

3. Water Conveyance Structures

3.1 Intakes and Head Race

3.1.1 Water Intake, Inlet Structures

The intake is a structure constructed at the entrance of a power canal or tunnel or pipe through which the flow is diverted from the source such as a river or reservoir. It is an essential component of hydropower schemes and provided as an integral part or in isolation from the diversion, weir or dam.

3.1.2 Functions of Intakes

The main functions are:

- i) To control flow of water in to the conveyance system. The control is achieved by a gate or a valve.
- ii) To provide smooth, easy and vortex or turbulence free entry of water in the conveyance system this is to minimize head loss. This can be achieved through providing bell-mouth shaped entrance.
- iii) To prevent entry of coarse river born trash matter such as boulders, logs, tree branches etc. Provisions of trash racks at the entrance achieve this function.
- iv) To exclude heavy sediment load of the river from interrering the conveyance system. Special devices such as silt traps and silt excluders are used to control & trap the silt.

3.1.3 Types of Intakes

Intakes are conveniently classified in to the following types depending on the power plant type and its layout.

- i) Run - of - river intakes
- ii) Canal intakes
- iii) Dam intakes
- iv) Tower intakes
- v) Shaft intakes
- vi) Intakes of special type

i) Run - of - river intakes

The component parts are

- Bell mouth entrance guarded by R.C or still grid forming the trash rack structure.
- Control gate situated immediately d/s of the bell mouth entry
- Upstream of the gate may be provided with stop-log groves for provision of access to the gate for repair.
- Special de-silting arrangement may be provided in silty rivers.

ii) Canal Intakes

- Water is admitted in to the diversion canal.
- Silt excluders or silt-traps are usually essential components of such intakes.
- The inlet invert level of the intake is raised to form a sill so as to prevent entry of rolling bed load.
- A skimmer wall (a diaphragm which extends below the water surface) abstracts the floating material from interrering in to the canal.

- Trash racks are also fitted at the entrance.
- Vertical lift gate with motorized operation are used to control the flow.

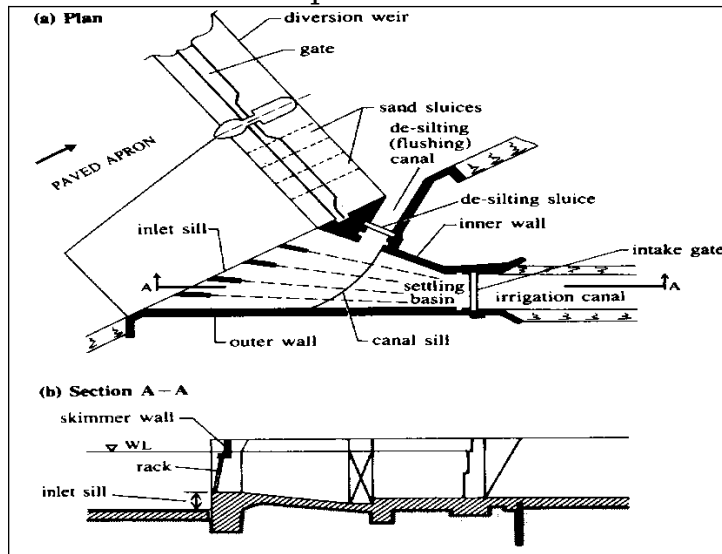
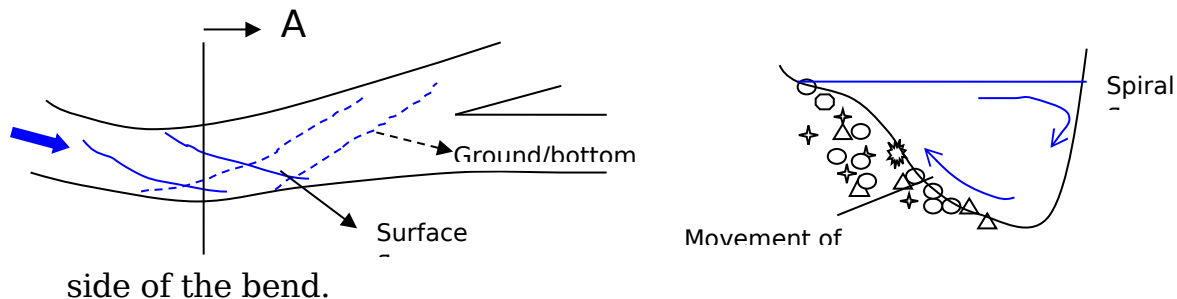


Figure 3.1: Example of canal intake layout

Basic principles governing selection of diversion site from streams:

1. Intakes from streams should be located, wherever possible on the concave

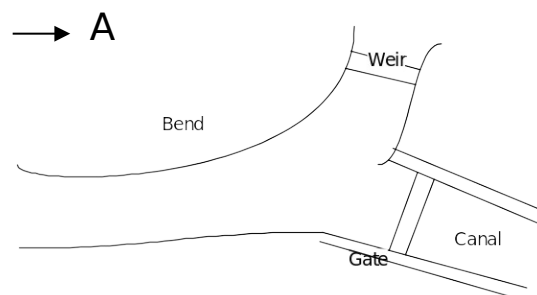


side of the bend.

Figure 3.2: Spiral flow in river bends

Section A-A

2. The effectiveness of the intake in preventing sediment entry increases with the sharpness of the bend.
3. Intakes from straight reaches can be made favorable by artificially forcing the water to follow a curved [path].



Typical layouts

Figure 3.3: In a bend

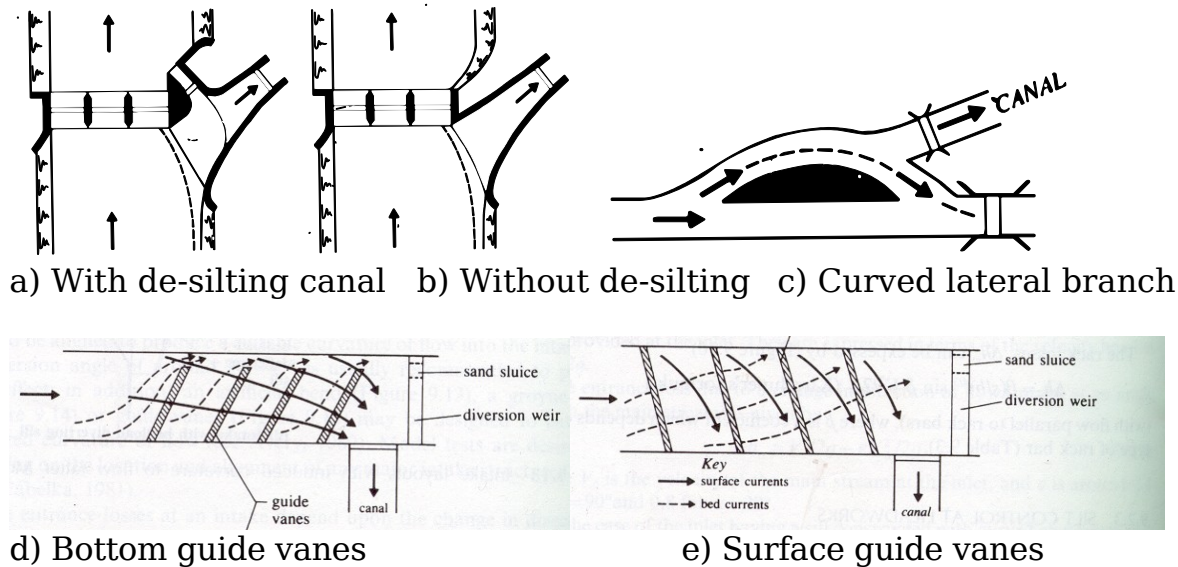


Figure 3.4: In straight reach

iii) Dam intakes

- For valley dam plants, the intake structure is provided usually in the body of the dam.

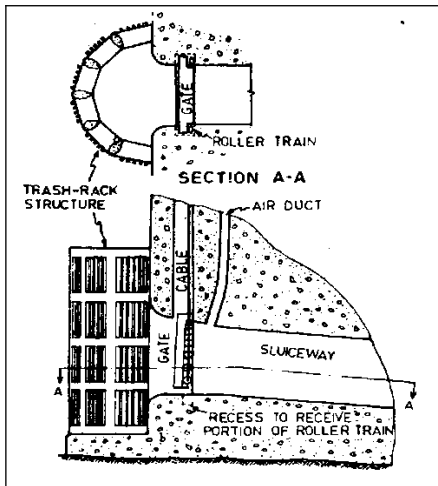


Figure 5.5: Example of a dam intake

- The penstocks are embodied in the dam.
- the main features of such an intake are
 - A trash rack structure in front of the dam.
 - A bell mouth inlet horizontal or inclined alignment
 - A control gate installed either at or after the bell mouth. Cage-shaped intakes resting against the face of the dam and supported on slab cantilevered from the dam provide larger area of entry than the penstock intake area, thus reducing entrance losses.

Multi-level water is also sometimes used in dam intakes.

iv) Tower Intakes

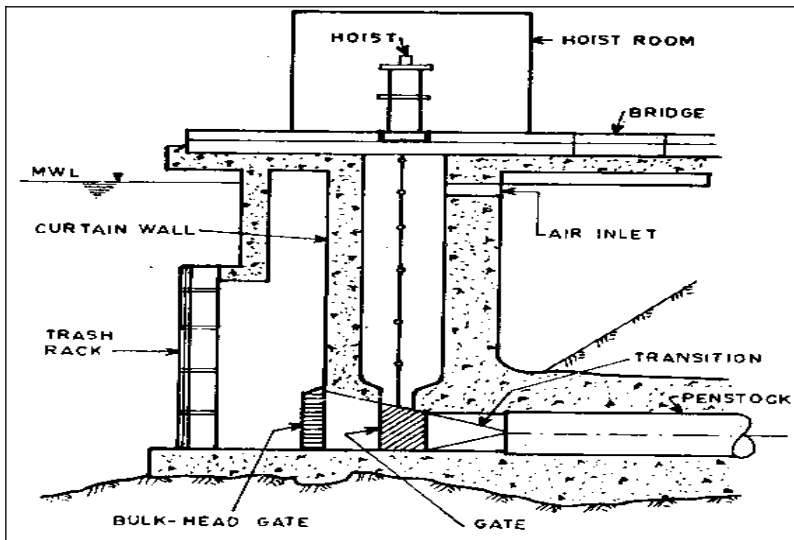


Figure 3.6: Tower Intake

- Used when it is not convenient to provide the simple intake directly on the u/s face of the dam.
- Also used when there are wide fluctuations in water level.
- Tower may be connected with main dam through a bridge when the tower is near the dam
- Flow in to the tower is controlled by a number of gates to close or open the ports at various levels.
- Flow through the pressure conduit is controlled by vertical uplift gates.
- The structure should be strong enough to withstand hydrodynamic, earthquake, wind, etc.

v) Shaft intakes

- This is a vertical shaft that carries water to the penstock tunnel. It consists of the following.
- The entrance structure with trash rack and rounded inlet.
- The vertical shaft followed by an elbow and transition connecting the shaft with the tunnel.
- The intake gate (cylindrical) and sometimes a stop-log closure.

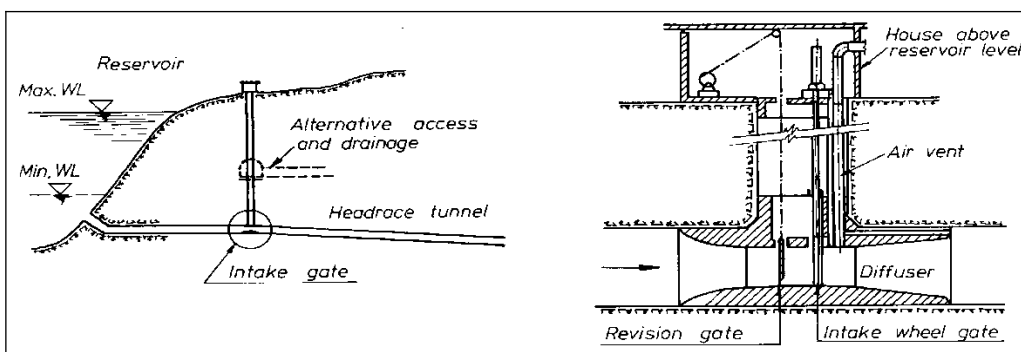


Figure 3.7: Shaft Intake**3.1.4 Losses in Intakes**

The intake losses include entrance loss, trash rack loss and head gate loss.

Entrance Losses

These comprises of:

- a) Loss due to change in direction is given by:

$$\Delta h_e = \frac{V^2}{2g} - C \frac{V_f^2}{2g} \quad \text{-----} \quad (5.1)$$

Where V is velocity in the diversion canal

V_f is velocity of flow in the main river

C is a constant which depends on the off-take angle of the diversion canal.

According to Mossonyi, C is equal to 0.8 for 30° off-take angle and 0.4 for 90° off-take angle.

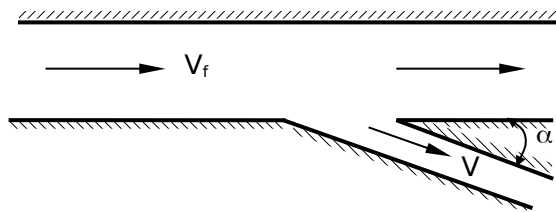


Figure 3.8 Losses due to change of flow direction

- b) The losses due to sudden contraction of the area at the inlet section is given by:

$$\Delta h_e = K \frac{V^2}{2g} \quad \text{-----} \quad (5.2)$$

Where K is a constant, which depends on the shape of the entry

$K=0.03$ for bell-mouthed entry

$K=1.3$ for sharp cornered entry.

In cases of the inlet having a sill constructed with curved abutments and piers, the head loss, Δh_e , is given by:

$$\Delta h_e = 0.3 \frac{V^2}{2g} \quad \text{-----} \quad (5.3)$$

$$\text{Therefore, maximum total entrance loss} = 1.3 \frac{V^2}{2g} - C \frac{V_f^2}{2g} \quad \text{-----} \quad (5.4)$$

Rack Losses:

There are numerous expressions available for predicting head loss across trash racks. One such expression (after Kirschmer's) is:

$$\Delta h_r = K_t \left(\frac{t}{b} \right)^{4/3} \frac{V_a^2}{2g} \sin \phi \quad \text{-----} \quad (5.5)$$

Where, K_t is trash rack loss coefficient (a function of bar shape), t is bar thickness, b is spacing between bars, V_a is approach velocity, and ϕ is angle of inclination of bars with the horizontal.

