

CHAPTER FOUR

Hydraulic Structures In Hydropower Stations

4. Introduction

Sources of Energy

The following two major sources of power generation may be identified on the basis of present day importance:

- a) *Conventional sources*
 - i) Thermal power
 - ii) Hydropower
 - iii) Thermo-nuclear power
- b) *Unconventional sources*
 - i) Tidal power
 - ii) Solar power
 - iii) Geothermal power
 - iV) Wind power
 - V) Wave power
 - Vi) Depression (solar) power

Hydropower Potential and Its Development in Ethiopia

Although there is no recorded history, the use of waterpower in Ethiopia in its non-electric form is estimated to exist since long period of time. It has been used in the water mills, and such practice is still under use in some rural areas of the country. The water power use in its more effective form, i.e. electricity generation, came to existence in the beginning of 1930's, when Abasamuel hydropower scheme is commissioned in 1932. This station was capable of generating 6MW and operational up to 1970. In Ethiopia, by 1990, about 94% of the energy requirement satisfied through the traditional energy sources, and the remaining 6% through modern sources such as fuel oil, gas and electricity.

According to Ministry of Mines and Energy, in 1990 the energy total requirement in Ethiopia was estimated at 177.6 TWh per year of which 76.1% from wood, 16.1% agricultural by-product, 5.3% from fuel oil and 1.1% from electricity, 0.8% from charcoal and 0.6% through others. The energy is used in the sectors of domestic in the town and rural areas, industry, service, agriculture and transport.

The 1.1% contribution of electrical energy of the total energy requirement is derived mainly from hydropower. This is used mainly for domestic use in the towns, industries and services. Ethiopia has got substantial hydropower potential from which less than 2% has been utilized, and the remaining should be developed at small to large scale so that source of energy for various uses can be replaced by this more environmentally friendly alternative source.

Estimation of Water Power Potential

It is essential to assess the inherent power available from the discharge of a river and the head available at the site before any power plant is contemplated.

The gross head of any proposed scheme can be assessed by simple surveying techniques, where as hydrological data on rainfall and runoff are essential in order to assess the available water quantities. The following hydrological data are necessary:

- a) The daily, weekly or monthly flow over a period of several years, to determine the plant capacity & estimated output.

b) Low flows, to assess the primary, firm, or dependable power.

The potential or theoretical power in any river stretch with a difference in elevation H is computed from:

$$P_p = \gamma \cdot Q \cdot H$$

This is a power that can be required for useful work by overcoming friction loss in watts.

Where

H = head in m

Q = discharge of streams in m^3/s

P_p = Potential (theoretical) power of the stream in KW

$$\gamma = \rho \cdot g \rightarrow \gamma = \rho \cdot g / 1000 = 9.81 \text{ KN/m}^3$$

Actually, g varies between 9.768 m/s^2 at equator to 9.83 m/s^2 at the poles (according to latitude) and according to local condition i.e. altitude, varies between -0.2 to -0.4 cm/s^2 in average -0.31 cm/s^2 per 1000m above sea level, see Mosonyi, E. (1987). Generally an average value of 9.81 m/s^2 is used.

The above equation neglects the difference in kinetic energy term. In low land rivers, with large magnitude of discharge and low head as in the runoff plants, neglecting the energy from this term may mean neglecting significant energy term.

From the above relationship:

$$P_p = \gamma \cdot Q \cdot H \text{ (KW)} = 9.81 Q \cdot H \text{ (KW)}$$

Since

$$1 \text{ hp} = 736 \text{ Watts}$$

$$P_p = 13.33 Q \cdot H \text{ (hp)}$$

The hydraulic power P is given by

$$P = \eta \cdot \gamma \cdot Q \cdot H = 9.81 \eta \cdot Q \cdot H \text{ (KW)}$$

Where η = is the total efficiency

If the river course is divided in to a number of n stretches, the total power can be described by:

$$P = \gamma \sum_{i=1}^n (Q \cdot H)$$

From the available stream flow data, one can obtain *flow duration curve* of the stream for a given site by plotting the discharge against the percentage duration of the time for which it is available. Similarly, *power duration curve* can be plotted since power is directly proportional to the discharge and available head.

Potential power resources can be characterized by values according to the discharge taken as a basis of computation. The conventional discharges are Q_{100} , Q_{95} , Q_{50} , Q_m . Thus we have,

i) *Minimum potential power* designated P_{p100} , computed from the minimum flow that is available for 100% of the time (365 days or 8760 hrs.)

ii) *Small potential power* computed from the flow available for 95% of the time. This represented by P_{p95}

iii) *Median potential power* is computed from the flow available for 50% of time. This is represented by P_{p50} .

iv) *Mean potential power* is computed from the average of mean yearly flows for a period of 10 to 30 years. This is designated as P_{pm} and is also known as *gross power potential*.

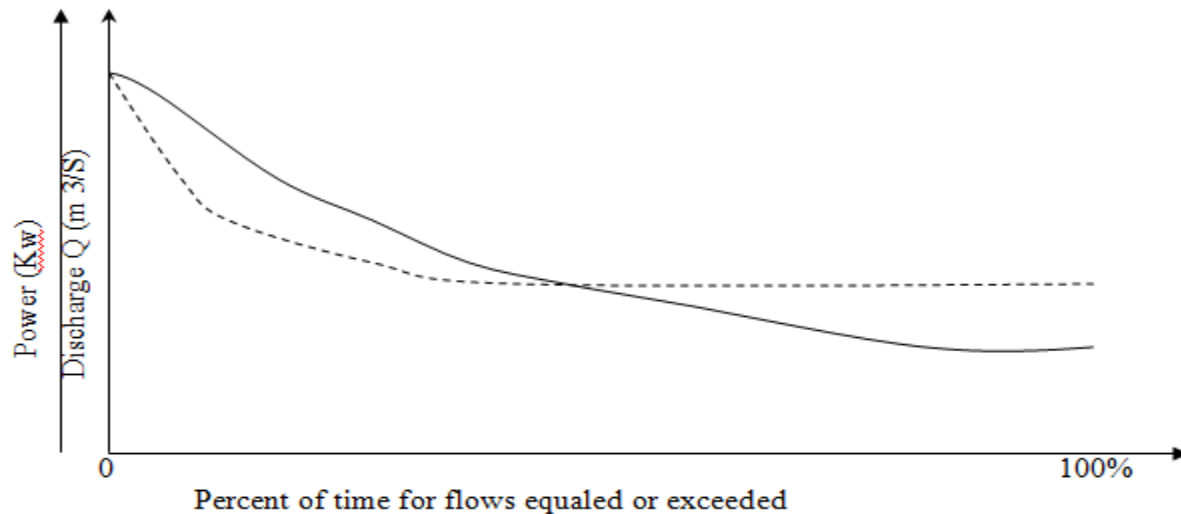


Figure 4.1: Flow/Power duration curve (Power scale multiplying factor = $9.81 \eta H$)

4.2 Classification and Types of Hydropower Development

Classification and Basis

Hydropower plants could be classified on the basis of

- Location & topographical features
- Presence or absence of storage
- The range of operating heads
- The hydraulic features of the plant
- Operating features etc.

A complete understanding of the type requires information under all such categories. All the above classification basis are not mutually exclusive.

1) Classification based on hydraulic features

The basic hydraulic principle governs the type.

i) *Conventional Hydro-plants*

- ♦ Use normally available hydraulic energy of the flow of the river.
- ♦ Run-of river plant, diversion plant, storage plant

ii) *Pumped storage plants*

- ♦ Use the concept of recycling the same water.
- ♦ Normally used with areas with a shortage of water
- ♦ It generates energy for peak load, and at off-peak periods water is pumped back for future use.
- ♦ A pumped storage plant is an economical addition to a system which increases the load factor of other systems and also provides additional capacity to meet the peak load.

iii) *Unconventional Hydro-plants*

a) Tidal power plant

- Use the tidal energy of the sea water.
- Very few have been constructed due to structural complication.

b) Wave power plant

c) Depression power plant

- Hydropower generated by diverting an ample source of water in the natural depression

- Water level in the depression is controlled by evaporation

2) Classification on the basis of operation

Based on actual operation in meeting the demand one can have:

- ♦ isolated plant - operating independently (not common now a days)
- ♦ interconnected in to grids

Thus in a grid system, a power station may be distinguished as a *base load* plant or *peak load plant*. Hydropower plants are best suited as *peak load plants*, because hydropower plants can start relatively quickly and can thus accept load quickly.

