LECTURE NOTES ON

IRRIGATION STRUCTURES AND WATER POWER ENGINEERING

UNIT-I
GRAVITY DAMS-EARTH DAMS

A **gravity dam** is a dam constructed from concrete or stone masonry and designed to hold back water by primarily utilizing the weight of the material alone to resist the horizontal pressure of water pushing against it. Gravity dams are designed so that each section of the dam is stable, independent of any other dam section

FORCES ACTING ON GRAVITY DAM:

In the design of a dam, the first step is the determination of various forces which acts on the structure and study their nature. Depending upon the situation, the dam is subjected to the following forces:

1. Water pressure
2. Earthquake forces
3. Silt pressure
4. Wave pressure

Ice pressure

6. Self weight of the dam.

The forces are considered to act per unit length of the dam.

For perfect and most accurate design, the effect of all the forces should be investigated. Out of these forces, most common and important forces are water pressure and self weight of the dam.

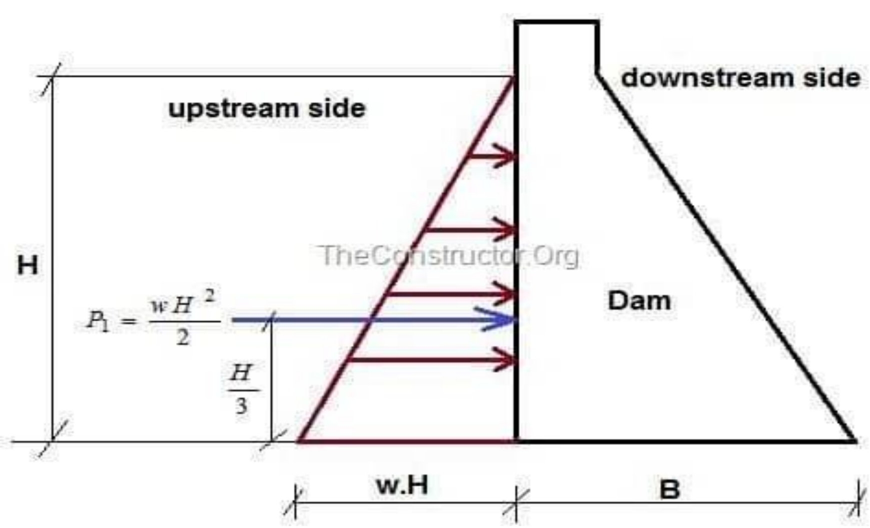
1. Water Pressure

Water pressure may be subdivided into the following two categories:

I) External water pressure:

It is the pressure of water on the upstream face of the dam. In this, there are two cases:

(I) Upstream face of the dam is vertical and there is no water on the downstream side of the dam (figure 1).



The total pressure is in horizontal direction and acts on the upstream face at a height $\frac{H}{3}$ from the bottom. The pressure diagram is triangular and the total pressure is given by $P_1 = \frac{wH^2}{2}$

Where w is the specific weight of water. Usually it is taken as unity.

H is the height upto which water is stored in m.

(ii) Upstream face with batter and there is no water on the downstream side (figure 2).