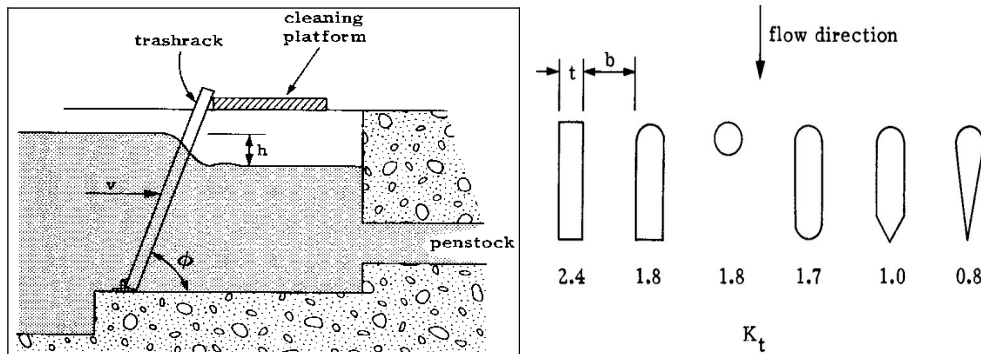


Figure 3.9: Rack losses

Gate Losses:

Head loss due to gates (at part gate opening) is given by:

$$\Delta h_g = \frac{1}{2g} \left(\frac{Q}{C_d A} \right)^2 \quad \text{-----} \quad (5.6)$$

Where, Q is flow in the canal or conduit, A is area of gate opening, and C_d is discharge coefficient which varies between 0.62 and 0.83.

3.1.5 Velocity through Trash Racks

Velocity should be sufficiently low to avoid high head loss and should be sufficiently high to avoid large intake and trash rack cross section. The following are suggested limiting entrance velocities:

i) Justin and Creager formula:

$$V \leq 0.12 \sqrt{2gh} \quad \text{-----} \quad (5.7)$$

h = head from center line of gate to normal water surface

ii) Mosonyi's formula to eliminate eddies and vortices:

$$V \leq 0.075 \sqrt{2gh} \quad \text{-----} \quad (5.8)$$

iii) U.S.B.R's criterion: permissible velocity in the range of 0.6 to 1.5 m³/s

3.1.6 Air Entrainment and Vortex Formation at Intakes

Air entrainment is due to development of vortices and also due to partial gate opening that facilitates hydraulic jump formation. Effects of air entrainment are: Additional head losses, reduction in discharge and drop in efficiency of turbines. Minimizing vortex formation and avoiding hydraulic jump formation help in preventing air entrainment.

A vortex which forms at the inlet to power conduit occasionally can cause troubles by itself. It can induce loss of turbine efficiency, possible cavitation, surging caused by the formation and dissipation of vortices, and flow reduction as air replaces part of the water through the inlet. It can also draw floating debris into the conduit. Thus, it is necessary to prevent formation of vortices at intakes and air entrainment.

Vortices are formed due to the following factors:

- i) Hydraulic jump formation
- ii) Velocities at intakes
- iii) Submergence at intakes
- iv) Geometry of approaching flow at intakes

Designing for a low velocity into the conduit and increasing submergence of the inlet can help prevent the formation of vortices. Flow approaching the intake asymmetrically is more prone to vortex formation than symmetrical flow. It is therefore important that flows upstream of the inlet area be as straight and uniform as possible.

For the condition of no vortices at intakes (after J. B. Gardon),

$$Y_s > 0.545 V \sqrt{D} \quad \text{for symmetrical approach}$$

$$Y_s > 0.725 V \sqrt{D} \quad \text{for asymmetrical approach}$$

Where, Y_s is necessary submergence depth, V is

Velocity at inlet to the canal, and D is diameter of the conduit.

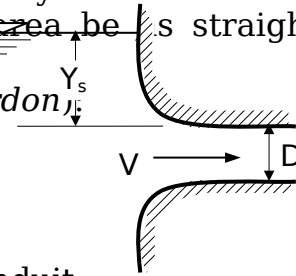


Figure 5.10: Sketch of submergence depth arrangement

Another remedy to vortex formation is provision of a floating raft or baffle which disrupts the angular momentum of the water near the surface.

3.1.6 Inlet Aeration

Intakes normally have a bulk head gate at the front and a control gate inside on the downstream side. An air vent is always provided just downstream of a control gate. The functions are:

- i) To nullify vacuum effect, this could be created when the penstock is drained after control gate closure.
- ii) Intake gates operate under conditions of balanced pressure on both sides of the gate. Thus the conduit is required to be filled with water through a by-pass pipe. The entrapped air is there for driven out through the air vent.

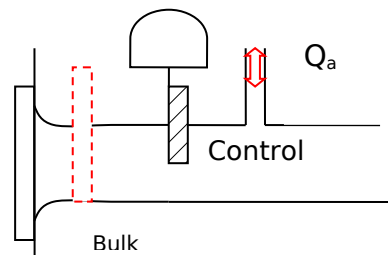


Figure 3.11: Air Vent Arrangement

Size of the air vent: There are several recommendations

$$1. \quad Q_a = 400 C a (p)^{1/2} \quad \text{-----} \quad (5.9)$$

Where Q_a = Discharge of air in cumecs a = Area of vent pipe in m^2
 C = Constant ~ 7

p = Pressure difference between the atmosphere and pressure in the penstock in kg/cm^2

2. 4th Congress on Large Dams (ICOLD)

Area of air vent = 10% of control gate area

3. USBR design guide: Capacity of air vent = 25% of conduit discharge

3.2 Head Race

Head race may be a power canal, a pressure tunnel, or a pipe, which in most cases conveying water from intake structure to surge tank, forebay or pressure shaft, depending on the arrangement of the scheme.

3.2.1 Canals

Canals are appropriate choice when the general topography of the terrain is moderate with gentle slopes. However, when the ground is very steep and

rugged, it becomes uneconomical to construct canals as it follows longer distances and/or needs provision of cross-drainage works and deep cuts and fills at a number of appropriate locations. In such cases, it is advisable to go for tunnels or pipes. The choice, in fact, has to be made based on economic analysis. Where the topography of the region presents special formations, the alternating use of open-canal and open-surface tunnel sections may ensure the most economical development.

The tracing of a power canal should closely follow the contour lines of the terrain. Based on the topographic conditions, a canal may be constructed through cuts, over fills, and in cuts-and-fills as shown in Figure below.

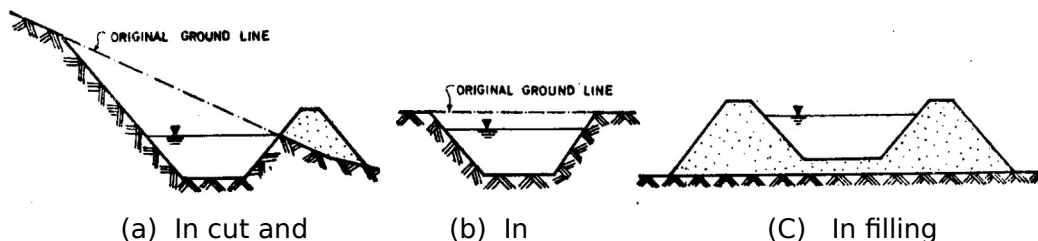


Figure 3-16: Canal Sections

A very important feature of an open canal is its vulnerability to damage from such sources as landslide and rock falls, and from storm water runoff crossing its path. The cost of protection from these eventualities and their associated repair costs must be included in an estimation of the canal cost.

Canal Lining

Power canals may be lined or unlined. The lined canals are usually lined with impervious material such as concrete, masonry, or clay.

Canal lining might be carried out to:

- Reduce seepage losses
- Reduce canal surface roughness
- Prevent the growth of weeds
- Reduce damage caused by erosion, rodents, and livestock
- Reduce the required volume of excavation
- Permits the use of rectangular x-section

Concrete Lining:

This is the most extensively used material in power canal lining. It is mostly used with trapezoidal cross sections. The excavation of a canal for a thin, unreinforced concrete lining is similar to that of an earth canal. It should be done carefully to ensure that the lining conforms closely to the desired profile when it is completed; otherwise, the lining will require more material than expected. The foundation should be adequately compacted and moistened before the concrete is placed. The necessity of moistening the foundation is to prevent the sub-grade from absorbing moisture from the freshly laid concrete making it weak and porous.

Concrete sections of a lining can be hand-formed at the site. If the side slopes exceed 1 in 1, form works may be necessary to hold the concrete in place until it sets. The thickness of the concrete lining may range from 10 to 20 cm, depending on the quality of the concrete and the soil conditions.

In preparing concrete for lining a canal, it is important to use the minimum amount of water needed for workability. Excess water will cause the concrete to slump and not stay on the canal side slopes. A mixture of a 1:3:4.5 (cement : sand : gravel) volume proportion is adequate for lining a power canal.

Brick or Stone Masonry:

This is another most frequently used type of power canal lining. Use of stone masonry permits savings by reducing the quantity of cement required as compared to concrete lining. Masonry lining consumes only about 25 - 30 % of the amount of cement required for poured concrete. Whether brick or stone is used depends on their relative availability and cost.

With masonry linings, a rectangular canal section is often used. In this case, the sides of the canals are constructed as retaining walls to counteract the lateral forces of either the earth backfill or the water within the canal.

In the construction of a power canal lined with masonry, the excavation must allow for the thickness of the lining. Before the lining is placed, the canal bottom and sides should be properly compacted to avoid future settling and cracking of the lining. They should also be wetted slightly to prevent the mortar from drying too fast.

To reduce resistance to flow and possible seepage, the masonry surface shall be plastered.

Note:

- In addition to the commonly used lining materials discussed above, other materials such as bituminous mixtures, soil-cement, wood, clay, chemical sealant, shotcrete, and impermeable membranes are also sometimes used to line power canals
- In special circumstances where the canal alignment is through a terrain having seasonally high water table or where the soils are not freely draining, *under-drainage* should be provided in order to protect the lining from damage due to uplift pressures.
- Due to temperature variations and shrinkage, cracks may be developed in canal linings and may result in appreciable leakage from the canal. In order to minimize these effects, it is necessary to provide *contraction joints* in the lining at suitable intervals (usually 3 to 8 m).

Canal Design

Canal design involves determination of the following:

- Carrying capacity, velocity of water in the canal & roughness coefficient of the canal surface
- Canal slopes
- Cross-sectional profile of the canal

i) Carrying Capacity and Velocity

For the hydraulic design, the discharge is computed from continuity equation as $Q=V.A$. The mean velocity, V , is determined from any of the continuity equations. The roughness coefficient is specified from the bed material type.

A Chezy's equation

$$V=C\sqrt{RS}$$

Several equations are available to determine the value of C .

1. Bazin's formula
$$C = \frac{87}{1 + \frac{m}{\sqrt{R}}}$$
 Where m is roughness factor
2. Maning's Formula
$$C = \frac{1}{n} R^{1/6}$$
 Where n is Maning's roughness coefficient
- The Chezy-Maning equation
$$V = \frac{1}{n} R^{2/3} S^{1/2} = MR^{2/3} S^{1/2}$$
 Use Tables for M & n.
3. Kutter Formula
$$C = \frac{41.65 + \frac{0.00281}{S} + \frac{1.811}{n}}{1 + \frac{n}{\sqrt{R}} \left(41.65 + \frac{0.00281}{S} \right)}$$
 in English units
4. The Agroskin formula
$$C = 17.72 (K + \log R)$$
- When using Maning's n,
 - add 0.001 to the values of the Table if the water carries small amount of silt
 - add 0.002 if the bed load is heavy
- B) Forch-Heimer Formula

$$V = CR^{0.7} S^{0.5}, C = \frac{1}{n}$$

Some designers prefer this formula for V and given by:

Apart from the hydraulic computations, the flow velocities in the canal or other water conduits in general are determined according to *economic point of views* (investments, head losses, wear and tear of material, danger of erosion and silting). The velocity must be high enough to prevent sedimentation. It has to be low enough to prevent bed erosion for unlined- and wear by abrasion for lined-canals.

Lowering the velocity keeps the head loss over the length of the canal to a minimum; however, it increases the cost necessary to construct the canal as the cross-sectional area increases when the velocity lowers. Table 3.1 provides permissible velocity ranges.

Maximum velocity		Minimum velocity
Bed Material	$V_{\max}$ (m/s)	$V_{\min}$ (m/s)
Sand	0.4	To keep any sediment from settling out, the minimum velocity in a canal should not be less than 0.3 m/s.
Sandy loam	0.6	
Loam	0.6	
Clayey loam	0.8	
Clay	2.0	
Gravel	3.0	
Masonry	3.5	
Asphalt	4.0	
Concrete	5.0	

Table 3.1 Permissible flow velocities in a canal

As stated above, in unlined canals flow velocities are limited by the resistance of the bed material to erosion. In unlined canals flow velocities are limited by resistance against wear.

Maximum Velocities

Critical bottom velocity (w.r.t. erosion) is given by:

Strenberg: $V_b = \zeta \sqrt{2d}$ for 'd' is particle size in meters, $\zeta = 4.43$

Maximum permissible mean velocity according to Bogardi and Yen is given by:

$$V = 22.9 d_m^{4/9} \sqrt{S_s - 1}$$

Where d_m is mean particle size and S_s is specific gravity of particles.

Minimum Velocities

There are various recommendations for non-silting velocity

- 1) According to Ludin

If $V_{\min} > 0.3$ m/sec, there will be no silting (for silty sediments)

$V_{\min} > 0.3$ to 0.5 m/sec, there will be no silting (for sandy sediments)

- 2) According to R.C. Kennedy

Non-scouring and non-silting velocity is given by: $V = Ch^{0.64}$

Where h is depth of water in meters and C is coefficient varying from 0.54 to 0.7, depending on silt load.

- 3) According to E.A. Zamarian, the requirements for silting or non-silting of unlined canal is given by:

$$G_0 = 700 \frac{V}{\omega_0} \sqrt{\frac{RSV}{\omega}}$$

Where G_0 = Silt load carrying capacity of canal in Kg/m³

V = mean velocity (m/s)

$\omega_0 = \omega$ if $\omega > 2$ mm/sec

= 2 if $\omega \leq 2$ mm/sec

ω = settling velocity in silt water (mm/sec)

R = hydraulic radius

S = bed slope

$$G_0 = \frac{900RS}{\omega_0 - 1.25}$$

For approximate values:

Find the actual G (sediment load) and compare it with G_0 . If $G > G_0$ there will be deposition. If $G < G_0$, there will be no deposition. The formulae are valid for:

1) Canal free from aquatic growth

2) Q is between 0.2 and 150 m³/s

3) $V > 0.3$ m/s

4) $\omega < 10$ mm/sec

- 1) According to M.M. Grishing, approximate $V_{\min}$ is given by: $V_{\min} = AQ^{0.2}$

Where A is coefficient, which is f (ω) and Q is design discharge.