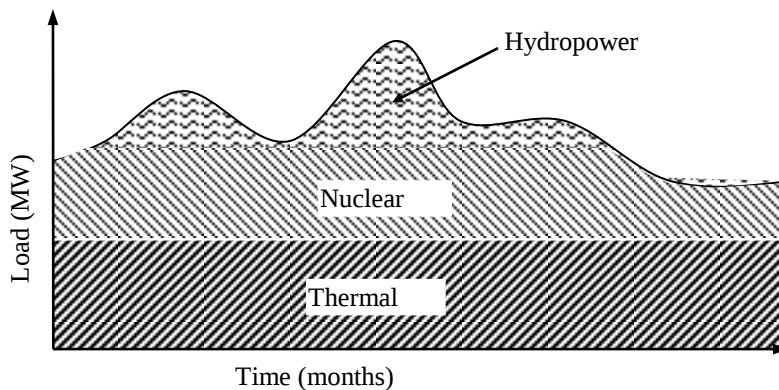


Figure 4.2: Place of hydropower in a power system.

3) Classification based on plant capacity

Classification based on plant capacity changes with time as technology improves. Thus we have the following classification according to Mossonyi, and present day trend classification.

According to Mossonyi

- i) Midget plant up to 10 KW
- ii) Low capacity < 1000KW
- iii) Medium capacity < 10,000KW
- iv) High capacity > 10,000KW

Present day classification

- i) Micro hydropower < 5 MW
- ii) Medium plant 5 to 100 MW
- iii) High capacity 100 to 1,000 MW
- iv) Super plant above 1,000 MW

Thus must hydropower plants in Ethiopia may be classified as medium to high.

4) Classification based on head

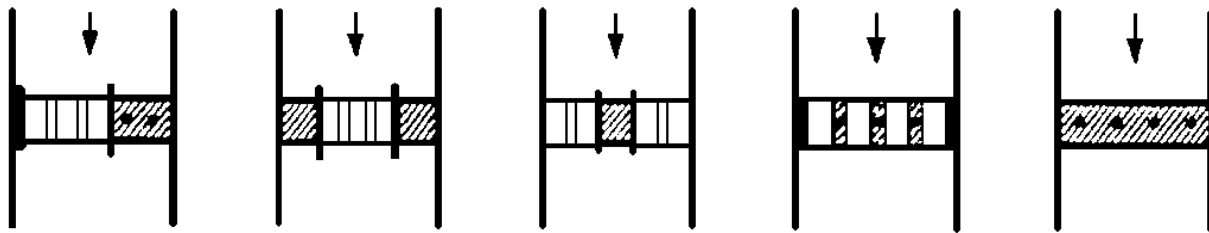
The most popular & convenient classification is the one based on head on turbine. On this basis:

- i) Low head plants < 15m
- ii) Medium head plants 15-50m
- iii) high head plants 50-250m
- iv) very high head plants > 250m

The figure may vary depending on the country standard

5) Classification based on constructional features (layouts)

i) Run-off-river plants (low to medium head plants)



a) Block power plant b) Twin block plant c) Island plant d) pier head plant e) Submersible plant

Figure 4.3: Run-off-River Plant Arrangement

ii) Valley dam plants (medium to high head plants)

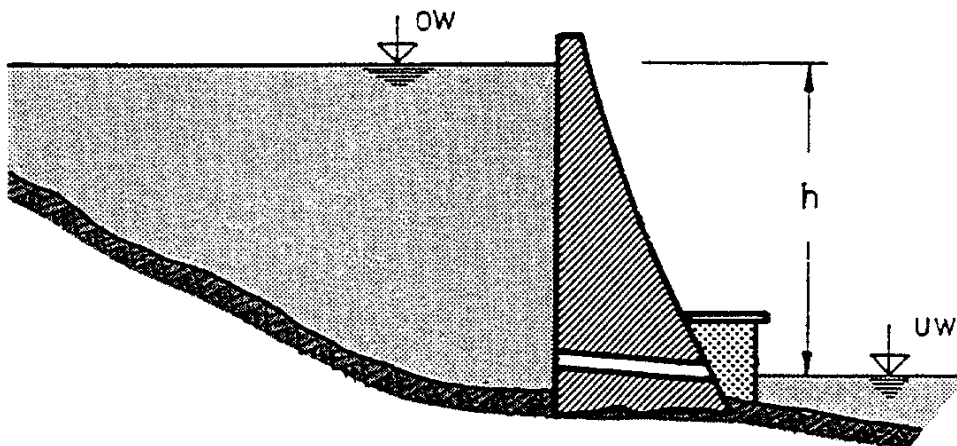


Figure 4.4: Valley Dam Plant Arrangement

iii) Diversion canal plant

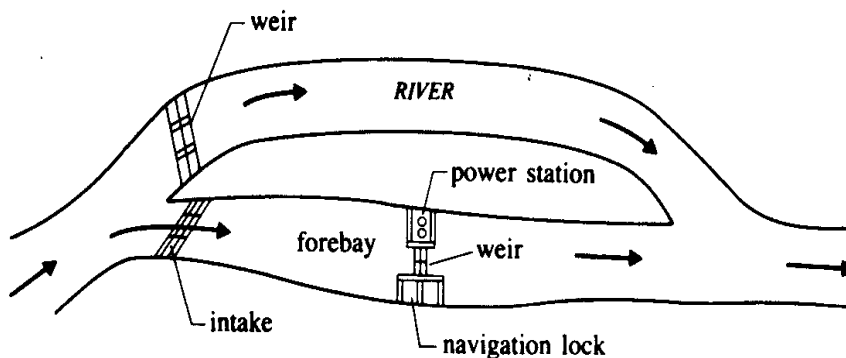


Figure 4.5: Diversion Canal Plant Arrangement

iv) High head diversion plants

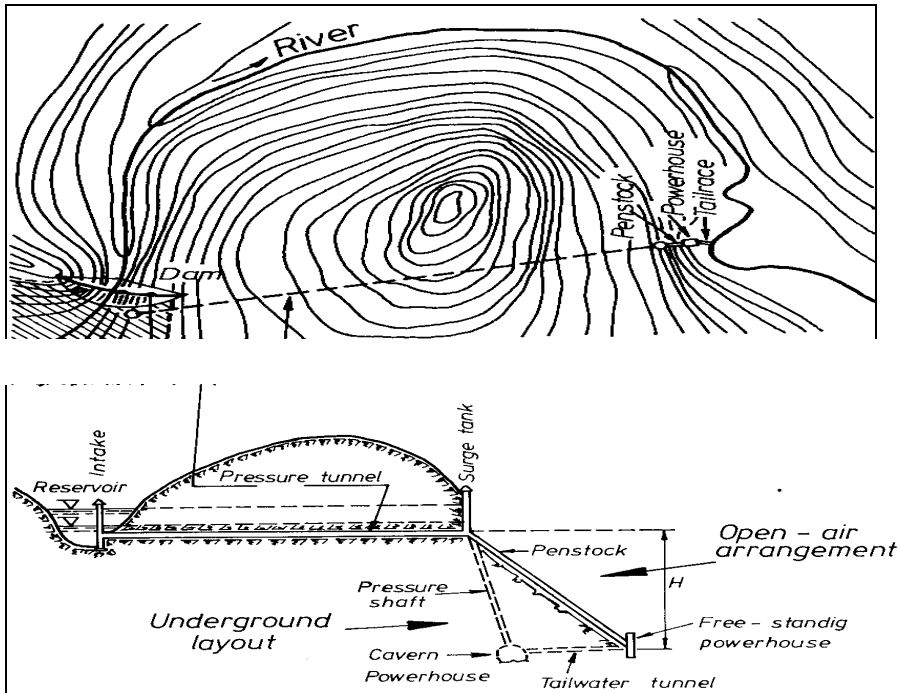


Figure 4.6: High Head Plant Arrangement

v) Pumped Storage Plant

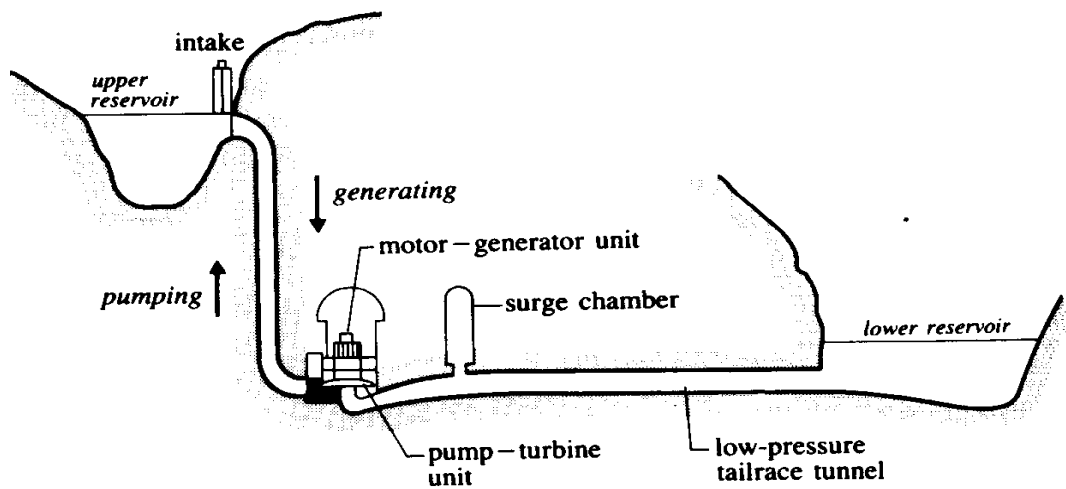


Figure 4.7: Pumped Storage Plant Arrangement

4.2 Site Selection, Layouts and Arrangements:

Run-off-river plants (low to medium head plants)

- ♦ The normal flow of the river is not distributed
- ♦ There is no significant storage

- ♦ A weir or barrage is built across a river & the low head created is used to generate power. It also acts as a controlled spilling device.
- ♦ The power house is normally in the main course of the river
- ♦ Preferred in perennial rivers with moderate to high discharge, flat slope, little sediment and stable reach of a river.

