. Creation of such a velocity condition is called "attraction water", as its is intended to attract the fish.
2. The upstream exit for fish from the fishway should be in a quiet area well away from the overflow section or sluiceway; otherwise the fish may be carried back downstream.
3. Maximum velocities in the fishway should not exceed the burst speed (or darting speed) for the fish. This is the speed that the fish can swim for a few second and is in the order of 8 to 12 body lengths per second. A typical velocity is 2.5 m/s. At this velocity the drop in elevation between pools is limited to about 0.3m, since this drop is converted to velocity

head between pools. On short fishways higher drops have sometimes been used, up to 0.6m. It has been observed that 2.5 m/s is a velocity that the fish can comfortably swim against for a short duration. Higher velocities tend to discourage some fish from using the fishway; whereas, lower velocities would increase the length and cost of the structure.

4. Average velocities in the fishway should be about 0.30 to 0.45 m/s. This velocity is the basis for selecting the fishway discharge from Q = VA, in which A is the cross sectional area of the ladder. This relatively low average velocity has been found necessary to allow rest stops for the fish while going up the ladder. Fish do not feed during upstream migration, but may rest about 4 hours each day.
5. The volume of the fishway should provide from 0.06 to 0.12 m^3 of water per fish, depending on the size of the fish. It has been observed that, given sufficient space to maneuver, the fish will not injure themselves even on the sharpest corners or baffles. *Items 1 to 5 from Smith (1995).*
6. For fishway design flow, first determine the mean river flow for the season of upstream migration (MSF). Base design of fishway on the lesser of 0.10 MSF or 5.7/m^3s.
7. Minimum depth of water opposite the entry to fishway: 0.6m to 0.9 m and at the exit 0.6m.

Given data on the peak migration rate and the rate of climb, a suitable fishway can be designed with no other information except the foregoing seven principles. Fish are overly cautious when proceeding in unfamiliar channels and the average rate of climb is surprisingly low often only 2.5 to 3.5 m/h. Biologists may estimate migration rate from fish tagging data or by use of counting fences.

9.3.4 Design Procedure:

A stepwise design procedure is suggested, as outlined below:

9.3.4.1. *Establish Biological Criteria:*

The following biological design criteria must be decided:

- Target specie or species and sizes.
- Period (season) of upstream migration.
- Migration rate (for peak week) fish per hour (N_a).
- Fish swimming characteristics (V_b).
- Vertical rate of ascent through the fishway (V_a).

9.3.4.2. *Collect site data*

The following data are required:

- Daily flow data.
- Bathymetry at inlet and outlet zones.
- Head - flow relationships for fishway inlet and outlet zones.

9.3.4.3 Determination of Design Flow (Q_d)

Calculate mean river flow for migration season and determine fishway design flow as the lesser of 0.10 MSF or 5.7 m^3/s (Q_d)

9.3.4.4 Estimate Head Difference (ΔH) Between Pools and Number of Pools

Let $V_s = 0.9$ Vb

Then $\Delta h = \dfrac{V_s^2}{2g}$ and

Number of pools $(N) = \left(\dfrac{HWL - TWL}{\Delta h}\right)$ --------(1)

Rounded up to nearest whole number

Where: *HWL* = *Head water level*
TWL = *Tail water level*

9.3.4.5 Estimate the number of Fish per pool

Number of fish per pool $(N_f) = \dfrac{N_o}{N}$ --------(2)

Then volume of pool $(V_o) = N_f \times \nu$ ------(3)

where:
V_o = Volume of pool
ν = volume of water per fish (0.06 m^3 to 0.12 m^3).