For ω (mm/sec)	< 1.5	1.5 - 3.5	> 3.5
A	0.33	0.44	0.55

Table 3.2: Coefficients of velocity

ii) Roughness coefficient

As water flows in a canal, it losses energy in the process of sliding, path the walls, and bed material; the rougher the material, the more frictional loss and the greater the head drop or slope needed for a given velocity. The roughness coefficient, n , for various canal materials are given in Table 5.3 below

iii) Power Canal Slopes

In plain areas use slope between 5 to 20 cm/km (0.005 to 0.02 %); in mountainous areas slopes are as steep as 1 to 2 m/km. The canal bed slope can also be estimated using the Manning's equation:

$$S = \frac{n^2 V^2}{R^{4/3}}$$

The slope found from the above equation should nearly coincide with the available natural topography. Otherwise, a different slope should be computed by choosing other values for the velocity within the permissible limit until a satisfactory result is obtained.

Canal material		Roughness coefficient
Earth canals	Clay, with stones and sand	0.020
	Gravelly or sandy loam	0.030
	Lined with coarse stones	0.040
Rock canals	Medium coarse rock muck	0.037
	Rock muck from careful blasting	0.045
	Very coarse rock muck	0.060
Masonry canals	Brickwork, well pointed	0.015
	Normal masonry	0.017
	Coarse rubble masonry	0.020
Concrete canals	Smooth cement finish	0.010
	Concrete, unplastered	0.015
	Coarse concrete	0.018
	Irregular concrete surfaces	0.020
Wooden canals	Planed, well jointed boards	0.011
	Unplanned boards	0.012
	Older wooden canals	0.015

Table 3.3 Values of roughness coefficient 'n' for different canal materials

iv) Cross-sectional Profile

The material in which the canal is constructed generally dictates its cross-sectional profile. The common cross-sections used for canals are described in the following paragraphs.

A **semi-circular** cross-section is the most efficient profile because, for a given canal slope and cross-sectional area, it conveys the maximum flow. However, this form is impractical to excavate. It is therefore used primarily with materials which lend themselves to this shape. Examples are prefabricated concrete, sheet metal, and wood-stave sections.

A **trapezoidal** cross-section is the most widely used profile for both lined and unlined canals excavated in earth. If the canal is unlined, the maximum side slope is set by that slope at which the material will permanently stand under

water. The magnitude of the side slope of a lined trapezoidal canal depends on the nature of the material on which the lining will rest, but usually steeper than unlined canals. In general, it should be nearly equal to the angle of repose of the natural soil so that no earth pressure is exerted on the back of the lining. The banks of a lined canal resting on almost any free-draining material require slopes not steeper than 1:1.

For a trapezoidal canal with a given side slope, the most efficient cross-section is one in which a semi-circle can be inscribed in the wetted area. For this section, it can be shown that the length of either sloping side of the wetted area is half its top width.

Canal material	Side slope (V:H)
Firm rock	1:1/4
Fissured and disintegrated rock	1:1/2
Clay	1:3/4
Clayey loam	1:1
Loam	1:3/2
Sandy loam	1:2
Sand	1:3
Lining	1:1

Table 3.4 Suggested side slopes for trapezoidal canals

A **rectangular** cross-section is often most appropriate when excavation is undertaken in firm rock. It is also commonly used when the canal incorporates properly constructed masonry walls. Use of a rectangular canal reduces the excavation required. For the most efficient rectangular cross-section, the width of the canal is twice the depth of the wetted area and, like a trapezoidal section, is a section in which a semi-circle can be inscribed.

Freeboard Allowance:

Freeboard is provided above the design water level for safety purposes. For earth canal the lower limit is 35 cm and the upper limit is 140 cm. Generally the free board = $[0.35 + 1/4h]$ m. Where h is depth of flow. Allowances should be made for bank settlements. For lined canals, the top of the lining is not usually extended for the full height of the free board. Usually it is extended to 15cm to 70cm above the design water level.

Water Loss in Power Canals

Water losses are due to

- seepage
- evaporation
- leakage at gates

Generally b) and c) are generally of minor importance. Seepage losses from earth canal may be described according to the following procedures:

1) Davis and Wilson

$$q = \frac{C}{10,000} P^3 \sqrt{h} \quad (q \text{ in m}^3/\text{sec-km})$$

Where C = coefficient depending on soil type and lining (see Table 5.5 below)

P = wetted perimeter of canal in m

H = depth of water in m

Canal	Lining or soil type	C
Lined	Concrete lining 75 to 100mm thick	1
	Clay lining, compacted, 150 mm thick	4
	Light bituminous lining	5
	Clay lining, compacted 75 mm thick	8
	Thin lining of asphalt or cement mortar	10
Unlined	In loam	12
	In loamy silt soil	15
	In silty soil	20
	In sandy silt soil	25
	In silty sand soil	30
	In fine sand	40
	In sands of medium fineness	50
	In sandy gravel	70

Table 3.5: C parameter for seepage computation

2) B.A. Etchiverry gives specific seepage coefficient based on measurements on American irrigation canals. Higher values apply for newly constructed channels.

Material	Specific seepage (m ³ /day-m ²)
Slightly pervious loam	0.08 - 0.11
Loam of moderate permeability	0.11 - 0.15
Pervious loam or silt	0.15 - 0.23
Gravelly or sandy loam	0.23 - 0.30
Loose sand	0.46 - 0.53
Gravelly sand	0.61 - 0.76
Gravel	0.76 - 0.91
Coarse gravel	0.91 - 1.83

Table 3.6: Specific seepage of canals

3) A.N. Kostyakov (USSR):
Values of C

$$q = \frac{C}{100} Q \quad q \text{ in m}^3/\text{sec-km}$$

a) for soils of high permeability,

$$C = \frac{3.4}{Q^{0.5}} \%$$

b) for soils of moderate permeability,

$$C = \frac{1.9}{Q^{0.4}} \%$$

c) for soils of low permeability,

$$C = \frac{0.7}{Q^{0.3}} \%$$

Exercises

- Design a settling basin for high-head power station using the simple settling theory. The basin should serve to remove particles greater than 0.5mm diameter from the water in which the sediment is mainly sand. Let the design discharge be 5m³/s and assume an initial value of 3.2m for the basin depth. Take the water sediment mixture density to be $\rho = 1.064$
- A power canal with a slope of 0.0001044 and R=1.76 m is unlined with coarse sand bed material. Determine the safe velocity in the canal if the heaviest bed load in the original canal of the course is G=0.34kg/m³. The water carries silt with

an average diameter of 0.08 mm. Assume $d_m = 2.00$ mm as size of bed material and $h = 2.5$ m.

3. Determine the seepage loss from a power canal constructed in a sandy soil of medium fineness by the Davis and Wilson, Etchevery and Kostyakov methods discussed. The following data are given:

Trapezoidal canal: bottom width, $b = 10.0$ m
Water depth, $D = 2.5$ m
Side slope 2H: 1V
Mean velocity $= 0.5$ m/s
Effective size of soil particle, $d_m = 0.1$ mm
Permeability coefficient, $k = 1 \times 10^{-5}$ m/s

3.3 Tunnels

3.3.1 General

Tunnels are underground conveyance structures constructed by special tunneling methods without disturbing the natural surface of the ground. In many modern high head plants, tunnels form an important engineering feature.

In the headrace of water conveyance system, tunneling is popular because of the following reasons:

- i) It provides a direct and short route for the water passage thus resulting in considerable saving in cost
- ii) Tunneling work can be started simultaneously at many points thus leading to quicker completion
- iii) Natural land scape is not disturbed
- iv) Tunneling work has become easier with development techniques of drilling and blasting and new mechanical equipment
- v) Development of rock mechanics and experimental stress analysis has given greater confidence to engineers regarding stability of tunnels.

Tunnels of hydropower projects fall into two categories: *water carrying tunnels* and *service tunnels*.

- a) **Water carrying tunnels** : These include head race or power tunnels, tail race tunnels or diversion tunnels. Flows in water tunnels are usually under pressure (pipe flow), but sometimes free-flow (open channel flow) can be experienced, especially, in tailrace tunnels. The design of free-flow tunnels follow the same principles as used in the design of open canals.
 - *Head race tunnels*: are tunnels that convey water to the surge tank. These are pressure tunnels
 - *Tail race tunnels*: could be free flowing or pressure tunnels depending on the relative position of turbine setting and tail water level.
 - *Diversion tunnels*: are constructed for the purpose of diverting the stream flow during construction period. Normally they are not of high pressure but should have sufficient flood carrying capacity. Such tunnels either plugged with concrete or converted in to some use such as spillway tunnel at the completion of the project.
- b) **Service tunnels**: These may be:
 - *Cable tunnels*: to carry cables from underground power house to the switch yard
 - *Ventilation tunnels*: fitted with fans at the open end to supply fresh air to the underground
 - *Access or approach tunnels*: this is a passage tunnel from surface to underground power house.

3.3.2 Classification of Tunnels

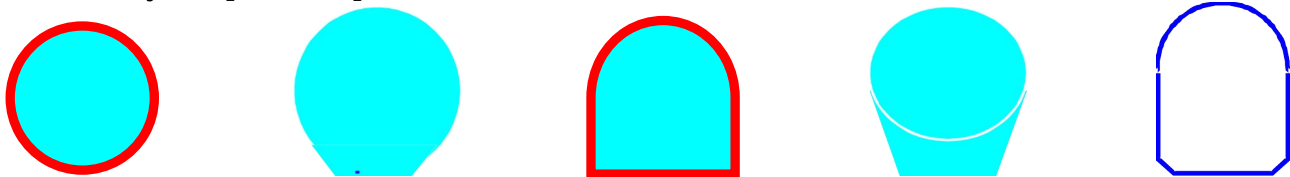
In addition to the above classification tunnels may be classified on the basis of shape, alignment and design aspects.

Shape: Tunnels are either circular or non-circular in shape.

Circular tunnels: are most suitable structurally. They are more stable when the internal pressure is very high.

Non-circular tunnels: have a flat floor, nearly vertical or gently flaring walls and an arching roofs. The horse-shoe shape is the most popular and convenient from the point of view of construction.

Commonly adopted shapes:



a) Circular shape

c) D-shape

b) Horseshoe shape

Figure 3.15: Tunnel shapes

Alignment: A name tunnel indicates a very small bottom slopes, i.e. tunnels are aligned nearly horizontal. Shaft is a tunnel with vertical alignment or inclined shaft when it is steeply inclined to the horizontal. It is very crucial to investigate in detail the geology of the strata through which a tunnel would be passing. Sound, homogenous, isotropic, and solid rock formations are the most ideal ones for tunneling work. However, such ideal conditions are rarely present, and rather the rock mass exhibits various peculiarities. There may be folds, faults, joint planes dipping in a particular direction, weak strata alternating with good strata, etc. Thus, the alignment of a tunnel should be fixed keeping in view these phenomena. The alignment, for instance, should as far as possible avoid major fracture planes.

Design Aspects: Aspects of lining, pressure condition, etc., can be considered to identify different types of tunnels.

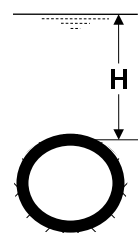
Lining: Lining is a protective layer of concrete, R.C. or steel on the inner surface of the tunnel and it is an important aspect in classification of tunnels. Thus tunnels may be lined, unlined or partially lined. Tunnels in good, sound rock may left unlined.

Lining of tunnels is required:

- i) For structural reasons to resist external forces particularly when the tunnel is empty and when the strata is of very low strength.
- ii) When the internal pressure is high, i.e. above 100m
- iii) When reduction in frictional resistance and therefore the head loss is required for increasing capacity
- iv) For prevention or reduction of seepage losses
- v) For protection of rock against aggressive water

Pressure tunnels: are classified according to pressure head above the soffit of the tunnel. Accordingly:

- Low-pressure tunnels ($H < 10$ m)
- Medium pressure tunnels ($10 \text{ m} < H < 100$ m)
- High-pressure tunnels ($H > 100$ m)



In the case of low-pressure tunnels the tunnel surface may frequently be left unlined except for visible fissures. A watertight lining is usually required for tunnels operating under medium and high heads. Seepage is more likely to occur as the head increases, water may leak through the smallest fissures and cracks. Moreover, under high-pressure it may penetrate the otherwise watertight rock and render it permeable.

Low Head Tunnels

- The trimmed rock surface may be sufficient by only sealing visible fissure with concrete or cement mortar or granite layer
- Full lining may be warranted only if external rock load or aggressivity or water head loss reduction justify it.

Medium head Tunnels

- A water tight lining concrete is almost always needed since seepage is more likely to occur under increasing head.
- If the lining is only for water sealing purposes, and no load is carried by it, the permissible internal water pressure head is determined by the depth of overburden and the quality of the rock.

Let h_r = depth of overburden rock
 γ_r = specific weight of the rock
 γ_w = specific weight of water.
 H = Internal pressure head of water.

Then for equilibrium: $\gamma_w H \leq \gamma_r h_r$

With $\gamma_w = 1 \text{ ton/m}^3$, we have $H \leq \gamma_r h_r$

$$H = \frac{\gamma_r h_r}{\eta} \text{ (m)}$$

Using a factor of safety of η ,

Recommended factor of safety $\eta = 4$ to 6 .

With $\gamma_r = 2.4 \text{ t/m}^3$ to 3.2 t/m^3 and using lower η values for good quality rock, one gets $H = (0.4 \text{ to } 0.8) h_r$

High Head Pressure Tunnels

- Usually steel lining is used (R.C. Concrete lining not satisfactory)
- The steel lining is embedded in concrete filling the annular space b/n the steel lining & the rock. In order to provide proper contact b/n rock and concrete and b/n steel lining & concrete, all voids are filled by grouting with cement mortar.
- The profile of the Pressure tunnel should be such that the roof should always be at least 1 to 2m below the hydraulic grade line
- Saddles should be provided with dewatering provisions and summits should be provided with outlets or shafts.
- To reduce construction costs, relatively high velocities (higher than in open channels) are permitted in tunnels.

The following velocities are suggested:

Very rough rock surface----- 1 to 2.0 m/s

Trimmed rock surface ----- 1.5 to 3.0 m/s

Concrete surface----- 2 to 4.0 m/s

Steel lining----- 2.5 to 7 m/s

The permissible velocity depends upon the sediment load carried by the water. The maximum values in the above recommendation apply when the sediment is of the silt fraction. For water carrying sharp edged sand in significant quantity, $V_{\max} = 2$ to 2.5 m/s even in lined section.