Water enters the power house through an intake structure incorporating some or all of the following.

- 1.- Entrance flume separated by piers and walls for each machine unit.
- 2.- Turbine chamber: scroll case with turbine
- 3.- Concrete or steel draft tube
- 4.- Power house building

Additional structures are

- deflector or skimmer walls
- forebay
- service bridge
- river training walls
- sediment trap and flushing sluices, where necessary

Valley dam plants (medium to high head plants)

- ♦ The dominant feature is the dam which creates the required storage (to balance seasonal fluctuation) and necessary head for the power house.
- ♦ Power house is located at the toe of the dam
- ♦ Water flows through the penstock embedded in the dam & enters the power house.
- ♦ Sometimes the power house is not immediately at the toe of the dam but at some distance (eg. the Koka power plant). This arrangement is more expensive (due to longer conveyance) and is used only when it offers advantages such as extra head due to advantageous topographical conditions.

Important components of a valley dam plant

- 1.- The dam with its appurtenance structures like spillway, energy dissipation arrangements etc.
- 2.- The intake with racks, stop logs, gates & ancillaries
- 3.- The penstock conveying water to the turbine with inlet valve & anchorage.
- 4.- The main power house with its components.

Diversion canal plant

- ♦ The distinguishing feature is the presence of power canal that diverts the water from the main stream channel.
- ♦ The power house is provided at suitable location along the stretch of the canal
- ♦ The water often flowing through the turbine is brought back to the old stream.
- ♦ Diversion canal plants are generally low head or medium head plants.
- ♦ They don't have storage.
- ♦ Pondage requirement is met through a pool called forebay located just u/s of the power house.

Ways of developing required head

- i) The flatter slopes of power canal and the absence of meander, by reducing length, helps in providing head.

Let distance from A to B along main river be 15km

>> average slope of main river be 1 in 500

∴ Total head difference b/n A & B = 30m.

Let length of power canal be 8km

>> average slope of power canal be 1 in 800

∴ Level difference b/n A & forebay = 10m

∴ Difference b/n forebay & B = 30-10 = 20m

ii) If the river has a natural fall, diverting the water from u/s side of the fall & locating the power house at the d/s side of the fall provide the required head.

iii) In inter-basin diversion, water may be diverted from a higher level river to a lower river through a diversion canal to the power house located at the lower river.

Main structures of the diversion canal plant:

- 1) Diversion weir with its appurtenant structures.
- 2) Diversion canal intake with its ancillary works such as sills, trash racks, skimmer wall, sluice, settling basin, disilting basin, disilting canal, silt exclusion arrangement is needed in some sediment laden streams.
- 3) Bridges and culverts of the canal.
- 4) Forebay & its appurtenant structures.

High head diversion plants

High head is developed by:

- i) Diverting the river water through systems of canals and tunnels to a downstream point of the same river.
- ii) Diverting the water through canals and tunnels to a point on another river which is at much lower level.

The main feature here is complicated conveyance system & relatively high head compared to the diversion type.

There may be two situation concerning storage situation

- a) A diversion weir to create pondage (and no storage). Here like run-off-plant the power production is governed by the natural flow in the river.
- b) Storage may be provided on the main river at the point of diversion. (This second situation is advantageous since the fluctuation in reservoir level does not materially affect the head and the power output can be adjusted by the controlled flow release from the reservoir. Eg. Fincha & Melka Wakana power plants)

Main Components of high head diversion plants:

- 1) Storage or diversion weir with appurtenant structures
- 2) The canal/tunnel
- 3) The head race either open cut or tunnel.
- 4) Forebay/surge tank
- 5) Penstock
- 6) Power house
- 7) The tail race

Pumped Storage Plant

Pumped storage plant is suitable where:

- ♦ The natural annual run-off is insufficient to justify a conventional hydroelectric installation
- ♦ it is possible to have reservoir at head & tail water locations.

This kind of plant generates energy for peak load, & at off peak period water is pumped back for future use. During off peak periods excess power available from some other plants in the system is used in pumping back water from the lower reservoir.

Various arrangements are possible for higher and lower reservoirs:

- i) Both reservoirs in a single river
- ii) Two reservoirs on two separate rivers close to each other and flowing at different elevations
- iii) Higher reservoir an artificially constructed pool and the lower reservoir on natural river
- iv) The lower reservoir in a natural lake while the higher is artificial

Another way of classifying is as pure pumped storage scheme and mixed plant scheme (total generation > pumping and higher reservoir on a natural system).

The most important basis of pumped storage plant is the relative arrangements of turbines and pumps

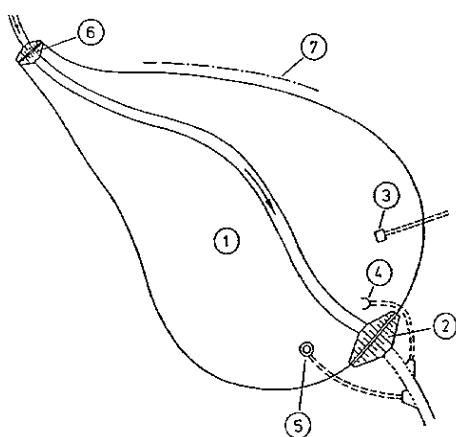
- four units -pump, motor, generator, turbine
- three units- pump, generator, turbine
- two units-generator, turbine > reversible pump-turbine installation

4.3 Storage and Pondage

Storage is providing to balance seasonal fluctuation by building reservoir dams. Pondage is provided through balancing reservoir or forebay for short term fluctuations (daily or hourly)

1.- Reservoir (storage) capacity

Reservoir capacity is determined by means of mass curve procedure of computing the necessary capacity corresponding to a given inflow and demand pattern. Reservoir capacity has to be adjusted to account for the dead storage, evaporation losses and carry over storage.



Storage	(1)
Dam	(2)
Appurtenant Structure: Intake and Spillage Structures:	
Spillage Structure:	Spillway (5)
Intake:	Service Intake (3) and Bottom Outlet (4)
Surveillance Structure (in Dam, in Foundation as well as Valley Sides)	
Diversion	(6)
Service Road	(7)

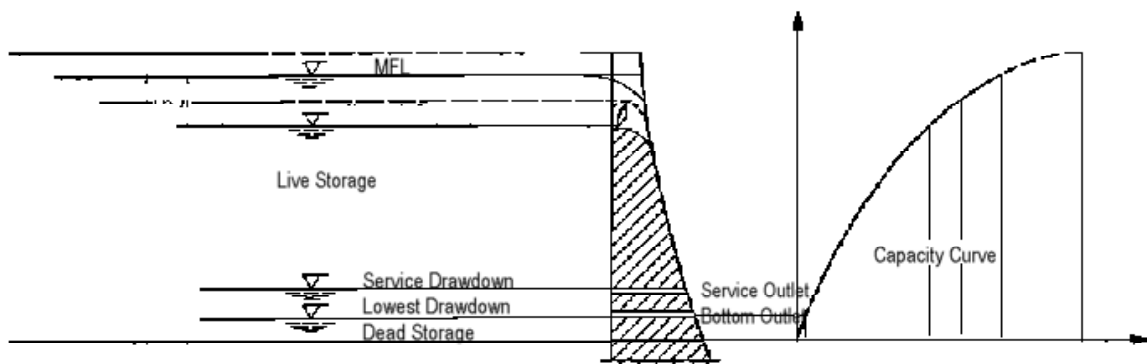


Figure 4.8: Reservoir Components

Dead storage

This is a storage capacity of the reservoir provided to accommodate the deposition of silt in the reservoir. It is expected that the dead storage space will eventually fill up with sediment at which time one says the dam has served its full purpose. The life of a reservoir is dependent on the silting capacity of the reservoir. Provisions for flushing out silt through deep seated bottom outlets/sluices is made in most dams. However this has a limited effectiveness.

Evaporation Loss

Provision should be made for evaporation since it is an important loss item actual evaporation rate depends upon location & meteorological factors. In arid and semi-arid regions at least 2 to 2.5m of depth should be added as a rule of thumb.

Carry over storage

Sometimes it may be required to carry over some of the live storage to the next year as a safety measure. This carry over storage is determined by analyzing the storage requirement for a sequence of two or three consecutive dry years.

2) Pondage Capacity

Pondage is provided to cater for short term fluctuations.