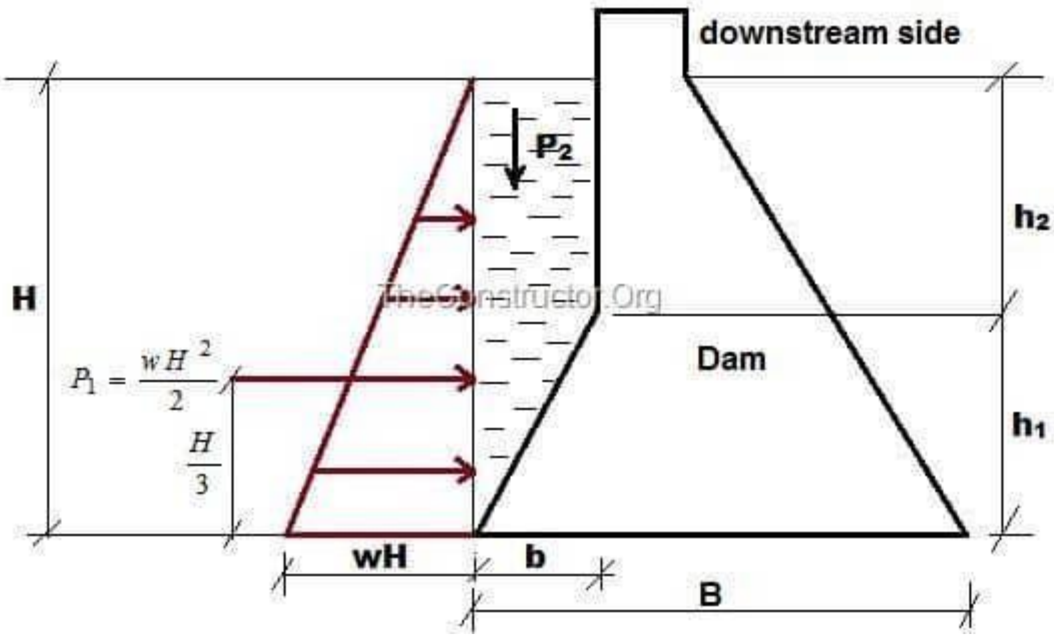


Figure 2

Here in addition to the horizontal water pressure P_1 as in the previous case, there is vertical pressure of the water. It is due to the water column resting on the upstream sloping side.

The vertical pressure P_2 acts on the length 'b' portion of the base. This vertical pressure is given by

$$P_2 = (b \times h_2 \times w) + \left(\frac{1}{2} b \times h_1 \times w\right)$$

Pressure P_2 acts through the centre of gravity of the water column resting on the sloping upstream face.

If there is water standing on the downstream side of the dam, pressure may be calculated similarly. The water pressure on the downstream face actually stabilizes the dam. Hence as an additional factor of safety, it may be neglected.

II) Water pressure below the base of the dam or Uplift pressure

When the water is stored on the upstream side of a dam there exists a head of water equal to the height upto which the water is stored. This water enters the pores and fissures of the foundation material under pressure. It also enters the joint between the dam and the foundation at the base and the pores of the dam itself. This water then seeps through and tries to emerge out on the downstream end. The seeping water creates hydraulic gradient between the upstream and downstream side of the dam. This hydraulic gradient causes vertical upward pressure. The upward pressure is known as uplift. Uplift reduces the effective weight of the structure and consequently the restoring force is reduced. It is essential to study the nature of uplift and also some methods will have to be devised to reduce the uplift pressure value.

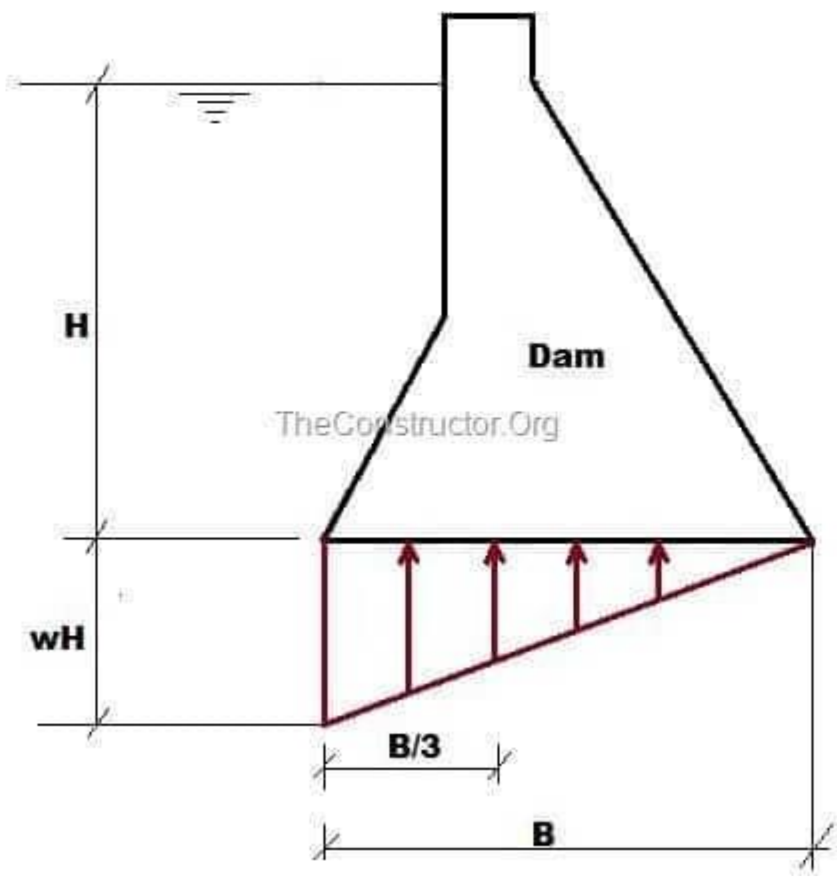


Figure 3

With reference to figure 3, uplift pressure is given by $P_u = \frac{wH \times B}{2}$

Where P_u is the uplift pressure, B is the base width of the dam and H is the height upto which water is stored.