- Size of tunnels cannot be reduced arbitrarily. Requirements of possibility limit the maximum size.

Minimum size of Tunnel: Circular, 1.8 m ϕ

Rectangular, 2m x 1.6m.

3.3.3 Tunnel Design Features

In addition to the general discussion in above, as design features *alignment, geometric shape, longitudinal slope, flow velocity, head loss, rock cover (overburden), lining requirements (also coupled with stress analysis), and economic x-section* come in to play.

Alignment:

In aligning water tunnels, the following points should be taken in to account:

- Length of the tunnel: as much as possible short route should be followed
- Location of surge tanks & adits: the alignment should provide convenient points for surge tanks & adits.
- Rock cover (overburden): sufficient rock cover should be available along the alignment
- Discontinuities: the alignment should, if possible, avoid crossing of weakness zones, joint planes, etc. If crossing of these features is unavoidable, suitable direction of crossing should be considered.
- Rock quality: good quality of rock mass should be sought in aligning the tunnel

Geometrical Shape:

- The choice of the cross-sectional profile of a tunnel depends on:
- Hydraulic considerations - *Circular is preferable*
- Stability considerations - *Circular is preferable*
- Convenience for construction - *Horseshoe is preferable*
- Available tunneling equipment - *If drilling rigs are available, a horseshoe cross-section has to be adopted.*

Longitudinal Slope:

The minimum slope for a pressure tunnel is limited on the basis of dewatering requirements. And also the longitudinal profile of the tunnel should be such that the roof remains below the hydraulic pressure line by 1 to 2 m. Likewise, the tunneling method and the equipment employed for transportation of the excavated material (rail or wheel transport) can limit the maximum slope possible to provide. The usual practice is to keep the slope of power tunnel gentle till the surge tank and then steeper (even vertical) for the pressure shaft.

Flow Velocity:

The allowable velocities in tunnels depend upon whether it is lined or unlined. In unlined tunnels, a velocity of 2 to 2.5 m/s is the upper limit, while in concrete lined tunnels 4 to 5 m/s is often employed. The velocities for the pressure shafts, which are generally steel

lined, are usually higher than that in the power tunnel. The normal range of velocities is between 5 to 8 m/s.

Rock Cover (overburden):

For pressure tunnels, it is obvious that the overburden on the roof of the tunnel serves to balance the effect of upward force due to internal pressure. The required depth of overburden may vary for lined and unlined tunnels.

In the case of **unlined tunnels**, the entire internal water pressure is resisted by the overburden rock pressure. Where a steep valley side constitutes the overburden above the tunnel, the rule of thumb equation, $H=(0.4 \text{ to } 0.8)h_r$ has to be modified and given by:

$$h_w = \frac{1}{\eta} \frac{\gamma_r}{\gamma_w} L \cos \beta$$

Where L is the shortest distance between the ground surface and the studied point of the tunnel (or shaft) and β is the average inclination of the valley side with the horizontal (see figure below).

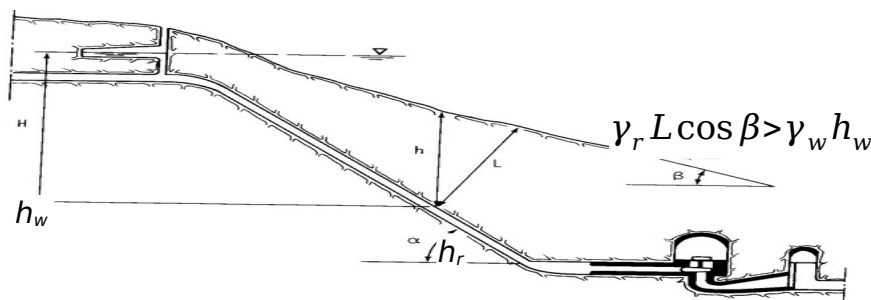


Figure 3.16 Overburden requirement in a steep valley side

In the case of concrete or steel **lined tunnels**, the linings are usually designed to carry part or full load of the internal water pressure, and the above equations, given for unlined tunnels, are modified accordingly in determining the required overburden depth. For detail see section on stress analysis

Head Loss:

Head losses in tunnels can be computed using Manning's, Darcy-Weisbach, or Hazen-Williams formulas.

Manning formula:
$$h_f = n^2 \frac{l v^2}{R^{4/3}}$$

Darcy-Weisbach formula:
$$h_f = \lambda \frac{l v^2}{2g D_{eq}}$$

Hazen-Williams formula (rarely used):
$$h_f = 6.84 \frac{l v^{1.85}}{C^{1.85} D_{eq}^{1.17}}$$

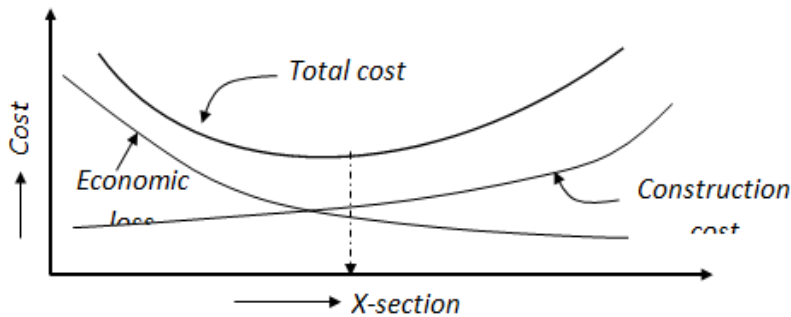


Figure 3.17 optimum tunnel cross-sections

Where, h_f is head loss due to friction, L is tunnel length, V is mean velocity of flow, R is hydraulic radius, D_{eq} is equivalent diameter ($D_{eq} = \sqrt{4A/\pi}$), A is area of the tunnel x-section, n is Manning's roughness coefficient, λ is Darcy-Weisbach friction factor (can be obtained from Moody diagram), and C is Hazen-Williams roughness coefficient.

Optimum X-section:

The optimum x-section of a tunnel or a shaft is one for which the sum of tunnel construction cost and the economic loss due to head loss is minimum.

For a quick initial estimate of the diameter of pressure tunnels, the empirical formula suggested by *Fahlbusch* can be used:

For concrete-lined tunnels: $D = 0.62 Q^{0.48}$

For steel-lined tunnels: $D = 1.12 \frac{Q^{0.45}}{H^{0.12}}$

3.3.4 Stress Analysis and Lining

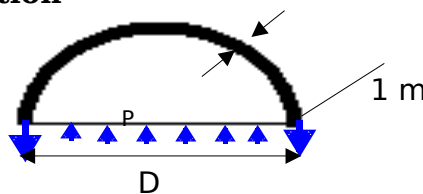
1 Lining designed to resist the *full internal pressure without support* from the surrounding rock i.e. no load sharing. Internal water pressure, $P = \gamma_w H$

If $\gamma_w = \text{t/m}^3$ & H in m

Then $P = 0.1 H$ kg/cm² for circular lining, if $\delta = \text{lining thickness (cm)}$

$D = \text{the internal diameter (cm)}$

Figure 3.18: Tunnel Section



Then the maximum stress developed is the tangential stress

Then for equilibrium $2 \sigma \delta 1 = P D 1$

or preferably $2 \sigma \delta \geq p \cdot D \quad \therefore \delta \geq \frac{P \cdot D}{2 \sigma}$

Then if σ_a is allowable stress in lining, then $\delta \geq \frac{PD}{2 \sigma_a}$

For R.C. lining

i) $2 \sigma_{ast} A_{st} = P \cdot D \quad \therefore A_{st} \geq \frac{PD}{2 \sigma_{ast}} \times 100$ Where A_{st} in cm²/m

ii) If no reinforcement, since the tensile stress in homogenous concrete is usually limited to 10 kg/cm^2 , then $\frac{PD}{2\delta} \leq 10 \text{ kg/cm}^2$ Where δ = thickness of concrete lining in cm.

II Lining designed assuming load sharing between rock & lining

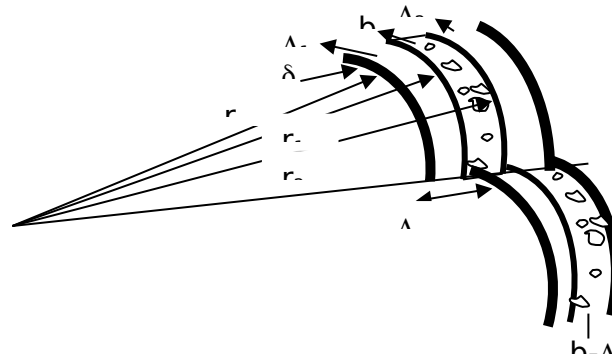


Figure 3.19: Schematic sketch of lining and displacement

This is a more realistic approach. Referring to the following schematic sketch

Δ_{st} = Radial displacement of steel

If P_1 = Load transferred to the rock, then $P - P_1$ = load carried by the steel lining

Tangential stress in an annular steel lining of radius r and thickness δ under load $(P - P_1)$ will be:

$$\sigma_{st} = \frac{(P - P_1)D}{2\delta}$$

D is internal diameter

$$\text{Unit radial strain} = \frac{2\pi \Delta_{st}}{2\pi r} = \frac{\Delta_{st}}{r} \quad \therefore E_{st} = \frac{\sigma_{st}}{\Delta_{st}/r}$$

$$\sigma_{st} = E_{st} \frac{\Delta_{st}}{r} = \frac{(P - P_1)D}{2\delta} \Rightarrow \Delta_{st} = \frac{(P - P_1)}{E_{st}} \frac{r^2}{\delta}$$

The radially fissured annular concrete layer is subjected to an internal pressure P_2

$$P_2 = P_1 \frac{r_1}{r_2}$$

Theoretical investigations & experiments show that

$$\Delta_c = \frac{P_1}{E_c} r_1 l_n \left(\frac{r_2}{r_1} \right)$$

The radius of the cross-section excavated in rock expands under load P_2 by an amount Δ_r

$$\Delta_r = \frac{1}{E_r} \left(\frac{m+1}{m} \right) r_2 P_2 = \frac{1}{E_r} \left(\frac{m+1}{m} \right) r_1 P_1$$

E_r = modulus of elasticity of the rock

$$\mu = \frac{1}{\nu}, \text{ where } \mu = \text{Poisson ratio of the rock}$$

Since

$$\begin{aligned} \delta + \Delta_1 + b + \Delta_2 + \Delta_r &= \Delta_{st} + \delta + b - \Delta_c \\ \Delta_1 + \Delta_2 + \Delta_r + \Delta_c &= \Delta_{st} \\ \Delta_1 + \Delta_2 + \frac{1}{E_r} \left(\frac{m+1}{m} \right) r_1 p_1 + \frac{P_1}{E_c} r_1 l_n \left(\frac{r_2}{r_1} \right) &= \left(\frac{p - p_1}{E_{st}} \right) \frac{r^2}{\delta} \end{aligned}$$

$P_1 = \eta P$, where η = load distribution factor

P_1 = Load carried by rock.

Now, assuming $r \ll r_1$,

$$\Delta_1 + \Delta_2 + \epsilon P \left[\frac{r}{E_c} \ln \left(\frac{r_2}{r_1} \right) + \frac{r}{E_r} \left(\frac{m+1}{m} \right) \right] = \frac{Pr^2}{E_{st} \delta} - \frac{\epsilon Pr^2}{E_{st} \delta}$$

Simplifying & Rearranging:

$$\epsilon = \frac{1 - \frac{E_{st}}{P} \frac{\delta}{r} \left(\frac{\Delta_1 + \Delta_2}{r} \right)}{1 + \frac{E_{st}}{E_c} \frac{\delta}{r} \ln \left(\frac{r_2}{r_1} \right) + \frac{E_{st}}{E_r} \frac{\delta}{r} \left(\frac{m+1}{m} \right)}$$

Checking purposes

This equation is useful for checking purposes for a given δ . For designing by trial & error, δ is assumed & η is computed.

Load carried by steel $P - P_1 = P - \eta P = (1 - \eta)P$

$$\sigma_{st} = \frac{(1 - \epsilon)PD}{2\delta}$$

Compute σ_{st} must be $\leq \sigma_{sta}$

For direct design purposes we use the following

We had: $\Delta_1 + \Delta_2 + \Delta_c + \Delta_r = \Delta_{st}$

$$\Delta_1 + \Delta_2 + \epsilon P \left[\frac{r}{E_c} \ln \left(\frac{r_2}{r_1} \right) + \frac{r}{E_r} \left(\frac{m+1}{m} \right) \right] = \frac{\sigma_{st} r}{E_{st}}$$

rearranging simplifying and putting in σ_{sta} for σ_{st} ,

$$\epsilon = \frac{1}{P} \frac{\sigma_{sta} - E_{st} \left(\frac{\Delta_1 + \Delta_2}{r} \right)}{\frac{E_{st}}{E_c} \ln \left(\frac{r_2}{r_1} \right) + \frac{E_{st}}{E_r} \left(\frac{m+1}{m} \right)} \text{ then } \delta = \frac{(1 - \epsilon) Pr}{E_{sta}}$$

□ Varies from 0 to 1

If □ ≥ 1 - no lining is required

If □ ≤ 0 - entire load is carried by lining

Normal values : $E_{st} = 2.1 \times 10^6 \text{ kg/cm}^2$

$$E_c = 2.1 \times 10^5 \text{ kg/cm}^2 \quad \frac{E_{st}}{E_c} = 10$$

The value of E is insensitive to ν. Therefore an average value of m=6 is used

The value of $\Delta_1 + \Delta_2 = \Delta$ depends up on workmanship

For carefully executed work with grouting, $\Delta_1 + \Delta_2 = 0.5 \text{ to } 1 \text{ mm. (see examples)}$

3.3.5 Tunneling Methods

There are two commonly used types of tunneling techniques:

- Conventional “Drill and Blast”
- Use of tunnel boring machines (TBM)

Conventional “Drill and Blast” Method:

In this method of tunneling is used, the main equipment used is the so-called *drilling jumbo* or *drilling rig*, which performs the main operation. The following are the main sequences to be followed during excavation of each round:

- | | |
|------------------------|-------------------------|
| i) Drilling | ii) Charging |
| iii) Blasting | iv) Ventilating |
| vi) Scaling | vi) Mucking and hauling |
| vii) Tunnel supporting | |

Drilling is carried out by the percussion method. The principle is to force a drill rod with a suitable drill bit against the tunnel face, generate a stroke in order to obtain rock spalling, then rotate the rod before it is once again forced against the hole and a new stroke is generated. The power for the drilling thrust and rotation is transmitted by hydraulic systems.