- For run-of-river plants the pondage is provided by the main weir on its side.
- For diversion canal plants, the pondage is provided at the end of the canal in the form of forebay reservoir.

Reasons for short term fluctuations are:

- i) Sudden increase or decrease in load on the turbine. The pondage would provide the extra water when needed and retain excess water when not needed.
- ii) The load and thus the water demand may be steady but the supply may undergo a change. Breaches in the supply canal may lead to this.

Pondage capacity is determination for varying inflow is similar to storage capacity determination.

If hourly inflows for a typical day are known, one can calculate the average hourly requirement and determine the total maximum cumulative departures from the average over a 24 hour period. This will then be the pondage needed to equalize the daily flow fluctuations.

4.4 Intakes and Head Race

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.

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. Provision of trash racks at the entrance achieves this function.
- iv) To exclude heavy sediment load of the river from entering the conveyance system. Special devices such as silt traps and silt excluders are used to control & trap the silt.

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

Intakes of special type

Settling Basins

The water drawn from a river for power generation may carry suspended sediment particles. This silt load may be composed of hard abrasive materials such as *quartz* and will cause damage or wear to the hydro-mechanical elements like turbine runners, valves, and penstocks. To remove this material a structure called *settling basin* should be constructed, where the velocity of the flow will be reduced resulting in settling out of the material, which has to be periodically or continuously flushed out.

In order to satisfy the requirement for a good hydraulic performance the basin is divided into three main zones: *inlet zone*, *settling zone*, and *outlet zone*.

Inlet Zone:

The main function of the inlet is to gradually decrease the turbulence and avoid all secondary currents in the basin. This is achieved by decreasing the flow velocity through gradually increasing the flow cross-section, i.e., by providing gradual expansion of the width and depth (*see figure 4.9*).

To achieve optimum hydraulic efficiency and effective use of the settling zone, the inlet needs to distribute the flow uniformly over the cross-section of the basin. To achieve uniform flow distribution, the following techniques, in addition to the provision of gradual expansion, may be adopted at the inlet zone:

- Use of submerged weir
- Use of baffles
- Use of slotted walls

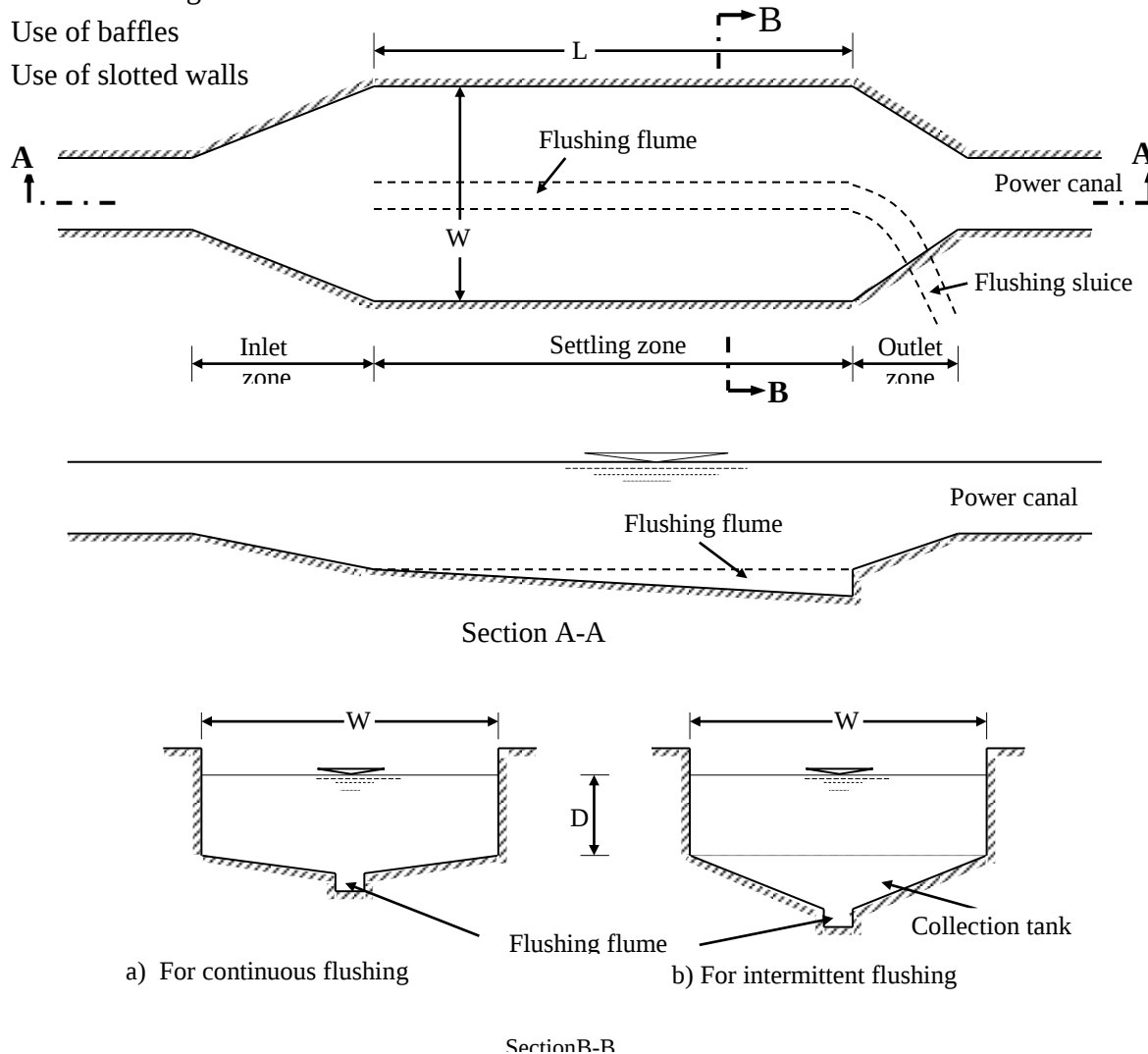


Figure 4.9 Settling basin

Settling Zone:

This is the main part of the basin where settling of the suspended sediment is supposed to take place. The dimensions of this zone can be determined through calculations.

Outlet Zone:

This is a kind of transition provided following the settling zone to facilitate getting back the flow into the conveyance system with the design velocity by gradually narrowing the width and depth. The outlet transition may be more abrupt than the inlet transition.

Note: The cross-section of a settling basin is usually tapered at the bottom forming a sediment-collecting flume, built with a gradient in the direction of flow.

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.

4.5 Tunnels

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 follows 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.

Classification of Tunnels

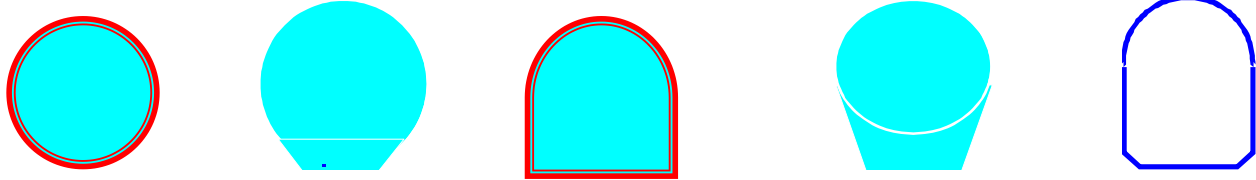
In addition to the above classification tunnels may be classified on the basis of shape, alignment and design aspects.

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 roof. The horse-shoe shape is the most popular and convenient from the point of view of construction.

Commonly adopted shapes:



a) Circular shape

b) Horseshoe shape

c) D-shape

Figure 4.10: 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.

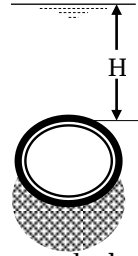
Lining: Lining is a protective layer of concrete, R.C. or steel on the inner surface of the tunnel and it is an important aspect in classification of tunnels. Thus tunnels may be lined, unlined or partially lined. Tunnels in good, sound rock may left unlined.

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



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 aggressively 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 $H \leq \gamma_r h_r$

Using a factor of safety of η , $H = \frac{\gamma_r h_r}{\eta} (m)$

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 passability limit the maximum size.

Minimum size of Tunnel: Circular, 1.8 m ϕ

Rectangular, 2m x 1.6m.

4.6 Water Hammer, Surge Tanks and Forebays

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 with in 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 pressure only fractures 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).

Surge Tanks

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 4.10*).