This total uplift acts at $\frac{B}{3}$ from the heel or upstream end of the dam.

Uplift is generally reduced by providing drainage pipes or holes in the dam section.

Self weight of the dam is the only largest force which stabilizes the structure. The total weight of the dam is supposed to act through the centre of gravity of the dam section in vertically downward direction. Naturally when specific weight of the material of construction is high, restoring force will be more. Construction material is so chosen that the density of the material is about 2.045 gram per cubic meter.

2. Earthquake Forces

The effect of earthquake is equivalent to acceleration to the foundation of the dam in the direction in which the wave is travelling at the moment. Earthquake wave may move in any direction and for design purposes, it is resolved into the vertical and horizontal directions. On an average, a value of 0.1 to 0.15g (where g = acceleration due to gravity) is generally sufficient for high dams in seismic zones. In extremely seismic regions and in conservative designs, even a value of 0.3g may sometimes by adopted.

Vertical acceleration reduces the unit weight of the dam material and that of water is to $(1 - k_v)$ times the original unit weight, where k_v the value of g accounted against earthquake forces, i.e. 0.1 is when 0.1g is accounted for earthquake forces. The horizontal acceleration acting towards the reservoir causes a momentary increase in water pressure and the foundation and dam accelerate towards the reservoir and the water resists the movement owing to its inertia. The extra pressure exerted by this process is known as hydrodynamic pressure.

3. Silt Pressure

If h is the height of silt deposited, then the forces exerted by this silt in addition to the external water pressure, can be represented by Rankine formula

$P_{\text{silt}} = \frac{1}{2}\gamma_s h^2 k_a$ acting at $\frac{h}{3}$ from the base.

Where,

k_a = coefficient of active earth pressure of silt = $\frac{1 - \sin\phi}{1 + \sin\phi}$

ϕ = angle of internal friction of soil, cohesion neglected.

γ_s = submerged unit weight of silt material.

h = height of silt deposited.

4. Wave Pressure

Waves are generated on the surface of the reservoir by the blowing winds, which exert a pressure on the downstream side. Wave pressure depends upon wave height which is given by the equation

$h_w = 0.032\sqrt{PV} + 0.763 - 0.271 \times (F)^{1/4}$ for F < 32 km, and

$h_w = 0.032\sqrt{VF}$ for F > 32 km

Where h_w is the height of water from the top of crest to bottom of trough in meters.

V – wind velocity in km/hour

F – fetch or straight length of water expanse in km.

The maximum pressure intensity due to wave action may be given by

$P_w = 2.4\gamma_w h_w$ and acts at $\frac{h_w}{2}$ meters above the still water surface.