In tunneling, blast hole diameters of 45 to 50 mm are most common. Medium size tunnels (about 40 m²) may require about 60 to 70 drill holes. The length of the drill holes usually varies from 3 to 5 m.

Once the drilling operation is completed, **charging** of the drill holes with explosives will be performed. There are different types of explosives, the most common one being dynamite.

If the holes close to the planned contour of the tunnel are too heavily loaded with explosives, a considerable “over-break” and a rough, uneven contour may result. This over-break greatly increases the need for scaling and tunnel support. If the tunnel is unlined, it will also greatly increase the head loss. In order to minimize the over-break in the walls and roof of the tunnel, reduced charges are used close to the contour.

In rock **blasting** the main principle is to break the rock and push the rock fragments towards a free surface. In a tunnel the degree of confinement of the blast volume is far higher than in a quarry. In order to obtain a satisfactory result from a tunnel blast it is, therefore, necessary to include the so-called “**cut**” in the blast hole design. These consist of holes of larger diameter than the blast holes and are usually left unloaded.

After blasting the round, **ventilation** has to be carried out to lower the concentration of blasting fumes to a satisfactory level. The fans are usually started just after the explosion. It is very seldom possible to enter the working face area until 15 minutes after the blast, but this depends on the ventilation capacity.

After each blast round, **scaling** (removing loose rocks from the roof and walls of the tunnel) is done for the sake of safety. For small tunnels the scaling is made directly from the muck pile, in larger it is often carried out from the wheel loader.

The selection of equipment for **mucking and hauling** largely depend on the cross-sectional area and the gradient of the tunnel. For tunnels with cross-sections smaller than 16 m², the only alternative for transportation is *rail transport* system. In larger tunnels, *wheel transport* system shall be used. For wheel transport in cross-sections between 16 and 30 m² “**niches**” are required every 100 to 150 m for the purposes of loading and turning trucks.

A major restriction for the rail transport alternative is that the maximum gradient has to be less than 2%. For wheel transport gradients up to 15% may be tolerated.

Use of Tunnel Boring Machines (TBM):

A tunnel boring machine (TBM) is a complex and very advanced piece of machinery designed to excavate the entire cross-section in a single operation without the use of explosives. Tunnels with diameters of about 1.8 m to more than 11 m have been excavated with tunnel boring machines.

TBM consists of a wheel cutter head fitted with teeth or rollers to cut or spall the rock. The wheel is slightly smaller than the bore of the tunnel and is equipped with disc-cutters to produce the designed bore. The wheel is forced against the tunnel face by hydraulic jacks and is made to rotate. As excavation proceeds, the rock-cuttings are picked up in buckets attached around the rim of the wheel and are discharged on to a conveyor belt incorporated with the machine as shown in Figure 5.20.

The diameter of the cutters is normally within the range of 45 to 50 cm, and the total number of cutters varies from 20 for smallest machines to more than 70 for the largest.

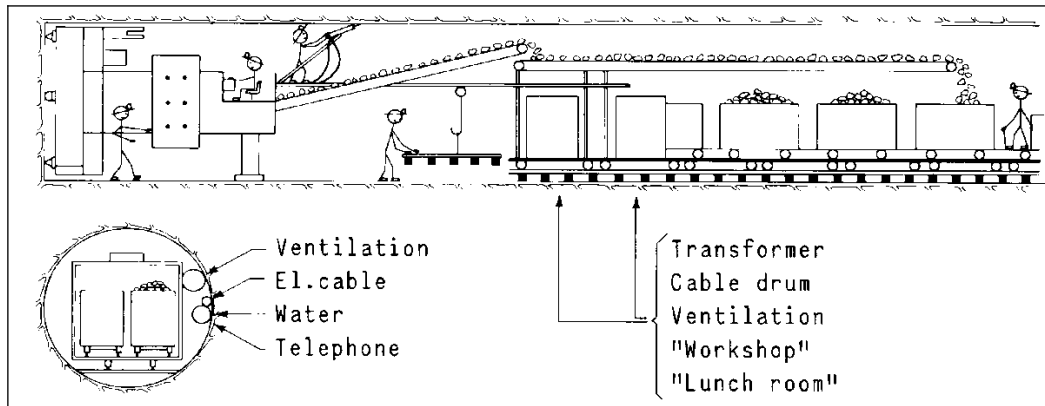


Figure 3.20: Typical sketch for TBM tunneling

Compared to conventional D & B tunneling technique, TBM tunneling has the following advantages:

- ♦ For long tunnels (> 3 km) the excavation time and the costs in many cases are considerably lower due to a higher advance rate combined with reduced requirement for tunnel support and ventilation.
- ♦ The tunnel will have a circular profile with a smooth contour, which is of particular importance in reducing head loss in unlined water tunnels.
- ♦ Potential problems due to blast vibrations are eliminated, especially in populated areas.
- ♦ Less number of “adits” is required.

There are, however, some limitations in connection with the use of TBM in tunneling. These include:

- ♦ Initial cost of the machine is high
- ♦ Requires detail geological investigation than the D & B alternative
- ♦ Less flexible than Conventional D & B technique in tackling stability problems.

Tunnel Supports

A basic philosophy in tunneling is that the extent of installed tunnel support should reflect the actual rock conditions. In good quality rock the *self-supporting* capacity of the rock mass should be used to its advantage, and the amount of tunnel support kept at a minimum. In poor quality rock the design of support should be based on a sound understanding of the character and extent of the stability problem.

The various geological factors which may influence the stability are:

- The degree of jointing and their character
- Weakness zones and faults
- Rock stresses
- Groundwater inflow

Tunnel support may be installed either at the working face (*immediate support*), or behind the face (*permanent support*). Whenever possible the design for the immediate support should be chosen which makes it possible to act later as permanent support.

The following support methods are the most commonly used in hydropower tunneling today:

- Rock bolting
- Concreting

- Grouting
- Concrete lining

Rock Bolting:

A rock bolt is a steel bar, which is inserted into a hole drilled in a rock to improve the rock competency. The distant end has a device which permits it to firmly anchored in the hole and the projecting end is fitted with a plate which bears against the rock surface (see Figure 5.21). The bolt is placed in tension between the anchor and the plate, thereby exerting a compressive force on the rock.

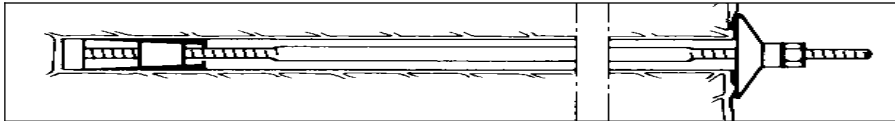


Figure 3.21: Principle of rock bolting

Rock bolting in tunnels is carried out according to one of the following two main principles:

- Spot bolting of individual, unstable blocks
- Systematic bolting of a section of the tunnel in a definite pattern

On average, the length of rock bolts in water tunnels is 1.5 to 4 m and the diameter is 16 to 25 mm. Rock bolting is usually used for an immediate support near the tunnel face.

Shotcreting:

A shotcrete is a quick-setting concrete plaster shot at rock surfaces pneumatically. It creates the best possible rock support condition, which makes it an economical, rapid, and effective means of providing tunnel support. In practice the shotcrete is placed in 5 cm layers until a desired thickness is attained.

The use of steel fibres in the concrete mix has an effect of increasing the strength of the shotcrete. For an immediate support in areas of heavily jointed rock masses or in areas of high rock stresses, steel fibre reinforced shotcrete is commonly used.

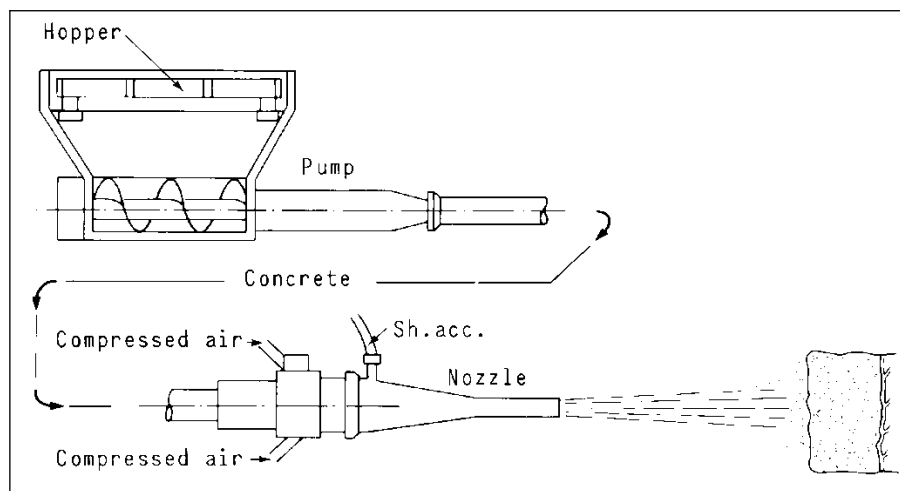


Figure 3.22: Principle of shotcreting

In many cases, the shotcrete is combined with rock bolting for use as a permanent support. A combination of steel fibre reinforced shotcreting and systematic rock bolting can replace concrete lining alternative, provided that water inflow and active gouge material in the discontinuities are minimal or absent.

A general restriction in the use of shotcrete is in areas with water leakage. The main restriction, however, is where weakness zones contain swelling clay (smectite). If shotcrete is applied on such zones, there will be no room for expansion of the swelling clay, and high swelling pressure will be activated when the zones are exposed to water. This may easily destroy the shotcrete lining.

Grouting:

A grout is a mixture of cement and water forced in to rocks around the tunnel periphery. Grouting may be performed ahead of the tunneling face (*pre-grouting*) or behind the tunneling face (*post-grouting*). Pre-grouting is necessary in areas where groundwater inflow makes tunnel driving difficult (*see Figure 5.23*). Probe holes are drilled ahead of the tunnel face to perform permeability testing before deciding the necessity of pre-grouting. Post-grouting is done to improve the stability of the rock mass behind the tunnel face.

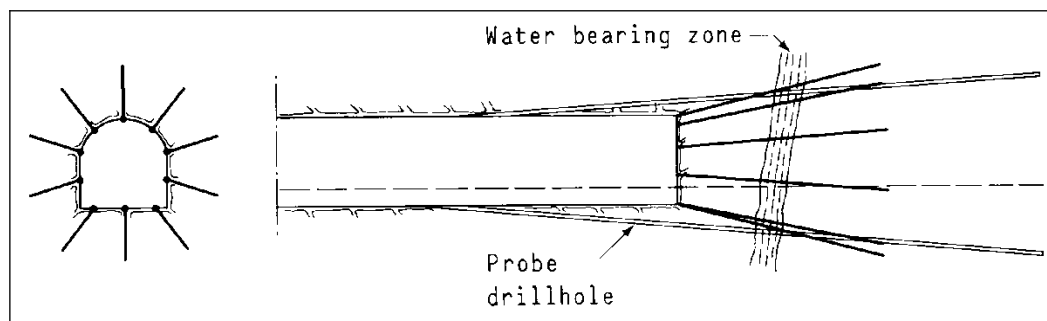


Figure 3.23: Principle of pre-grouting in a water-bearing zone

3.4 Water Hammer, Surge Tanks and Forebays

3.4.1 Water Hammer

A sudden change of flow rate in a large pipe line (due to valve closure) may involve a great mass of water moving within the pipe walls. The force resulting from changing the speed of the water mass causes a pressure rise in the pipe with a magnitude several times greater than the normal static pressure in the pipe. This phenomenon is commonly known as water Hammer because of the noise & vibration with which it is sometimes accompanied. The excessive pressures only fracture the pipe water unless it is properly analyzed & accounted for in the design of the pipe line.

The determination of Water Hammer pressure is amenable to mathematical analysis. Practical problems may, however, be of considerable complexity owing to the many variables involved. Here, we will consider only elementary class. In this regard the simplest procedure is to regard the water as incompressible & the pipe as rigid (The so-called rigid Water-Column theory).

The assumption is obviously not very realistic but can lead to reasonable estimations in the certain cases. The more realistic situation is the one that takes the elasticity of the water & the pipe in to account (The Elastic Water Column Theory)

A Rigid Water Column Theory

Assumptions: 1. Pipe is rigid
2. Fluid is rigid

Considering a pipe line of uniform cross sectional area A , length L , connected to a reservoir (or surge tank).

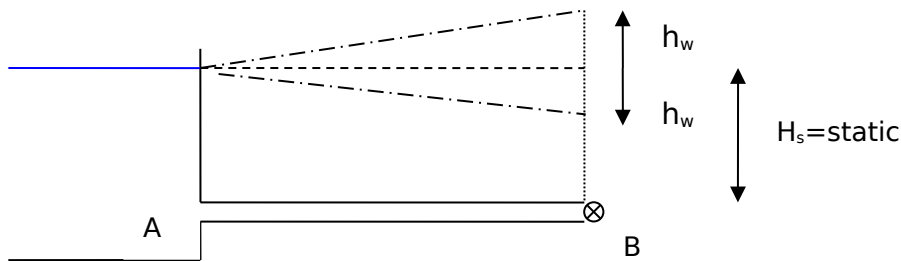


Figure 3.24: Schematic diagram of water hammer pressure in pipe

The pressure increase swings the normal HGL from CD to EF . Since the pressure the reservoir surface is atmospheric & hence constant the positive swing results in back flow from the pipe to reservoir. As the water flows back in to the reservoir it crates partial vacuum condition in the pipe & the pressure in the pipe swings in the negative direction. Thus, induces the reservoir water to flow back in to the pipe . But the value being partially closed, much of the water is again retarded giving rise to positive swing of pressure again. Thus, a valve closure brings about pressure oscillations. The maximum additional water hammer pressure head h_w can be determined from Newton's 2nd law.

Mass of water retarded $= \rho \cdot A \cdot L$

Pressure force at the valve $= \Delta P \cdot A$

$$\therefore \Delta p \cdot A = \rho \cdot A \cdot L \cdot \left(\frac{\Delta v}{\Delta t} \right)$$

$$h_w = \rho g = \rho L \left(-\frac{\Delta v}{\Delta t} \right) \quad h_w = -\frac{L}{g} \frac{\Delta v}{\Delta t}$$

pressure rise !

Thus the total head @ the valve $= H_s + h_w$

$$= H_s - \frac{L}{g} \frac{\Delta v}{\Delta t}$$

Considering friction and local loss, the total head becomes

$$= H_s - \left(\lambda \frac{L}{D} + k \right) \frac{v^2}{2g} - \frac{L}{g} \frac{\Delta v}{\Delta t}$$

If the retardation is known, the above equation can be solved.