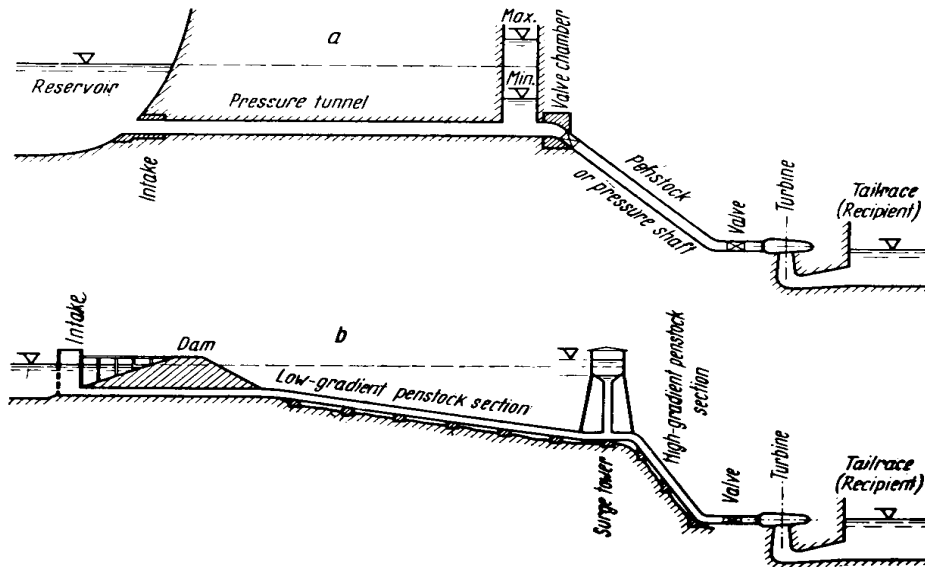


Figure 4.10: Typical Arrangements of Surge Tanks

Functions 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.

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 4.11)
 - over ground surge tank (Free standing surge tank, see Figure 4.10)

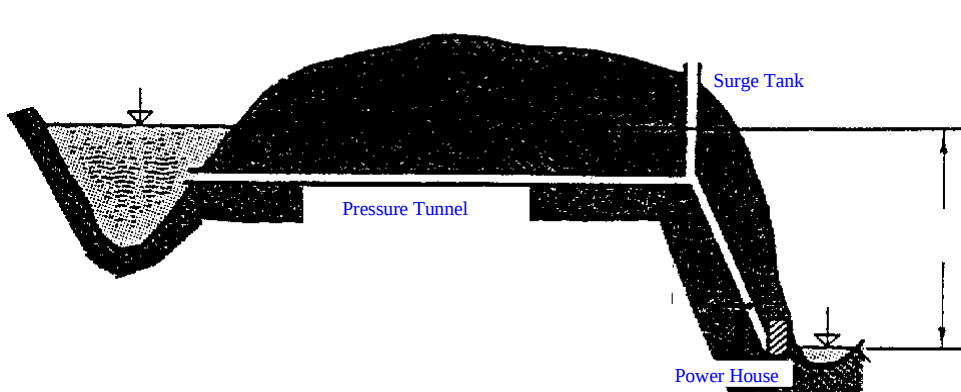


Figure 4.11: Underground Surge Tank and Power House

- c) Location in the hydraulic system
- Upstream surge tank (u/s to the power house) on the headrace tunnel (see Figures 4.11 and 4.12).
 - Downstream surge tank on the tailrace tunnel (see Figure 4.12).

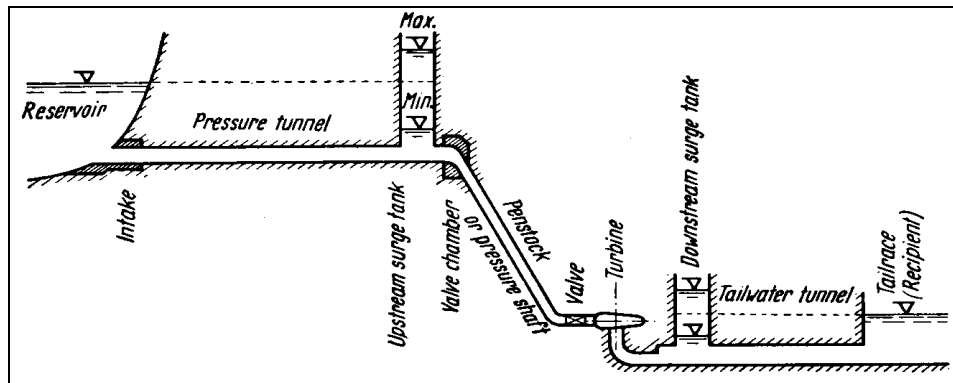
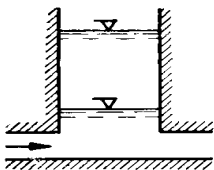
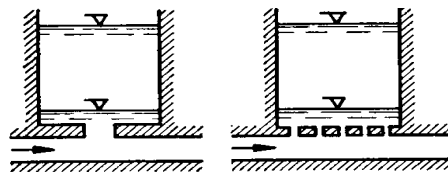


Figure 4.12: Downstream Surge Tank

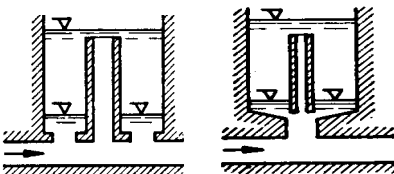
- d) Hydraulic functioning & cross-sectional shape
- The most useful classification is on the basis of their shape, which also determines their hydraulic characteristic. A usually followed classification is as follows:
- ♦ Simple surge tanks
 - ♦ Restricted orifice (or throttled) surge tanks
 - ♦ Differential surge tanks
 - ♦ Surge tanks with expansion chambers and others



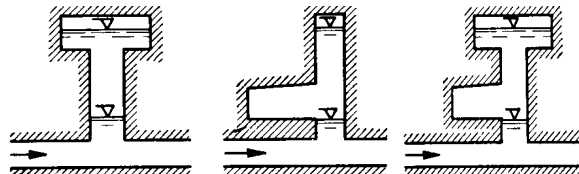
a) Simple surge tank



b) Restricted orifice surge tank



c) Differential surge tank



d) Surge tanks with expansion chambers

Figure 4.13: Surge Tank Types

Forebays

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 4.14 shows typical forebay.

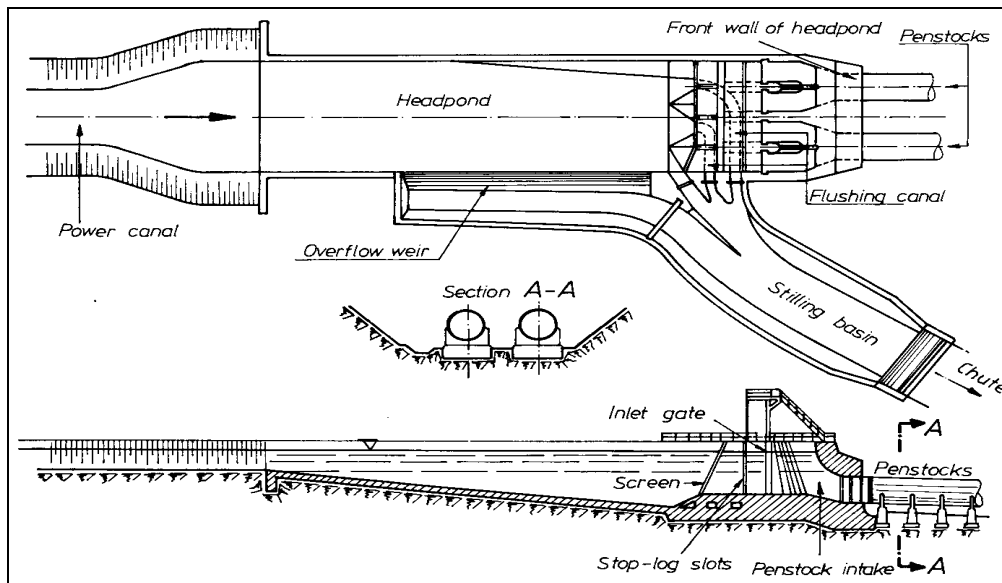


Figure 4.14 General Arrangement of a Forebay.

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.

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.

4.8 PENSTOCK

The penstock is high pressure pipeline between forebay (surge tank or reservoirs) and the turbine. The design principle of penstocks is 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 find two or more turbines through a special branching pipe called Manifold.

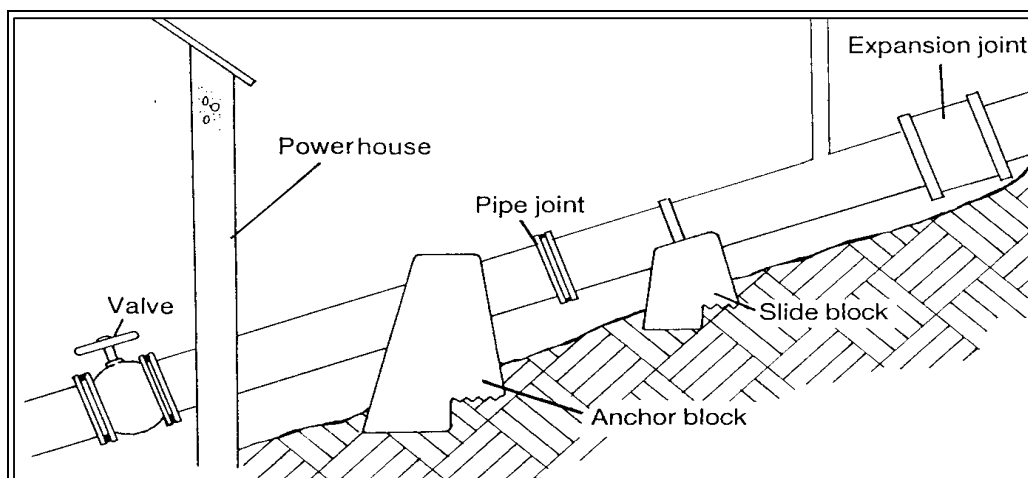


Figure 4.15: Components of a penstock

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 steep slopes
- 3- Difficulty in maintenance
- 4- Expensive for large 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.