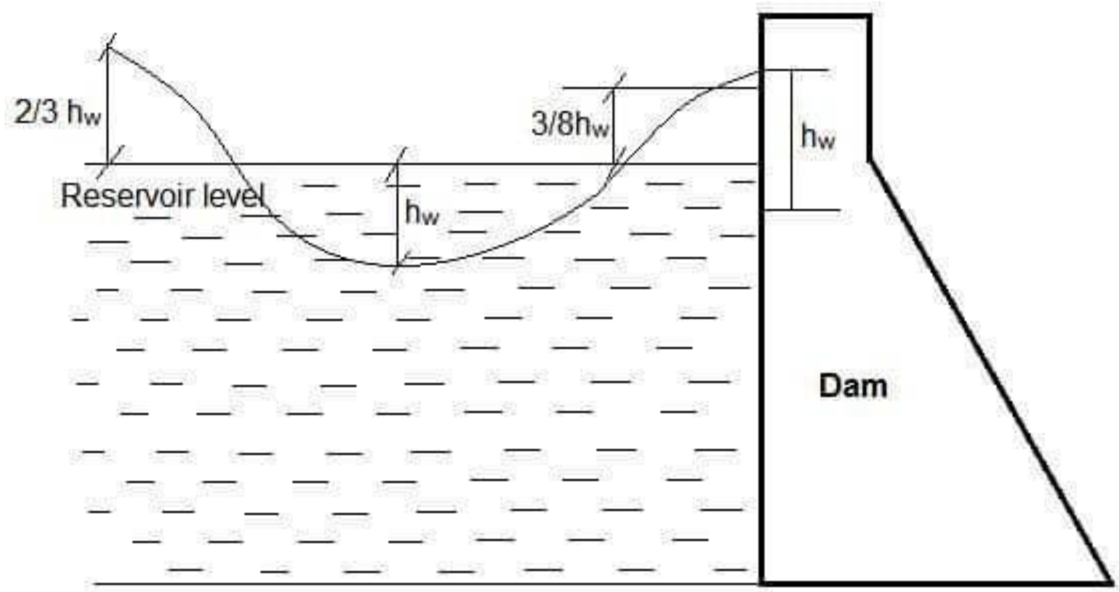


Figure 4

The pressure distribution may be assumed to be triangular of height $\frac{5h_w}{3}$ as shown in figure 4. Hence total force due to wave action P_w

= $\frac{1}{2} \times (2.4 \gamma_w h_w) \times \frac{5}{3} h_w$ acting at $\frac{3}{8} h_w$ above the reservoir surface.

5. Ice Pressure

The ice which may be formed on the water surface of the reservoir in cold countries may sometimes melt and expand. The dam face is subjected to the thrust and exerted by the expanding ice. This force acts linearly along the length of the dam and at the reservoir level. The magnitude of this force varies from 250 to 1500 kN/sq.m depending upon the temperature variations. On an average, a value of 500 kN/sq.m may be taken under ordinary circumstances.

6. Weight of dam

The weight of dam and its foundation is a major resisting force. In two dimensional analysis of dam

FAILURES OF GRAVITY DAM

Failure of gravity dam occurs due to overturning, sliding, tension and compression. A gravity dam is designed in such a way that it resists all external forces acting on the dam like water pressure, wind pressure, wave pressure, ice pressure, uplift pressure by its own self-weight. Gravity dams are constructed from masonry or concrete. However, concrete gravity dams are preferred these days and mostly constructed.

The advantage of gravity dam is that its structure is most durable and solid and requires very less maintenance.

Causes of failure of a Gravity Dam:

A gravity dam may fail in following modes:

1. Overturning of dam about the toe
2. Sliding – shear failure of gravity dam
3. Compression – by crushing of the gravity dam
4. Tension – by development of tensile forces which results in the crack in gravity dam.

Overturning Failure of Gravity Dam:

The horizontal forces such as water pressure, wave pressure, silt pressure which act against the gravity dam causes overturning moments. To resist this, resisting moments are generated by the self-weight of the dam.

If the resultant of all the forces acting on a dam at any of its sections, passes through toe, the dam will rotate and overturn about the toe. This is called overturning failure of gravity dam. But, practically, such a condition does not arise and dam will fail much earlier by compression.

The ratio of the resisting moments about toe to the overturning moments about toe is called the factor of safety against overturning. Its value generally varies between 2 and 3.

Factor of safety against overturning is given by

FOS = sum of overturning moments/ sum of resisting moments

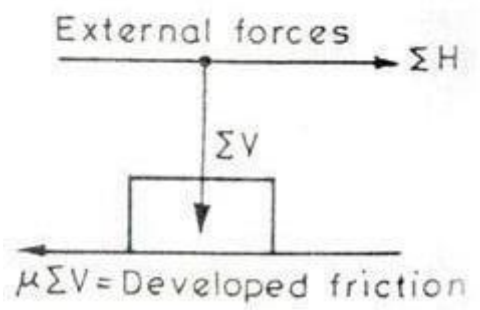


Fig:sum of external horizontal forces greater than vertical self-weight of dam (overacting, sliding occurs)

Sliding Failure of Gravity Dam: When the net horizontal forces acting on gravity dam at the base exceeds the frictional resistance (produced between body of the dam and foundation), The failure occurs is known as sliding failure of gravity dam.