Eg. If steady flow velocity V_0 is reduced to zero @ a uniform rate during t_c , then the max. pressure head @ the valve due to water hammer will be

$$h_w = -\frac{L}{g} \left(\frac{0 - v_0}{t_c} \right) = \frac{L v_0}{g t_c}$$

Now, Considering that the pressure wave is propagated from the valve to the reservoir with a speed c , the time required for the pressure wave to travel from B to A is L/c . In this time, the mass brought to rest is $\rho A L$. thus:

$$\Delta p \cdot A = -\rho A L \left(\frac{0 - v_0}{t/c} \right)$$

$$\therefore \Delta p_{\max} = \rho c v_0$$

$$\& \quad h_{w_{\max}} = \frac{\Delta p_{\max}}{\rho g} = \frac{c v_0}{g}$$

The above is sufficient for estimating pressure rise for rapid valve closure i.e. when

$t_c \leq \frac{2L}{c}$, when $t_c \geq \frac{2L}{c}$ the reflected negative pressure reduces the pressure rise at the

$$\frac{\Delta p}{\Delta p_{\max}} = \frac{2L}{c} \cdot \frac{1}{t_c}$$

value. In such cases one can use

B Elastic Water Column Theory

This takes the effect of compressibility of the water column in the pipe & the dilation of the pipe under high pressure in to consideration. Thus, for sudden valve closure, it is assumed that all the kinetic energy of the water is converted to the strain energy of the water (compression) & strain energy of the pipe (tensile)

In real world friction is present $\therefore$ wave dies away.

In absence of friction

$$h_w = \Delta h = -\frac{L}{g} \frac{dv}{dt} \dots\dots\dots (1).$$

For complete closure, $\Delta v = -v_0$.

Time of travel of wave is, $\Delta t = L/c \dots\dots (2)$

Considering the two cases

$$\text{Fluid volume diminished} = \Delta V = \frac{1}{K} \Delta p \cdot \frac{\pi D^2}{4} L$$

$$\text{where } K = \frac{\Delta p}{\Delta v/V}$$

$$\text{Pipe distended} \quad \Delta V = \frac{\pi D^3}{4 E} \cdot L \cdot \Delta P$$

$$\Delta V_{\text{total}} = \frac{\pi D^2 L}{4} \Delta p \left[\frac{1}{K} + \frac{D}{E} \right].$$

Combining

$$\text{thus } \delta L = \Delta V/A$$

$$\delta L = L \Delta p \left(\frac{1}{K} + \frac{D}{E} \right) \text{-----}(3)$$

$$\Delta p = \rho c v_0 \quad \text{Allevi expression}$$

$$c = \frac{1}{\sqrt{\rho \left(\frac{1}{K} + \frac{D}{T.E} k \right)}}$$

where $k = \left(\frac{5}{4} - \nu \right)$, for pipes free to move longitudinally
 $= (1 - \nu^2)$, anchored both ends against longitudinal movement.
 $= (1 - 5\nu)$, for pipes with exp. Joint.
 $= 1$ if longitudinal stress in the pipe is neglected.
 ν = poisson's ratio, for common pipe material
 $= 0.25$

For detailed design, one has to consider the following additional factors.

- i) The effect of friction in the penstock.
 - Friction is major factor which dampness the pressure oscillation. Incorporation of frictional effects helps to decrease the water hammer pressure.
- ii) The interaction b/n the size effect & the water hammer effect
- iii) The branching of the penstock – This further complicates the problem. In general more advanced analysis (Schuynder-Bergeron, Finite Difference, Integration) & package software are available.

3.4.2 Surge Tanks

General

The surge tank, also called the expansion chamber, is a structure which forms an essential part of the pressure conduit conveyance system whenever such system is long. Surge tanks may be considered essentially as a forebay close to a machine. Their primary purpose is protection of long pressure tunnel in medium and high-head plants against high water hammer pressure arising from sudden rejection or acceptance of load. The surge tank converts these high frequency, high pressure transients (water hammer) in to low frequency low pressure, mass oscillation.

It is located between the almost horizontal or slightly inclined pressure conduit and the steeply sloping penstock/pressure shaft. It is designed either as chambers excavated in the mountain or as a tower raising high above the surrounding terrain (see *Figure 5.24*).

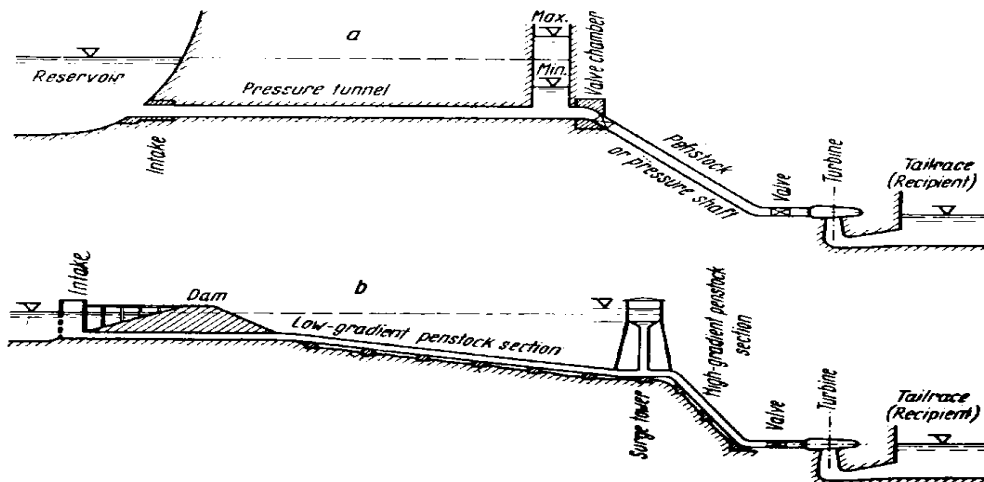


Figure 3.24: Typical Arrangements of Surge Tanks

A Function of Surge Tanks

The surge tank serves the following purpose

- It provides a free reservoir surface close to the discharge regulation mechanism. This will cut short & limit the conduit length liable to water hammer.
- It supplies the additional water required by the turbine during load demand (and during starting up) until the conduit velocity has accelerated to the final steady state level.
- It stores water during load rejection i.e closure until the conduit velocity has decelerated to the new steady state condition.
- It ensures that the water level oscillation following small and large load changes are dissipated rapidly.

B Types of Surge Tanks

Surge tanks may be classified according to :

- a) Material of construction, example. Concrete or steel
- b) Location relative to terrain
 - underground surge tank (excavated surge tank, see Figure 5.25)
 - over ground surge tank (Free standing surge tank, see Figure 5.24)

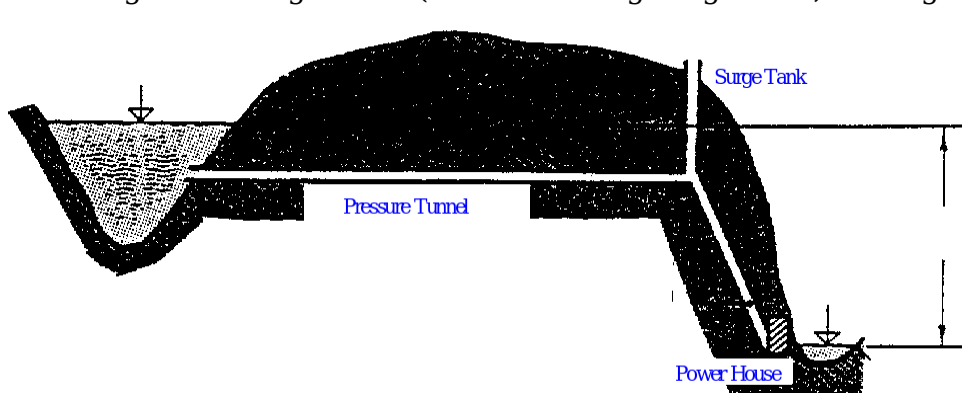


Figure 3.25: Underground Surge Tank and Power House

C Location in the hydraulic system

- Upstream surge tank (u/s to the power house) on the headrace tunnel

(see Figures 5.25 and 5-26).

- Downstream surge tank on the tailrace tunnel(see Figure 5-26).

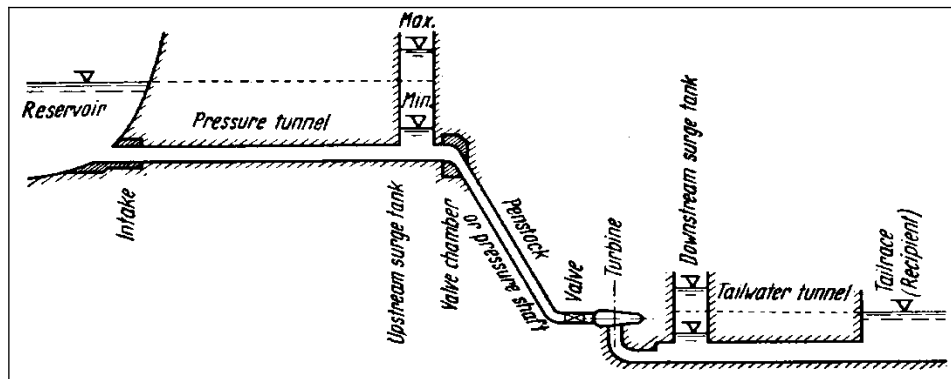
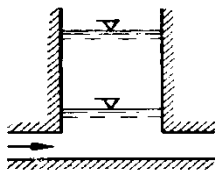


Figure 3-26: Downstream Surge Tank

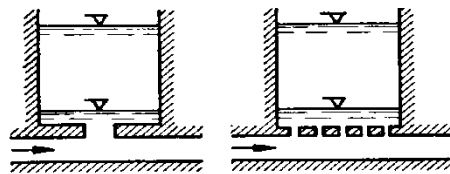
D Hydraulic functioning & cross-sectional shape

The most useful classification is on the basis of their shape, which also determines their hydraulic characteristic. A usually followed classification is as follows:

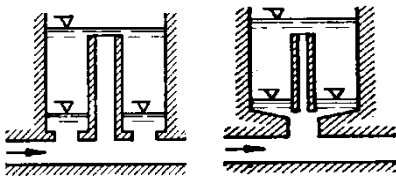
- ◆ Simple surge tanks
- ◆ Restricted orifice (or throttled) surge tanks
- ◆ Differential surge tanks
- ◆ Surge tanks with expansion chambers and others



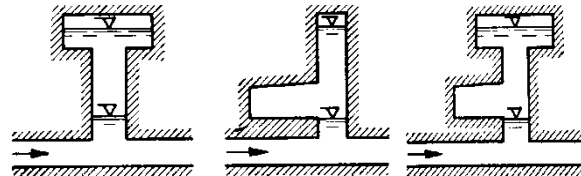
a) Simple surge tank



b) Restricted orifice surge tank



c) Differential surge tank



d) Surge tanks with expansion chambers

Figure 3-27: Surge Tank Types

E Design Consideration of Surge Tank

The hydraulic design of surge tank concerns with two main aspects.

- Its height
- Its cross-sectional area.

These aspects are decided up on with the view to fulfilling the following criteria:

- ◆ The surge tank must be locates so that the positive & negative water hammer pressures are kept within acceptable limits.

- ♦ The tank must be stable i.e. water surface oscillation must be damped out
- ♦ The tank must accommodate maximum upsurge & lowest down surge

a) Height

The total height of the surge tank should be such that both the maximum up-surge and down-surge is contained within the surge tank height. Worst conditions should be considered to determine the height.

- ♦ For up-surge, the worst conditions are:
 - Instantaneous total closure
 - Reservoir level at its maximum
- ♦ For down-surge, the worst conditions are:
 - Instantaneous total opening
 - Reservoir level at its minimum

The lowest possible level of down surge must be sufficiently above the conduit top level by a certain height h (see Figure 5.28) in order to avoid vortex formation at entrance to the penstock/pressure shaft.

In general, the total height of the surge tank would be (see Figure 5.28):

$$H_{\text{tank}} > Z_{1\text{ up}} + Z_{1\text{ down}} + H_{\text{live}} + Z_0 + h$$

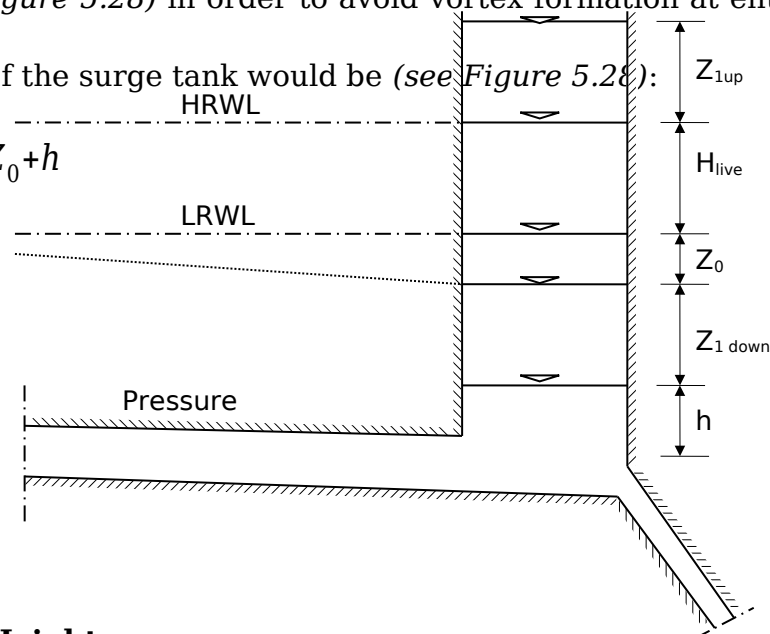


Figure 3-28: Surge Tank Height

In order to determine the surge height and thus necessary tank height, etc., it is necessary to carry out water hammer analysis and determine corresponding surge heights under various closure and opening (load rejection and acceptance) conditions.

i. Surge analysis in surge tank (simple surge tank):

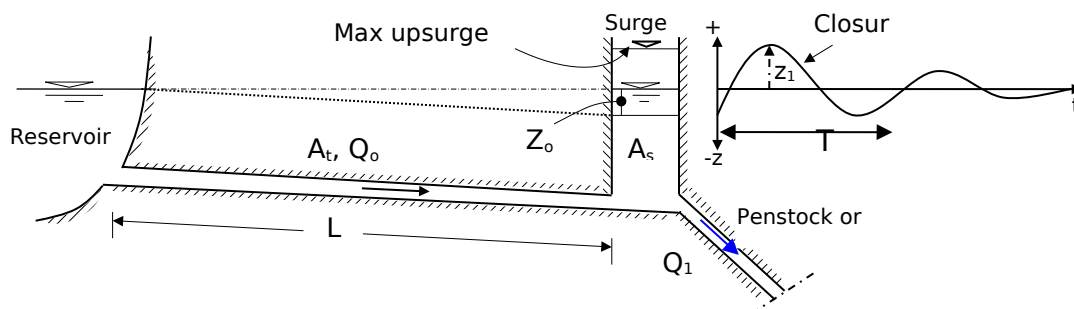


Figure 3-29: Damped surge oscillations - instantaneous closure**Continuity equation**

$$V A_t = A_s \frac{dz}{dt} + Q_1 \Rightarrow \frac{dz}{dt} = \frac{A_t V - Q_1}{A_s} \text{-----(1)}$$

Momentum equation (The dynamic equation)

$$\text{At steady state } -z_o = \lambda \frac{L v_o^2}{2gd} \quad (\text{if only friction loss is considered})$$

If at time, t, the water level is higher than that at steady state, a deceleration head is

$$\frac{\lambda L V^2}{2gd} - (-z)$$

being applied to the pipe line. i.e.

$$F_T = \frac{\lambda L}{2gd}$$

Denote

$$\therefore \Delta h = F_T V^2 + z = -\frac{L}{g} \frac{dv}{dt} \quad (\text{Wave theory, rigid column theory})$$

If surge tank throttle loss is considered

$$F_S V_S^2 + F_T V^2 + z = -\frac{L}{g} \frac{dv}{dt} \text{-----(2)}$$

1 & 2 could be combined to give 2nd order d.e. but not analytical solution.

b) Cross-sectional Area (Stability Consideration)

Characteristic oscillation in the surge tank damped by hydraulic friction, in the conduits;

$$\text{Amplitude of oscillation } \propto \frac{1}{A_s}.$$

The required cross-sectional area of a surge tank is determined based on stability considerations for the surge oscillations in the tank. Stability conditions of the surge system were established by *Thoma*. He stated that in order to prevent the development of unstable oscillations the cross-section of the surge tank should exceed a certain critical magnitude.