Disadvantages

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

- 3- Stability is insured with proper anchorage

When the situation warrants, partly buried system, may be adopted this 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 variation is moderate.
- b) Semi- rigid pipes: Here each member of the pipe line is fixed at one end 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. Further more, 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.

Hydraulics

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.

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

Head losses

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

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

	K
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: $H = H_g - \sum \text{losses}$ 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)

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 is large, small h_f , greater output , larger initial cost .

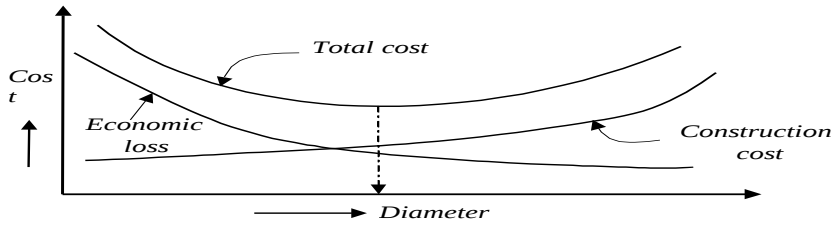


Figure 4.16: Economical Diameter of Penstock

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 e 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)

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

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 water hammer analysis

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

For instant closure i.e. $t_c \leq \frac{2L}{C}$

$$h = \frac{Cv_o}{g}$$

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, $p = H$

$$t = \frac{pD}{2\sigma}$$

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

$N =$ 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 piper where $\frac{D}{t} < 20$, the following formula gives sufficient accuracy

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

The ASME gives the formula for thickness as $t = \frac{pr}{\sigma_a\phi - 0.6P} + 0.15$

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

Introducing a S.F. η , $Pe = \frac{1}{\eta} 2 \left(\frac{t}{D} \right)^3$ $n=2$ for burried pipes; $n=4$ for exposed pipes

$$t = D \sqrt[3]{\frac{\eta Pe}{2E}}$$

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

4.8 Hydraulic Machines (Hydraulic turbines and their selection)

General

Hydraulic turbines may be considered as hydraulic motors or prime movers of a water power development, which convert water energy (hydropower) in to mechanical energy (shaft power). The shaft power developed is used in running electricity generators directly coupled to the shaft of the turbine, thus producing electrical power.

Classification

All types of turbines basically fall in to two categories impulse and reaction turbines.

Impulse turbine: All the available potential energy is converted in to kinetic energy with the help of contracting nozzle/s. The water after impinging on the curved vanes or bucket is discharged freely to the downstream channel (eg. Pelton wheel)

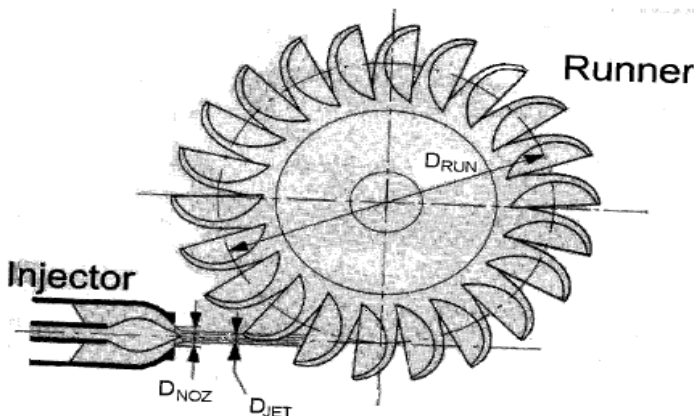


Figure 4.17: Example of Pelton turbine arrangement (2 nozzles)

Reaction turbines: In this type the water enters the turbine in a circumferential direction in to the scroll case and moves into the runner through a series of guide vanes, called wicket gates. The available energy partly converted to kinetic energy & substantial magnitude remains in the form of pressure energy (eg. Francis, Kaplan, Propeller, Bulb, etc)

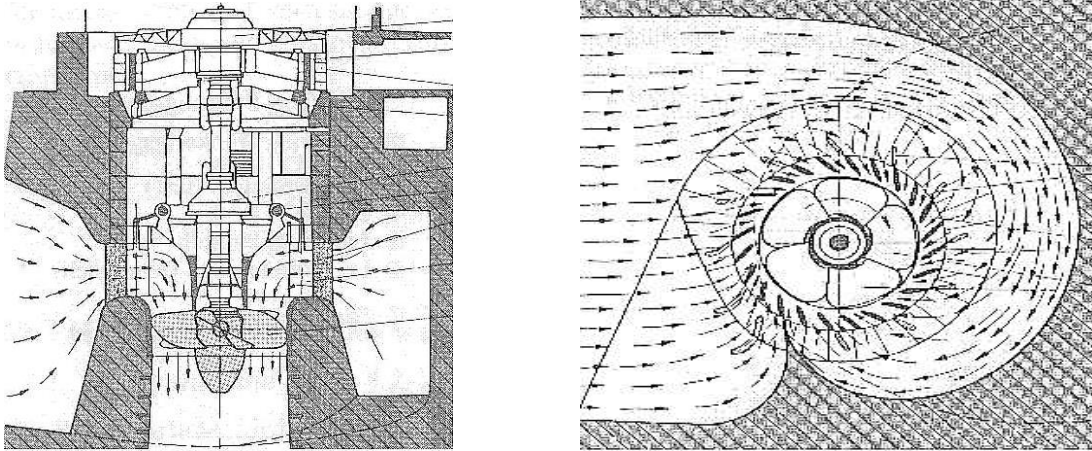


Figure 4.18: Representation of flow pattern in Kaplan turbine

Turbines may also be classified according to the main direction of flow of water in the runner as

- ♦ Tangential flow turbine (pelton wheel)
- ♦ radial flow “ (Francis, Thomson, Girard)
- ♦ Mixed flow “ (modern Francis)
- ♦ Axial flow turbine of fixed blade (propeller) or movable blade (Kaplan or bulb) type.

Furthermore, turbines may be classified based on head, discharge, speed, specific speed.

4.8.1 Characteristics of Turbines

Specific speed: is useful parameter for the selection of turbine for a given condition: It is defined as the speed at which a geometrically similar runner would rotate if it were so proportioned that it would develop 1 Kw when operating under a head of 1m , and expressed as (from dimensional analysis)

$$N_s = N \frac{\sqrt{P}}{H^{5/4}}$$

where N_s = Specific speed

N = rotational speed. (rpm)

P = Power developed (kw)

H = effective head (m)

Turbine or synchronous speed: Since turbine & generator are fixed, the rated speed of the turbine is the same as synchronous speed of the generator. The speed N , for synchronous running is given by :

$$N = 120 \frac{f}{p}$$

Where f = frequency cycle/sec (50-60 Hz c/s)

p = number of poles → (divisible by 4 for head up to 200 m)
(divisible by 2 for head above 200 m)

The speed of a turbine is an important parameter of design. The higher the speed, the smaller the diameter of the turbine runner & the cheaper the generator coupled to the turbine. High speed, however, makes a turbine more susceptible to cavitation.

The ratio of the peripheral speed of the bucket or vanes at the nominal diameter, D , to the theoretical velocity of water under the effective head, H , acting on the turbine is called the speed factor or peripheral coefficient, ϕ .

$$\phi = \frac{v}{\sqrt{2gH}} = \frac{\omega r}{\sqrt{2gH}}$$

but T in rad/sec; $\omega = \frac{2\pi N}{60}$ $r = D/2$

Therefore, $\phi = \frac{\pi DN}{60\sqrt{2gH}} = \frac{DN}{84.6\sqrt{H}}$ D and H in m; N in rpm

The following table suggests appropriate values of ϕ , which give the highest efficiencies for any turbine, the head & specific speed ranges & the efficiencies of the three main types of turbine.

Type of rammer	ϕ	Ns	H (m)	Efficiency (%)
Impulse	0.43 – 0.48	8-17 17 17-30	>250	85-90 90 90-82
Francis	0.6 – 0.9	40 – 130 130-350 350-452	25-450	90-94 94 94-93
Propeller	1.4-2.0	380-600 600-902	<60	94 94-85

Thus in general

- Pelton turbines are used for high heads & low discharges
- Francis types are used for medium & high head plants (has adjustable guide vanes but the runner is a disc with fixed passage)
- Propeller & Kaplan (Kaplan has adjustable blades) types are used for lows head plants with large discharges.