In low dams, the safety against sliding should be checked only for friction, but in high dams, for economical precise design, the shear strength of the joint is also considered

Factor of safety against sliding can be given based on Frictional resistance and shear strength of the dam

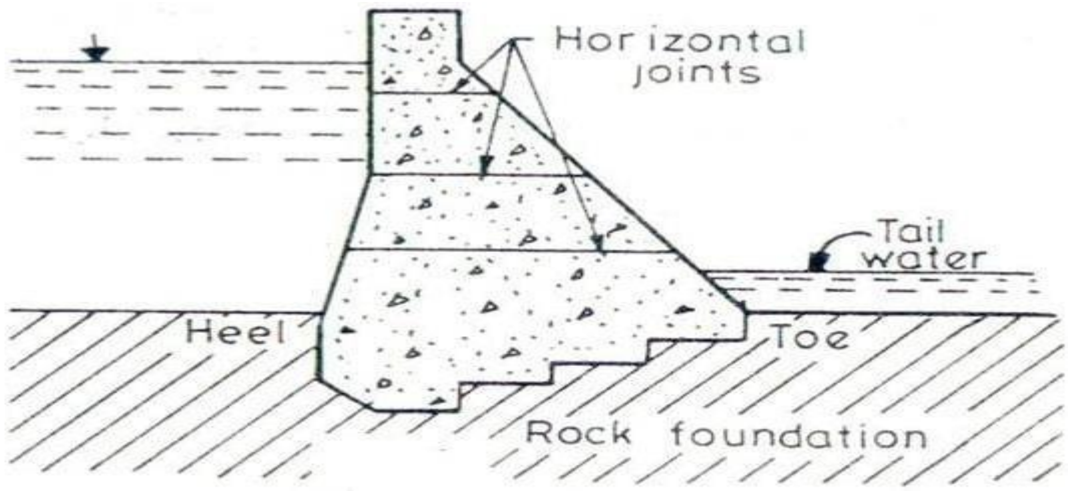
Factor of safety based on frictional resistance:

$$\text{FOS against sliding} = \text{FOS} = \frac{\mu \sum V}{\sum H}$$

μ=co-efficient of friction between two surfaces

$\sum V$ = sum of vertical forces acting on dam

$\sum H$ = sum of vertical forces acting on dam



Gravity Dam Failure due to Tension Cracks: Masonry and concrete are weak in tension. Thus masonry and concrete gravity dams are usually designed in such a way that no tension is developed anywhere. If these dams are subjected to tensile stresses, materials may develop tension cracks. Thus the dam loses contact with the bottom foundation due to this crack and becomes ineffective and fails. Hence, the effective width B of the dam base will be reduced. This will increase pmax at the toe. Hence, a tension crack by itself does not fail the structure, but it leads to the failure of the structure by producing excessive compressive stresses.

For high gravity dams, certain amount of tension is permitted under severest loading conditions in order to achieve economy in design. This is permitted because the worst condition of loads may occur only momentarily and may not occur frequently.

Gravity Dam Failure due to Compression:A gravity dam may fail by the failure of its material, i.e. the compressive stresses produced may exceed the allowable stresses, and the dam material may get crushed.

STABILITY ANALYSIS OF GRAVITY DAMS

General Selection of the method of analysis should be governed by the type and configuration of the structure being considered. The gravity method will generally be sufficient for the analysis of most structures, however, more sophisticated methods may be required for structures that are curved in plan, or structures with unusual configurations. 3-4.2 Gravity Method The gravity method assumes that the dam is a 2 dimensional rigid block. The foundation pressure distribution is assumed to be linear. It is usually prudent to perform gravity analysis before doing more rigorous studies. In most cases, if gravity analysis indicates that the dam is stable, no further analyses need be done.

Stability Analysis Assumptions:

1. The dam is considered to be composed of a number of Cantilevers, each of which is 1 m thick and each of which acts independently of the other.
2. No load is transferred to the abutments by beam action
3. The foundation and the dam behave as a single unit, the joints being perfect.
4. The material in the foundation and the body of the dam are isotropic and homogeneous.
5. The stresses developed in the foundation and the body of the dam is isotropic and homogeneous.
6. No movements of dams are caused by the Transfers of loads.

Stability Analysis Procedure

Two dimensional analysis can be carried out analytically or graphically

Analytical Method

1. Consider unit length of the dam
2. Work out the magnitude and direction of all the vertical forces acting on the dam and their algebraic sum i.e. $\sum V$
3. Similarly, work out all the horizontal forces and their algebraic sum, i.e.. $\sum H$
4. Determine the level arm of all these forces about the toe
5. Determine the moments of all these forces about the toe and find out the algebraic sum of all those moments i.e.. $\sum M$

Graphical method

In the graphical method, the entire dam section is divided into number of horizontal sections at some suitable interval. Particularly at the place where the slope changes.