According to Thomas, the limiting x-sectional all for small oscillation is given by:

$$A_{Sc} = \frac{V_o^2 A_t L_t}{2g P_o H_o} (m^2)$$

$$\text{where } P_o = h_f = Z_o$$

$H_o = H - h_f = \text{net head on turbine neglecting turbine loss.}$

$$\beta = \frac{1}{m^2 R^{4/3}} \quad m = 1/n$$

Assuming

Maning's n

R - tunnel hydraulic radius

$$A_{SC} = \frac{m^2 R^{4/3} A_t}{2g H_o} = \frac{m^2 D^{10/3}}{160 H_o}$$

$$A_S \approx (1.5 \text{ to } 1.8) A_{SC} \quad (\text{stable tank})$$

$$\text{for } m=85; n=0.0118; A_{SC}=45D^{10/3}$$

For large amplitude of oscillation, the Thoma formula was modified by Ch Jaeger as

$$A_S = \eta^* \frac{L A_t}{2g \beta H_o} = \eta^* \frac{m^2 R^{4/3} A_t}{2g H_o}$$

η^* = non constant factor of safety = 1 +

$$1.0482 Z_{\max}/H_o$$

$$\text{or } A_S = \frac{\eta^* m^2 D^{4/3}}{160 H_o} \quad \frac{Z_{\max}}{H_o} \quad Z_o = V_o \sqrt{\frac{L}{g} \frac{A_t}{A_S}} \quad (\text{undamped friction loss})$$

3.4.3 Forebays

General

A forebay, also called a head pond, is a basin located at the end of a power canal just before the entrance to the penstock or pressure shaft. It acts as a transition section between the power canal and the penstock. It is formed simply by widening the power canal at the end. Figure 5.31 shows typical forebay.

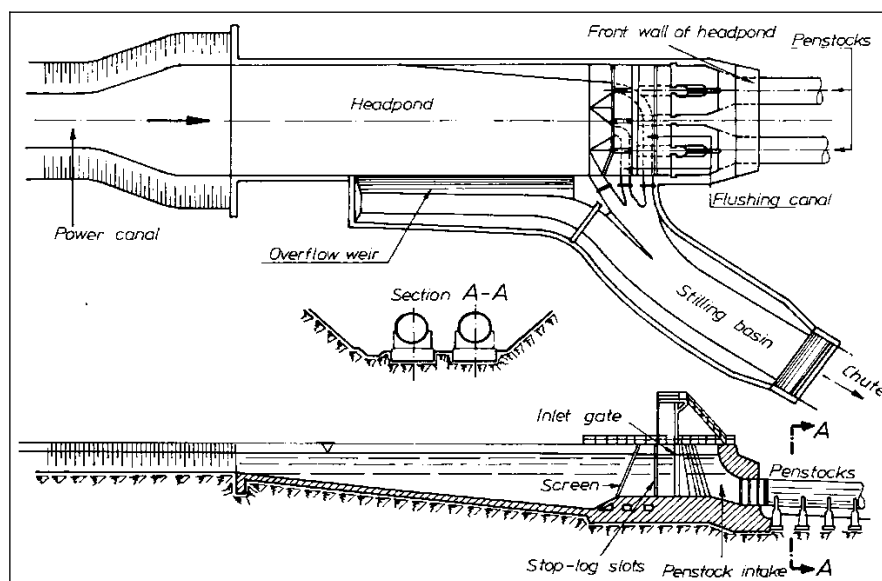


Figure 3.31 General Arrangement of a Forebay.

A. Functions of a Forebay

A forebay serves the following purposes:

It can serve as a *balancing reservoir*. Water is temporarily stored in the forebay in the event of a rejection of load (turbine closure) and the stored water will be withdrawn from it when the load is increased (turbine opening). In the case of low-head power plants, the forebay may even provide daily pondage for the plant. It can serve as a final *settling basin* where any water borne debris which either passed through the intake or was swept in to the power canal can be removed before the water passes into the turbine. In this case, the forebay must be large enough to reduce flow velocities sufficiently for settling to occur and to accommodate the sediment which accumulates between cleanings. It can serve to *distribute evenly the water* conveyed by the power canal among the penstocks, where two or more penstocks are provided.

B. Components of a Forebay

The following are the main components of a forebay:

- the *basin* used to store water and sediment (if any)
- the *spillway* used to dispose excess water that might enter the forebay
- the *bottom outlet* used for flushing out of the sediment stored in the basin as well as for de-watering the forebay and the power canal for maintenance
- the *penstock inlet* serves in controlling flow into the pressure conduit and in preventing floating debris from entering the conduit. It also provides smooth transition between the basin and the conduit.

C. Design Guidelines for a forebay

The location of the forebay is primarily governed by topographic conditions, yet of course, the geology of the site should also be considered. The site of both the forebay and the powerhouse should be selected simultaneously with a view to ensuring the shortest possible penstocks/pressure shafts. The entire basin of the forebay may be either excavated in rock or constructed above the terrain, enclosed by embankments and retaining walls.

The size of a forebay varies depending on the sediment content of the water conveyed in the power canal and whether it is to serve for storage. To be most cost-effective, the forebay must be of a size adequate to fulfill its function, neither significantly larger nor smaller. It is not advisable to design the forebay as a settling basin if the suspended sediment is fine to cause no damage to the turbines.

A gradual transition section should be provided between the power canal and the forebay basin. In the case of wide forebays, baffle piers are usually constructed at the basin inlet in order to ensure even distribution of flow to the basin.

The bottom of the forebay basin should be provided with a proper slope to enable periodical flushing of the silt deposited.

A bottom lining of the forebay basin is required in soils where large seepage is expected. As concrete lining of large basins is very expensive, a less costly solution of clay lining is usually adopted. The smoothed bottom of the basin is covered with plastic clay to a

thickness of 20 to 50 cm. The cover is compacted in several layers and is protected against disturbance due to soaking and wave action by a layer of gravel or crushed stone.

The spillway is usually an ogee type with stilling basin. If the discharge to be taken care of is great and if, at the same time, prevailing conditions do not permit the construction of a long overflow weir, water surface regulation within narrow limits can be attained by constructing a siphon type spillway.

The spillway and the bottom outlet canal should be combined immediately at the foot of the basin. Water spilling over the spillway crest and through the bottom outlet can be either diverted into a suitable river bed (if any) in a nearby side valley or conveyed by a special chute.

In designing a forebay tank, it is important to keep the entrance to the penstock fully submerged. This is to prevent air being drawn in to the penstock because of a vortex which can be formed if the penstock entrance is closer to the water surface in the basin.

The usual components of the intake such as trashracks, flow control devices (gates or valves), etc. must be provided at the penstock inlet. It is necessary to install an air vent behind the gate to prevent damage to the penstock if for some reason the penstock entrance is blocked or the gate is suddenly closed causing a low pressure inside the conduit which can make it collapse inwards. The air vent can also help remove air from inside the penstock during startup.

3.5 PENSTOCK

General

The penstock is high pressure pipeline between forebay (surge tank or reservoirs) and the turbine. The design principles of penstocks are the same as that of pressure vessels & tanks but water hammer effect has to be considered. For short length, a separate penstock for each turbine is preferable. For a moderate heads & long distances a single penstock is used to feed two or more turbines through a special branching pipe called Manifold.

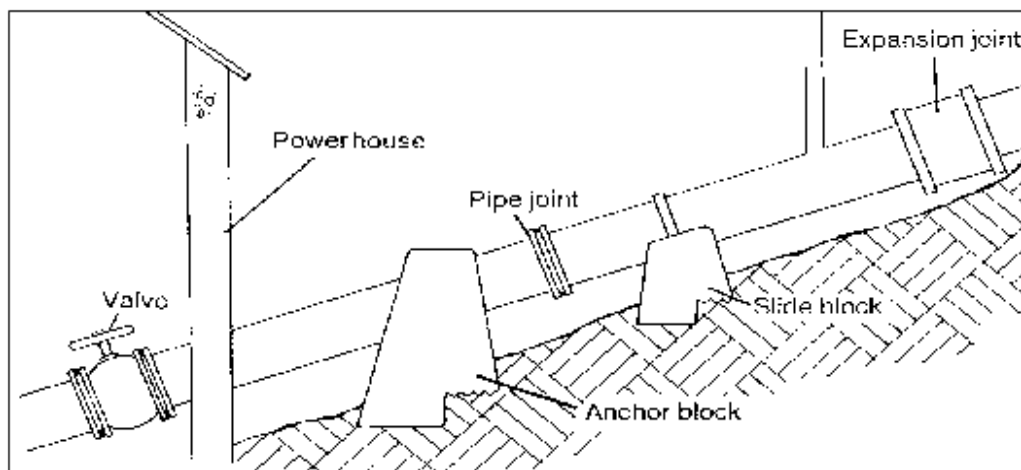


Figure 3.32: Components of a penstock

D. Classification of penstock.

Classification may based on :

- | | |
|--|------------------------|
| 1. The material of construction | 2. Method of support |
| 3. Rigidity of connection and support. | 3. Number of penstocks |

1) Material of construction

Factors for the choice of material are: head, topography & discharge. Various materials used are steel, R.C., asbestos cement, PVC, wood stave pipes, banded steel, etc. The following factors have to be considered when deciding which material to use for a particular project:

Required operating pressure	Diameter and friction loss	Weight and
ease of installation	Accessibility of site	Cost of the penstock
Design life		
Availability	Weather conditions	

2) Method of support

A penstock may be either buried or embodied underground (or inside dams) or exposed above ground surface & supported on piers.

Buried penstocks: are supported on the soil in a trench at a depth of 1 to 1.5m and back filled. The general topography of the land should be gentle sloping and of loose material.

Advantages

1. Continuity of support given by the soil provides better structural storability.
2. Pipe is protected from high temperature fluctuations
3. Conservation of natural land escape
4. Protection from slides , storms & sabotage.

Disadvantages

- 1- Difficulty in inspection
- 2- Possibility of sliding on step slopes
- 3- Difficulty in maintenance
- 4- Expensive for loge diameter in rocky soils.

Exposed penstocks: supported on piers or saddles.

Advantages

- 1- Ease in inspection of defects & maintenance
- 2- Economy in rocky terrain & large diameters.
- 3- Stability is insured with proper anchorage

Disadvantages

- 1- Direct exposure to weather effect
- 2- Development of longitudinal stress due to support and anchorage, thus requiring expansion joints

When the situation warrants, partly buried system, may be adopted which combines the advantages of both system.

3. Rigidity of connection & Support

There are three possible methods of support ,

- a) Rigid pipe support: Here every support is an anchorage so that any movement is checked. completely. This type is suitable when the temperature audition is moderate.
- b) Semi- rigid pipes: Here each member of the pipe line is fixed at one and leaving the possibility of movement over the other support.
- c) Flexible support (Flexible or loose- coupled pipes): Here expansion joint are introduced between each adjacent section

4. Number of Penstocks

The number of penstocks used at any particular installation can be single or multiple. The general trend at older power stations was to use as many penstocks between the forebay/surge tank and the powerhouse as the number of units installed. The recent trend is to use a single penstock, unless the size or thickness of the penstock involves manufacturing difficulties.

When a single penstock feeds a number of turbines, special sections called manifolds are used at the lower end of the penstock to direct flow to individual units. The design of such sections is an intricate job and has to be analyzed carefully.

The advantages of using a single penstock over the use of multiple penstocks are:

- ♦ The amount of material required to manufacture is less, making it economical.
- ♦ The cost of civil engineering components such as penstock supports and anchors is less.

On the other hand, the use of a single penstock means reduced safety of operation and complete shutdown will become necessary in case of repair. Furthermore, significant losses are usually experienced at the manifolds.

In general, the use of multiple penstocks is preferably employed for low-head plants with short penstocks; whereas for high-head plants requiring long penstocks, provision of a single penstock with manifold at the end usually proves economical.

3.5.1 Hydraulics

1) Permissible velocities.

3 to 5 m/s (no abrasion property settled water) for properly settled water in exceptional cases up to 5m/s may be tolerated.

$$A = \frac{Q_0}{v_0} = \frac{\pi D^2}{4} \Rightarrow D = 1.128 \sqrt{\frac{Q_0}{v_0}}$$

Therefore:

Head losses

$$\text{i) Frictional head loss} \Rightarrow h_f = \lambda \frac{L}{D} \frac{v^2}{2g}$$

$$\text{ii) other local losses } h_{f_c} = k \frac{v^2}{2g} \text{ where } k = \text{local loss coefficient}$$

Penstock with gradual transition entrance	0.10 - 0.20
---	-------------

Open butterfly valve (depending on disk thickness)	0.05 - 0.25
--	-------------

Needle valves	0.20 - 0.25
---------------	-------------

Bends (depending on deflection angle)	0.05 - 0.15
---------------------------------------	-------------

Net head: $H = H_g - \Sigma \text{ losses}$ where H = net head, H_g = gross head.