9.3.4.6 Pool Dimensions

Estimate slot width $(b_o) = 0.1 + 0.1\ Wt$ -------------- --(4)

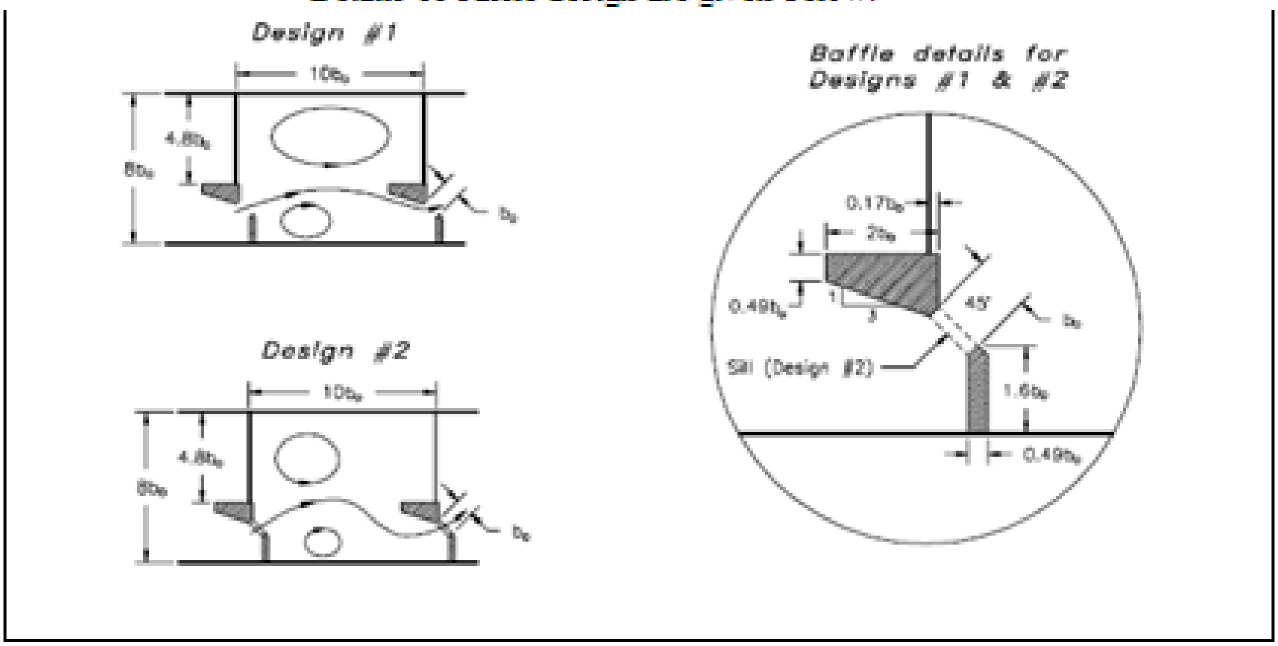
Where: Wt = weight of fish in kg.

Then depth downstream of baffle wall $(D_s) = \dfrac{Q}{V_s b_o}$ ---(5)

Following Katopodis, the recommended plan form geometry is:

Width (W_o) = 8 b_o
Length (L_o) = 10 b_o ----------------------------(6)

Details of baffle design are given below:



(From Katopodis – 1992).

$$Slope\ (s) = \frac{\Delta h}{L_p}\ \ typically\ \ S = 1(V) : 8\ to\ 10\ (H)$$

Mean pool depth (D_m) $> (D_2 + SL_P.0.5)$ *or* $0.6m$ −−−−−−(7)

Volume of pool $(V_p^{'} = W_p.L_p.D_2)$

Check the following:

- Volume of pool greater of V_p or V'_p.
- Mean velocity through pool $(U_m) = \frac{Q}{D_2.W_p}$ (usually 0.3 to 0.45 m/s)

Adjust dimensions, D_2, W_p and L_p to meet the above criteria, as required.

A sample design is shown below:

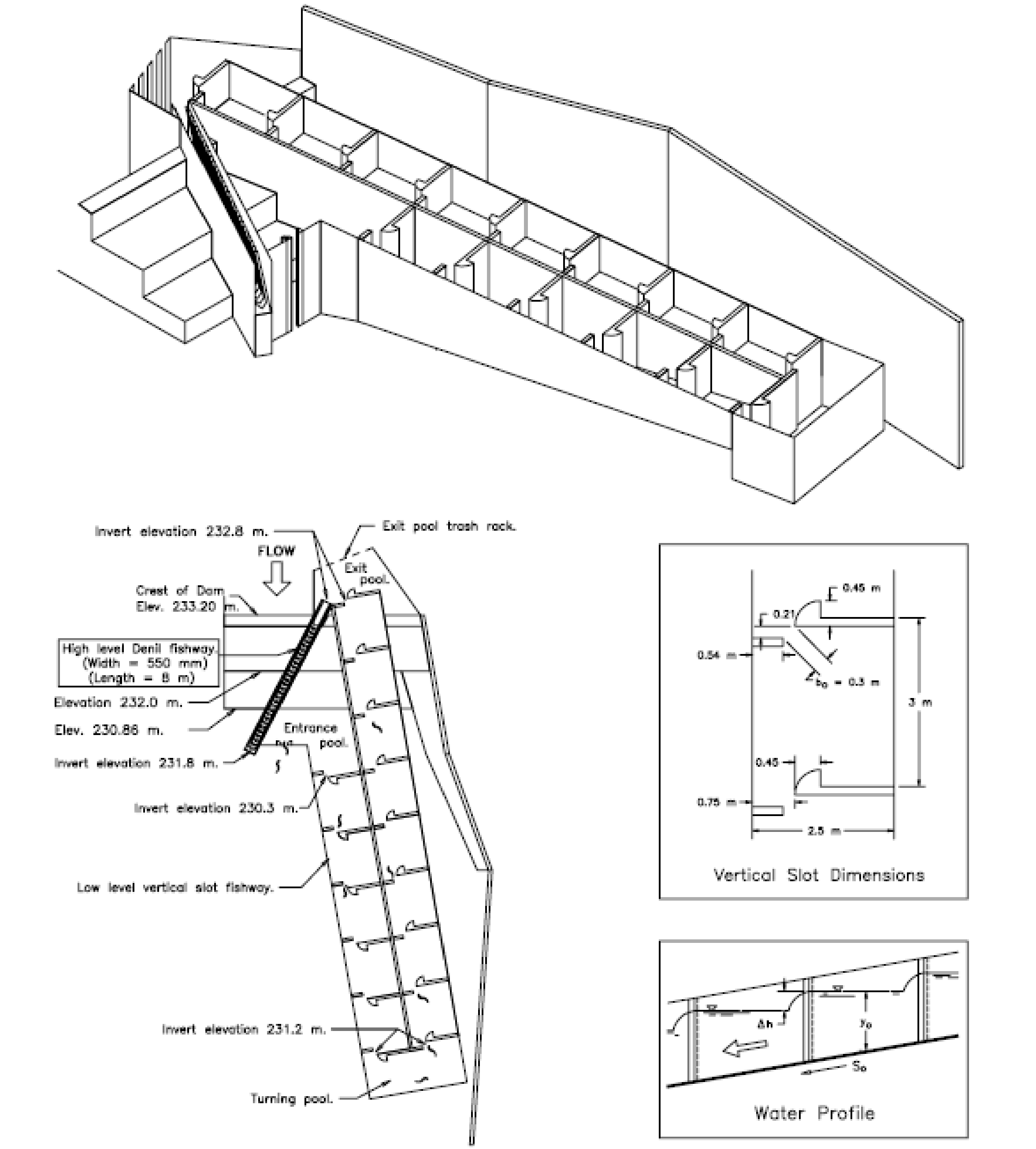


Figure 8.6. Isometric and plan view of the Hunt Dam fishways.

(From Katopodis – 1992)

Fish tend to migrate downstream during periods of high flow and on run-of-river plants at this time most fish follow the main current and pass downstream via overflow spillway or spillway gates and only a few fish are likely to enter the plant waterways. Therefore downstream passage of fish is not usually a problem with run-of-river plants. Exceptionally, guidance of downstream migrants away from plant intakes is a problem. Approaches to dealing with this problem are discussed in ASCE Guidelines for Design of Intakes and ESHA's Layman's Guidebook.

9.3.4.7 COMPENSATION CHANNELS

Artificial channels are sometimes constructed to compensate for lost habitat. These channels are called "compensation channels" and they are designed to mimic real rivers with pools and riffles and a variety of substrate as would be found in nature. A fisheries biologist should be consulted to advise biological requirements. Often a compensation channel can be constructed in the power plant tailrace channels. A discussion of design of artificial river habitat is given by Newbury and Gaboury (1994).

9.3.4.8 REFERENCES CITED

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