1. For each section, the sum of the vertical forces $\sum V$ and the sum of all the horizontal forces $\sum H$ acting above that particular section, are worked out and the resultant is drawn, graphically
2. This is done for each section and a line joining all the points where the individual resultants cut the individual sections, is drawn.
3. This line represents the resultant force and should lie within the middle third, for no tension to develop.
4. The procedure should be repeated for reservoir full as well reservoir empty case.

Profile of A Dam from Practical Considerations

• The elementary profile of a gravity dam, (i.e.. triangle with maximum water surface at apex) is only a theoretical profile. Certain changes will have to be made in this profile in order to cater to the practical needs.
These needs are,
(i) Providing a straight top width for road construction over the top of the dam
(ii) Providing a free-board above the top water surface, so that water may spill over the top of the dam due to wave action, etc.

The addition of these two provisions, will cause the resultant force to shift towards the heel. The resultant force, when the reservoir is empty, was earlier passing through the inner middle third point. This will, therefore, shift more towards the heel, crossing the inner middle third point and consequently, tension will be developed at the toe. In order to avoid the development of this tension, some masonry will have to be added to the upstream side., which shows the typical section along with the possible dimensions that can be adopted for a low gravity dam section. It should however, be checked for stability analysis.

Galleries in Gravity Dams

Galleries are the horizontal or sloping openings or passages left in the body of the dam. • They may run longitudinally (i.e. parallel to dam axis) or traversely (i.e. normal to the dam axis) and are provided at various elevations. All the galleries are interconnected by steeply sloping passages or by vertical shafts fitted with stairs or mechanical lifts.

Function and types of galleries in Dams

(i) Foundation Gallery

A gallery provided in a dam may serve one particular purpose or more than one purpose. For example, a gallery provided near the rock foundation, serves to drain off the water which percolates through the foundations. This gallery is called a foundation gallery or a drainage gallery.

1. It runs longitudinally and is quite near to the upstream face of the dam. Drain holes are drilled from the floors of this gallery after the foundation grouting has been completed. Seepages is collected through these drain holes.
2. Besides draining off seepage water, it may be helpful for drilling and grouting of the foundations, when this can not be done from the surface of the dam.

(ii)Inspection Galleries

The water which seeps through the body of the dam is collected by means of a system of galleries provided at various elevations and interconnected by vertical shafts, etc. All these galleries, besides draining off seepage water, serves inspection purpose. They provide access to the interior of the dam and are, therefore, called inspection purposes. They generally serve other purposes along with this purpose.

1. They intercept and drain off the water seeping through the dam body
2. They provide access to dam interior for observing and controlling the behavior of the dam.
3. They provide enough space for carrying pipes, etc. during artificial cooling of concrete
4. They provide access to all the outlets and spillway gates, valves, etc. by housing their electrical and mechanical controls. All these gates, valves, etc, can hence be easily controlled by men, from inside the dam itself.

5. They provide space for drilling and grouting of the foundations, then it cannot be done from the surface of the dam.

UNIT-II

EARTH DAMS

An **embankment dam** is a large artificial dam. It is typically created by the placement and compaction of a complex semi-plastic mound of various compositions of soil, sand, clay, or rock. It has a semi-pervious waterproof natural covering for its surface and a dense, impervious core. This makes such a dam impervious to surface or seepage erosion.[1] Such a dam is composed of fragmented independent material particles. The friction and interaction of particles binds the particles together into a stable mass rather than by the use of a cementing substance.

Embankment dams come in two types: the **earth-filled dam** (also called an earthen dam or terrain dam) made of compacted earth, and the **rock-filled dam**. A cross-section of an embankment dam shows a shape like a bank, or hill. Most have a central section or core composed of an impermeable material to stop water from seeping through the dam. The core can be of clay, concrete, or asphalt concrete. This dam type is a good choice for sites with wide valleys. They can be built on hard rock or softer soils. For a rock-fill dam, rock-fill is blasted using explosives to break the rock. Additionally, the rock pieces may need to be crushed into smaller grades to get the right range of size for use in an embankment dam

The building of a dam and the filling of the reservoir behind it places a new weight on the floor and sides of a valley. The stress of the water increases linearly with its depth. Water also pushes against the upstream face of the dam, a nonrigid structure that under stress behaves semiplastically, and causes greater need for adjustment (flexibility) near the base of the dam than at shallower water levels. Thus the stress level of the dam must be calculated in advance of building to ensure that its break level threshold is not exceeded.