For H_g : elevation of water level at the forebay or reservoir at the upstream end, and at d/s end - free tail water level in reaction type turbines (Francis) or elevation of jet nozzle in case of impulse/action turbine (Pelton)

1. Economical Diameter of Penstock

The diameter of the penstock is determined from economic consideration and then checked to see that acceptable velocities are not exceeded

Two approaches - Graphical (economic analysis)

- Empirical equations

i) **Graphical approach** : D - f (capital cost, running cost)

If D is small, large h_f , reduction in output, loss in revenue. If D_1 is large, small h_f , greater output, larger initial cost.

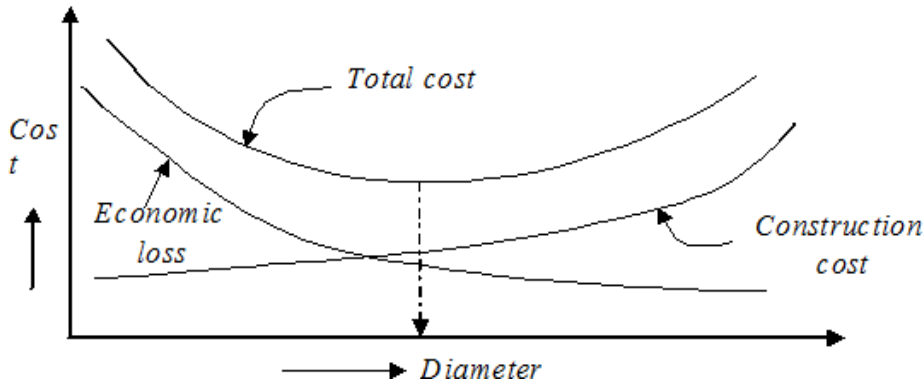


Figure 3.33: Economical Diameter of Penstock

ii) For preliminary design various **empirical formula** are available

1) SARKARIA'S Formula

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

D - penstock diameter (m)

P - hp transmitted by the pipe

H - net head at the end of the penstock is m.

2) USBR

$$v = 0.125 \sqrt{2gH}$$

v - Optimum velocity

3) Donald's formula

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

4) Fahlbusch (2982)

$$D = 0.52 H^{0.17} \left(\frac{P}{H} \right)^{0.43}$$

2) Structure Analysis of the Penstock

- It is necessary to construct pressure grade line
- In addition to pressure heads, to water hammer pressure have to be determined

From previous

$$C = \sqrt{\frac{E_m}{\rho}} \quad \text{Where} \quad \frac{1}{E_m} = \frac{1}{K} + \frac{D}{TE}$$

$$\text{For instant closure i.e. } t_c \leq \frac{2L}{C} \quad h = \frac{Cv_o}{g}$$

For all other closure $t_c > \frac{2L}{C}$, the max pressure rise at the valve

$$h_{\max} = \frac{h}{C} \cdot \frac{2L}{t_c} = \frac{C v_o}{g C} \cdot \frac{2L}{t_c} = \frac{2v_o L}{g t_c}$$

The value of water hammer pressure rise as computed above may be taken at the turbine gate, reducing to zero at intake or surge tank level. Values at intermediate location may be calculated assuming a straight line variation

Thus, design head H = static head plus water hammer head.

For thin walled vessels, where $\frac{D}{t} \geq 20$ $\sigma = \frac{pD}{2t}$

The design pressure, $p = \phi H$ $t = \frac{PD}{2\sigma}$

In the above $\sigma \leq \phi \sigma_a$; σ_a = allowable stress

ϕ = is coefficient depending on joint type. For welded joint, ($\phi = 0.85$ - 0.95)

For steel used in penstocks usually a factor of safety of 3 to 3.5 is used. Thus for material with ultimate tensile strength of 3700 kg/cm^2 ; $\sigma_a \leq 1200 \text{ kg/cm}^2$

Thus for design purposes, $t \geq \frac{pD}{2\phi\sigma_a}$

For protection against coating deterioration add 1 to 3mm to the above value.

For thick welded pipe where $\frac{D}{t} < 20$, the following formula give sufficient accuracy

$$t = \frac{D}{2} \left(\sqrt{\frac{\phi\sigma_a + 0.07H}{\phi\sigma_a - 0.13H}} - 1 \right) + (1 \text{ to } 3) \text{ mm}$$

$$t = \frac{pr}{\sigma_a \phi - 0.6P} + 0.15$$

The ASME gives the formula for thickness as

Where t in cm
 p pressure in kg/cm^2
 r internal radius in cm
 σ_a design stress in kg/cm^2
 ϕ Joint efficiency factor
 0.15cm is allowance for corrosion

In case where the negative water column gradient falls below the penstock center line, there is danger of collapse of the penstock due to external atmospheric pressure. The external pressure p_e likely to result in collapse may be computed from the formula by Mayer

$$p_e = \frac{3EI}{r^3} = \frac{24EI}{D^3} \text{ kg/cm}^2$$

I = moment of inertia of x-section of the pipe ring $\frac{t^3}{12} \text{ m}^3$

E = modulus of elasticity of steel

$$p_e = \frac{1}{\eta} 2 \left(\frac{t}{D} \right)^3$$

Introducing a S.F. η , $t = D^3 \sqrt[3]{\frac{\eta p_e}{2E}}$ $n=2$ for buried pipes; $n=4$ for exposed pipes

For example for complete vacuum, $t = D^3 \sqrt[3]{\frac{4 \times 1}{2 \times 2 \times 10^6}} = 0.01 D$

3) Penstock Joints

Penstock pipes are generally supplied in standard lengths, and have to be joined together on site. There are many ways of doing this, and the following factors should be considered when choosing the best jointing system for a particular scheme.

- ♦ Relative costs
- ♦ Suitability for chosen pipe material
- ♦ Ease of installation
- ♦ Degree of joint flexibility

Methods of pipe jointing fall roughly into four categories:

- ♦ Flanged joints
- ♦ Mechanical joints
- ♦ Spigot and socket joints
- ♦ Welded joints

Flanged Joints:

Flanges are fitted to each end of individual pipes during manufacture, and each flange is then bolted to the next during installation as shown in Figure 5.34.

A gasket or other packing material is necessary between each flange of a pair. Flange jointed pipes are easy to install, but flanges can add to the cost of the pipe. Flange joints do not allow any flexibility. They are generally used to join steel pipes, and occasionally ductile iron pipes.

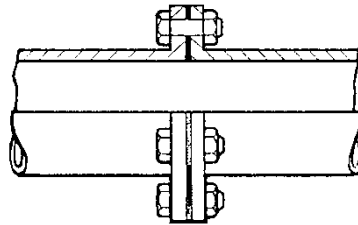


Figure 3.34 Flanged joints

Spigot and Socket Joints:

Spigot and socket joints are made by either fitting a collar to, or increasing the diameter during manufacture of, one end of each pipe such that the internal diameter of the collar or increased internal diameter of the pipe is the same as the external diameter of the pipe. The plain end of each pipe can thus be pushed into the collar or 'socket' in the next as shown in Figure 5.35.

A good seal is required between each pipe section, and this is achieved by either providing a rubber seal or special glue called solvent cement, depending up on the material of which the pipes are made.

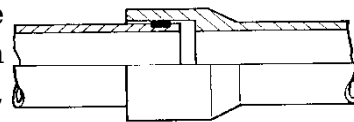


Figure 3.35 Spigot & socket joints

Spigot and socket joints are generally used to join ductile iron, PVC, concrete, and asbestos cement pipes.

Mechanical Joints:

Mechanical joints are rarely used on penstocks because of their cost. One important application of it is for joining pipes of different material or where a slight deflection in the penstock is required that does not warrant installing a bend.

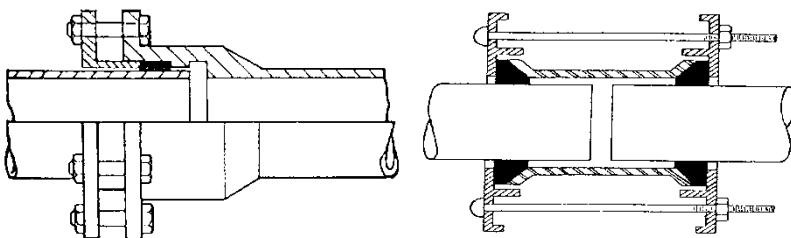


Figure 3.36 Mechanical joints

Welded Joints:

Welded joints are used on penstocks made of steel. Steel pipes are brought to the site in standard lengths, and then welded together on site. One advantage of welding on site is that changes in the direction of the pipe can be accommodated without preparation of a special bend section. It is relatively cheap method, but has the drawback of needing skilled site personnel.

Expansion Joints:

A penstock, specially exposed ones, will change in length depending on temperature fluctuations. If it is fixed the thermal expansion forces are substantial. It is possible to relieve these forces by incorporating special joints called *expansion joints*, which allow the pipe to expand and contract freely.

For short penstocks, provision of a single expansion joint may be sufficient, but for long penstocks with a multiple anchor blocks expansion joints should be placed below each anchor block. Another alternative to take care of thermal expansion is to take in to account the forces that result from it in designing anchors.

5.3.7 Penstock Supports and Anchors

Slide blocks, anchors, and thrust blocks all serve the same basic function – to constrain movement of the penstock. Different terms are used with these structures simply to indicate which specific function they serve, and this is discussed in the following paragraphs, see Figure 5.32 for the sketches.

Slide Blocks:

A slide block, also called supporting pier, carries the weight of pipe and water, and restrains the pipe from upward and sideways movements, but allows it to move longitudinally. In most cases the spacing between slide blocks are assumed equal to the length of each pipe.

If the penstock is buried, slide blocks are unnecessary; rather instead the pipe is laid in a trench on a bed of sand or gravel of consistent quality, with no big stones which could cut into the pipe or cause stress concentrations on the pipe wall.

Forces that act on slide blocks include:

Weight of the pipe and enclosed water: As slide blocks do not resist longitudinal forces, only the component of the weight perpendicular to the pipe will be considered.

Friction forces on the blocks. This is due to the longitudinal movement of the pipe over the blocks caused by thermal expansion and contraction.

Weight of the block itself

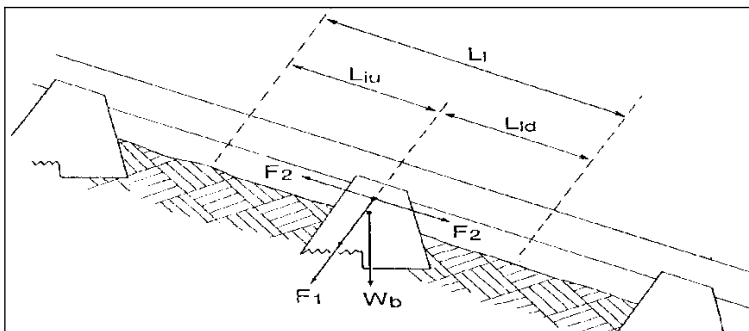


Figure 3.37 Forces on a slide block

Anchor Blocks:

An anchor block consists of a mass of reinforced concrete keyed to the penstock so that the penstock cannot move in any way relative to the block. It is designed to withstand any load the penstock may exert on it.

Anchors are often used at bends (horizontal and vertical) and before entrance to the powerhouse (see Figure 5.32). They can also be used along long straight sections of penstock, each one next to expansion joint.

Because an anchor is keyed to the penstock pipe and is also frequently located at a bend in the pipe, more forces act on an anchor than on a slide block. The major forces which act on anchor blocks are the following:

- Weight of the pipe and enclosed water
- Hydrostatic force on a bend
- Friction forces on slide blocks located between the anchor and expansion joint
- Thermally induced stresses, when expansion joints are not incorporated
- The weight of the anchor block itself

Thrust Blocks:

These are a special form of anchor whose sole purpose is to transmit forces primarily caused by hydrostatic pressures at horizontal bends along a buried penstock to undisturbed soil which provides the reaction force (see Figure 5.38). However, if the bend is vertical, an anchor block is still used if the back filled soil is not able to resist this force.

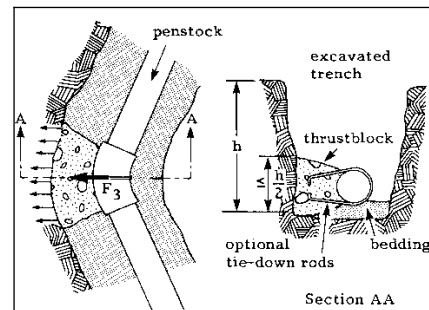


Figure 5.38 Thrust block

Conditions of Stability for Supports and Anchors:

For any penstock support or anchor to be stable and fulfill its intended purpose, the following conditions must be met:

The structure should be safe against *sliding*. For sliding not to occur: $\sum H < \mu \sum V$

Where, $\sum H$ and $\sum V$, respectively, are the sum of all horizontal and vertical forces, and μ is the coefficient of friction between the structure and the ground often assumed as 0.5.

The structure should be safe against *overturning*. For this condition to be fulfilled, the resultant force should act within the middle third of the base. In other words,

$$e < \frac{L_{base}}{6}$$

Where, e is eccentricity of loading and L_{base} is length of the structure base.

The pressure transmitted to the foundation must be within the safe bearing capacity of the foundation material. This can be expressed as:

$$\left(\text{Maximum pressure } \frac{\sum V}{A_{base}} \left(1 + \frac{6e}{L_{base}} \right) \right) < \left(\text{Bearing capacity of } \right)$$

4) 5.3.8 Penstock Valves

Valves are usually installed at two places in a penstock. One valve is provided at the upstream end of the penstock, i.e., at the forebay or immediately after the surge tank, and is called *penstock inlet valve*, while the second is provided at the downstream end of the conduit, immediately a head of the turbine, and is named as *turbine inlet valve*. The upper valve is sometimes replaced by a gate.

The main purpose of penstock inlet valve is for dewatering of the penstock in case maintenance of the penstock is required. But, it can be omitted for short penstocks where the closure of the power canal or power tunnel is possible from the intake.

The main purpose of turbine inlet valve is to close the penstock while the turbine is inoperative. It can also act as an emergency shut-off device. This valve cannot be omitted except under especial case where the penstock supplies a single unit having installed the penstock inlet valve. The number of turbine inlet valves required at a power station is governed by the number of turbine units installed, but not by the number of penstocks, as a single penstock can serve a number of units through a manifold at the end. There are various types of valves for use in hydropower installations.

The most frequently applied include:

- ♦ Gate valves
- ♦ Butterfly valves
- ♦ Spherical valves
- ♦ Needle valves