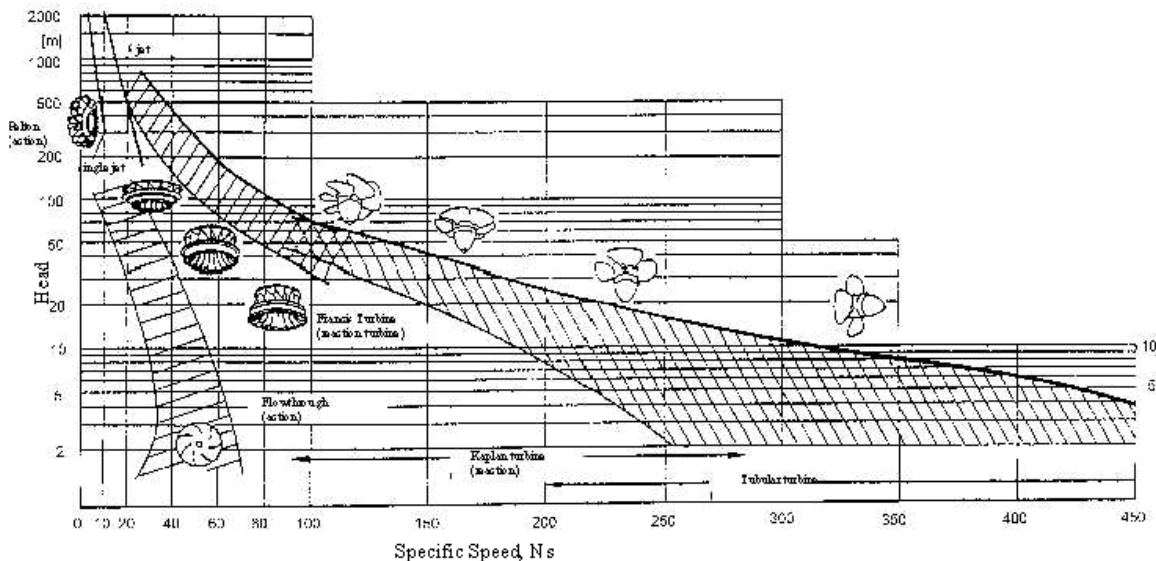


Figure 4.19: Application of turbine based on head and specific speed

4.8.2 Procedure in preliminary selection of Turbines

1. From design Q and H, calculate approximate P that can be generated , $P = \eta \gamma Q H$
2. From $N = 120 \frac{f}{p}$ calculate N (or assume) & computer N_s . From this, the type of turbine can be suggested
3. Calculate D from: $\phi = \frac{DN}{84.6\sqrt{H}}$

If D is found to be too large, either N can be increased or more units may be adopted. For approximate calculations of runner diameter; the following empirical formula may be used (Mosony)

$$D = a \left(\frac{Q}{N} \right)^{1/3} \quad D \text{ in m; } Q \text{ in m}^3/\text{s; } N \text{ in rpm}$$

a = 4.4 for Francis & propeller; a = 4.57 for Kaplan.

or $D = \frac{7.1\sqrt{Q}}{(N_s + 100)^{1/3} H^{1/4}}$ for propeller, H in m

Nominal diameter, D , of pelton wheel $D = 38 \sqrt{\frac{H}{N}}$

$$dj = 0.542 \sqrt{\frac{Q}{H}}$$

(dj is diameter of the jet for N=0.45)

Jet ratio given by $m = \frac{D}{d_j}$, is important parameter in design of pelton wheels.

Number of buckets, $n_b = 0.5 m + 15$ (good for $6 < m < 35$)

It is not uncommon to use a member of multiple jet wheels mounted on the same shaft so as to develop the required power.

A hydraulic turbine (runner) is designed for optimum speed & maximum efficiency at design head. But in reality, head and load conditions change during operation & it is extremely important to know the performance of the unit at other heads. This is furnished by manufacturer's curve.

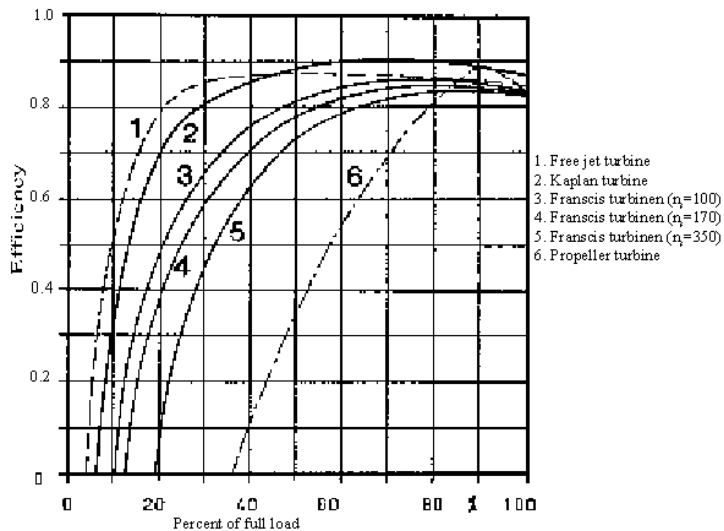


Figure 4.20 Variation of efficiency w.r.t. % of full load for various turbines

4.8.3 Runaway Speed

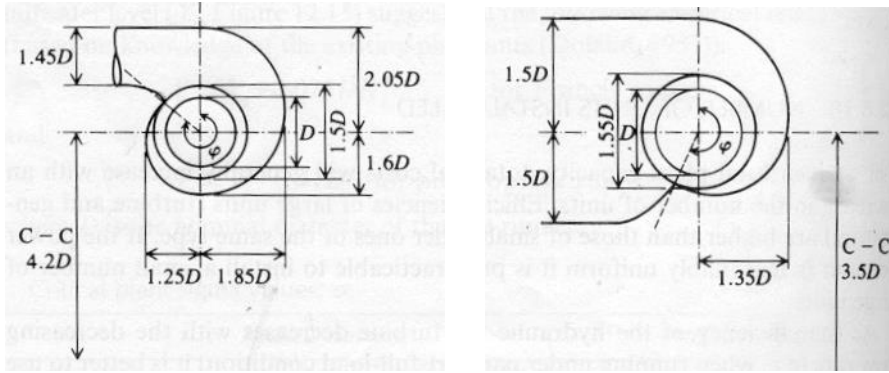
If the external load on the machine suddenly drops to zero (sudden rejection) and the governing mechanism fails at the same time, the turbine will tend to race up to the maximum possible speed, known as runaway speed. This limiting speed under no-load, maximum-flow must be considered for safe design.

Type of runner	Runaway speed (% of normal speed)	Acceptable head variation (% of design head)	
		Minimum	Maximum
Impulse (Renton)	170 - 190	65	125
Fiancés	200 - 220	50	150
propeller	250 - 300	50	150

Runaway speed and acceptable head variations

4.8.4 Turbine scroll case

A scroll case is the conduit directing the water from the intake or penstock to the runner in reaction type turbine installation (in case of impulse wheels a casing is usually provided only to prevent splashing of water & lead water to the tail race). A spiral shaped scroll case of the correct geometry ensures even distribution of water around the periphery of the runner with the minimum possible eddy formations.



a) Francis turbine with steel spiral case b) Propeller turbine with partial spiral
Figure 4.21: Recommended dimensions of scroll casings (a) full spiral b) partial spiral

These kinds of spiral case will generally used in medium and high head installations where discharge requirement is low. See Figure 6.4 a). Spiral cases with $320 < v < 340$ are also considered full.

The design of the shape of the spiral case is governed by the flow requirements. Initial investigation should be based on the following assumptions:

- spiral case of constant height
- an evenly distributed flow in to the turbine
- no friction losses

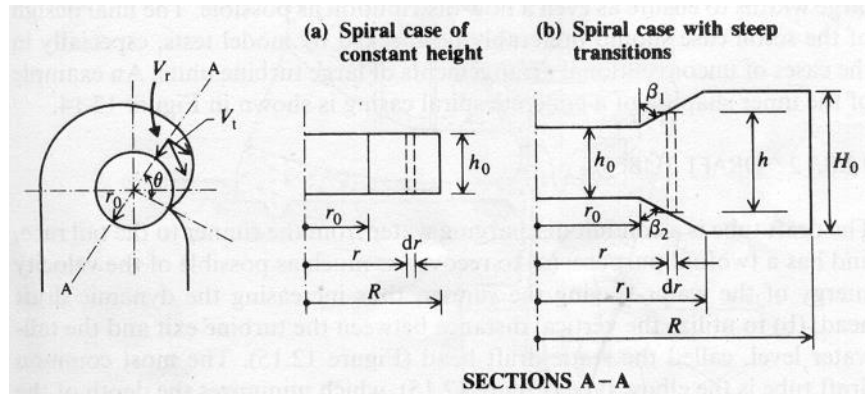


Figure 4.22: Typical cross-sections of spiral case

Referring to Fig 6.6 (a) the discharge in section of spiral case defined by angle 2 is

$$q = \frac{Q\theta}{2\pi} \quad \text{where } Q \text{ is the total discharge to the runner.}$$

$$v_t = \frac{k}{r} \quad \text{where} \quad k = 30 \frac{\eta g H}{N\pi} \quad (\text{from the basic Euler equation for the power absorbed by the machine})$$

and the discharge through two strip dq is given by

$$dq = v_t h_0 dr = k h_0 dr \quad \therefore \quad q = \int_{r_0}^R k h_0 \frac{dr}{r} = \frac{Q\theta}{2\pi} \text{ or } \ln \frac{R}{r_0} = \frac{Q\theta}{2\pi k h_0}$$

This shows for given vortex strength, k , a definite relationship exist between Q & R .

The most economical design of a power station substructure and the narrowest spiral case can be obtained by choosing a rectangular section adjoining the guide vanes (entrance ring) by step transition (symmetrical or asymmetrical) as shown in b.

$$h = h_0 + \alpha(r - r_0) \quad \text{where} \quad \alpha = \cot \beta_1 + \cot \beta_2$$

$$\frac{Q\theta}{2\pi k} = \int_{r_0}^{r_1} h \frac{dr}{r} + \int_{r_1}^R H_0 \frac{dr}{r}$$

Replacing and integrating

$$\frac{Q\theta}{2\pi k} = h_0 - \alpha r_0 \ln \left(\frac{r_1}{r_0} \right) + H_0 - h_0 + H_0 \ln \left(\frac{R}{r_1} \right)$$

Knowing r_1 from $r_1 = \left(\frac{H_0 - h_0}{\alpha} \right) + r_0$, the value of R defining the shape of the spiral case can be determined. The

height H_0 at any angle 2 may be assumed to be linearly increasing from h_0 at the nose towards the entrance. Shape at various 2 is determined by assuming existence of uniform velocity equal to entrance velocity,

$$v_0 \cong 0.2\sqrt{2gH} \text{ and } q_i = \frac{Q\theta_i}{2\pi}$$

$$A_i = \frac{q_i}{v_0} = 0.18 \frac{q\theta_i}{\sqrt{H}} \quad \text{Area of cross-section at angle } 2_i$$

4.8.5 Draft Tubes

A draft tube is a conduit discharging water from the turbine runner to the tailrace. It is employed in conjunction with reaction type turbines, and has twofold purposes:

- ♦ To recover as much as possible of the velocity energy of the water leaving the runner, which otherwise would have gone to waste as an exit loss, thus increasing the dynamic draft head.
- ♦ To utilize the vertical distance between the turbine exit and the tail-water level, called the static draft head. In other words, to allow the turbine to be set at higher elevation without losing the advantage of elevation difference.

The most common is elbow type which minimizes the depth of substructure compared to vertical one, it also has a desirable effect in directing the flow in the direction of the tail water.

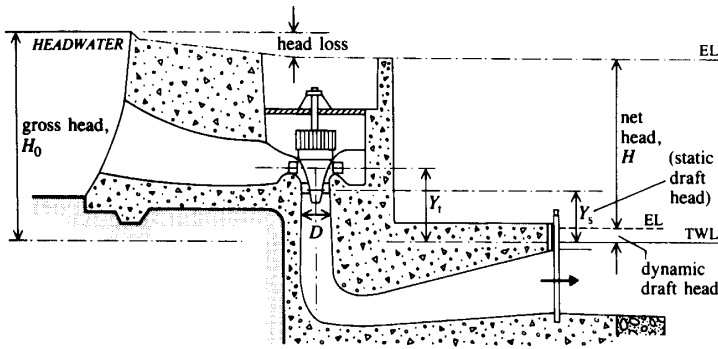


Figure 4.23: Elbow-type draft tube

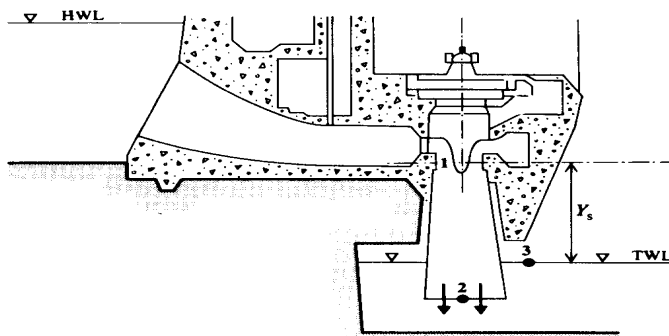


Figure 4.24: Straight conical draft tube

The straight conical draft tubes are the simplest in design and the most efficient type, but they are rarely used in actual practice. This is because, for effective recovery of velocity head, the outlet section has to be many times the inlet section of the draft tube. For smooth eddy-free flow (flow with no separation), the angle of flare of the tube has to be limited to 4 to 8 degrees. Hence, a considerable long tube is necessary to achieve the desired result. This increases the depth of excavation of the substructure, making it uneconomical and unsuitable from cavitation view point.

The elbow-type draft tube is often adopted, because of the following advantages it offers over the conical type:

- ♦ Minimizes the required depth of excavation
- ♦ Directs the flow in the direction of the tail-water flow
- ♦ Allows the provision of gate at the outlet of the tube which can facilitate the de-watering of the turbine for repairs, if necessary.

However from constructional point of view, the elbow draft tube presents more problems. Furthermore, the change of shape in the elbow naturally increases the turbulent losses in the draft tube.

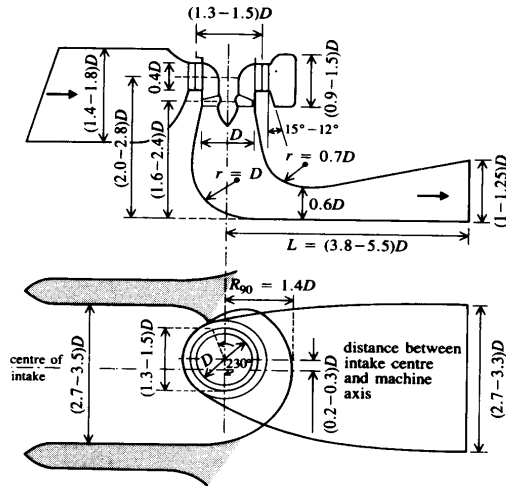


Figure 4.25: Recommended dimensions of an elbow-type draft tube (after Mosonyi)

Elbow type divided in to 3 – parts: vertical, bend, almost horizontal. Draft tubes of large Q & small H are mutually designed by model tests.

Between 1 & 3 in Figure 4.23,
$$Y_s + \frac{P_1}{\gamma} + \frac{v_1^2}{2g} = \frac{P_a}{\gamma} + \frac{v_2^2}{2g} + H_L$$

Therefore,
$$\frac{P_1}{\gamma} = \frac{P_a}{\gamma} - Y_s - \left(\frac{v_1^2}{2g} - \frac{v_2^2}{2g} - H_L \right)$$

$$H_d = \eta_d \left(\frac{v_1^2}{2g} - \frac{v_2^2}{2g} \right)$$

where η_d = efficiency of the draft tube

In order to avoid cavitation at the exit from the runner the condition $\frac{P_1}{\gamma} > \frac{P_v}{\gamma}$. (Saturated vapor pressure is around 0.3 on of water absolutes)

4.8.6 Cavitation in Turbine & Turbine Setting

Cavitation result is pitting, vibration & reduction in efficiency & is certainly undesirable. Cavitation may be avoided by suitably designing, installing, and operating the turbine in such a way that the pressures with is the units are above the vapor pressure of water.

Refering the previous Figures, Y_s is the most critical factor in the installation of reaction turbines.

$$\sigma = \left(\frac{H_a - H_v - Y_s}{H} \right) \quad \Phi = \text{cavitation coefficient or plant sigma}$$

$H_a - H_v = H_b$ = barometric pressure (10.1 @ sea level)

H = effective head.

$Y_{s, \max} = H_b - \Phi_c H$ (Thoma's formula, bottom of turbine setting)

If Y_s is negative runners must be below TWL. Where D_c is the minimum (critical) value of Φ at which cavitation occur.

Francis runners						Propeller runners			
N_s	75	150	225	300	375	375	600	750	1,5
Φ_c	0.02	0.10	0.23	0.40	0.64	0.64	0.8	1.5	3.5

The above may be approximated by

$$\sigma_c = 0.0432 \left(\frac{N_s}{100} \right)^2 \quad \text{for Francis}$$

$$\sigma_c = 0.28 + 0.0024 \left(\frac{N_s}{100} \right)^3 \quad \text{For propeller}$$

With an increase by 10% for Kaplan turbines.

The preliminary calculation for the elevation of the distributor above the TWL, Y_t is

$$Y_t = Y_s + 0.025 D N_s^{0.34} \quad \text{for Francis}$$

$$Y_t = Y_s + 0.025 D \quad \text{For propeller}$$

Where D is the nominal diameter of the runner.