Overtopping or overflow of an embankment dam beyond its spillway capacity will cause its eventual failure. The erosion of the dam's material by overtopping runoff will remove masses of material whose weight holds the dam in place and against the hydraulic forces acting to move the dam. Even a small sustained overtopping flow can remove thousands of tons of overburden soil from the mass of the dam within hours. The removal of this mass unbalances the forces that stabilize the dam against its reservoir as the mass of water still impounded behind the dam presses against the lightened mass of the embankment, made lighter by surface erosion. As the mass of the dam erodes, the force exerted by the reservoir begins to move the entire structure. The embankment, having almost no elastic strength, would begin to break into separate pieces, allowing the impounded reservoir water to flow between them, eroding and removing even more material as it passes through. In the final stages of failure the remaining pieces of the embankment would offer almost no resistance to the flow of the water and continue to fracture into smaller and smaller sections of earth or rock until these would disintegrates into a thick mud soup of earth, rocks and water.

Therefore, safety requirements for the spillway are high, and require it to be capable of containing a maximum flood stage. It is common for its specifications to be written such that it can contain a five hundred year flood. Recently a number of embankment dam overtopping protection systems have been developed. These techniques include the concrete overtopping protection systems, timber cribs, sheet-piles, riprap and gabions, reinforced earth, minimum energy loss weirs, embankment overflow stepped spillways and the precast concrete block protection systems.

The two principal types of embankment dams are earth and rock-fill dams, depending on the predominant fill material used. Some generalized sections of earth dams showing typical zoning for different types and quantities of fill materials When practically only one impervious material is available and the height of the dam is relatively low, a homogeneous dam with internal drain. The inclined drain serves to prevent the downstream slope from becoming saturated and susceptible to piping and/or slope failure and to intercept and prevent piping through any horizontal cracks traversing the width of the embankment.

Earth Dam with impervious cores, are constructed when local borrow materials do not provide adequate quantities of impervious material. A vertical core located near the center of the dam is preferred over an inclined upstream core because the former provides higher contact pressure between the core and foundation to prevent leakage, greater stability under earthquake loading, and better access for remedial seepage control. An inclined upstream core allows the downstream portion of the embankment to be placed first and the core later and reduces the possibility of hydraulic fracturing. However, for high dams in steep-walled canyons the overriding consideration is the abutment topography. The objective is to fit the core to the topography in such a way to avoid divergence, abrupt topographic discontinuities, and serious geologic defects. For dams on pervious foundations,seepage control is necessary to prevent excessive uplift pressures and piping through the foundation.

The methods for control of under seepage in dam foundations are horizontal drains, cutoffs (compacted backfill trenches, slurry walls, and concrete walls), upstream impervious blankets, downstream seepage berms, toe drains, and relief wells.A rock-fill dam with steep slopes requires better foundation conditions than an earth dam, and a concrete dam (or roller-compacted concrete dam) requires better foundation conditions than a rock-fill dam.

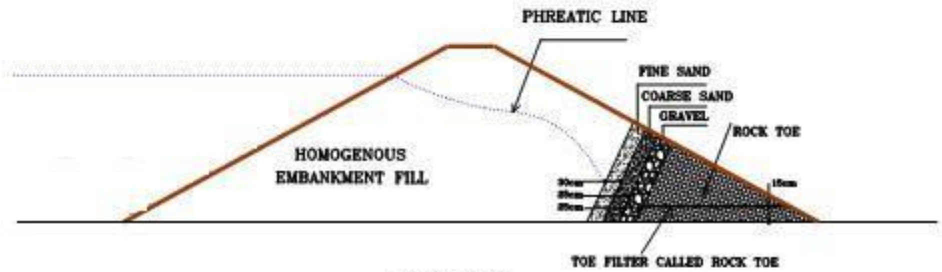
An earth dam is composed of suitable soils obtained from borrow areas or required excavation and compacted in layers by mechanical means. Following preparation of a foundation, earth from borrows areas and from required excavations is transported to the site, dumped, and spread in layers of required depth. The soil layers are then compacted by tamping rollers; sheep foot rollers, heavy pneumatic-tired rollers, vibratory rollers, tractors, or earth-hauling equipment. One advantage of an earth dam is that it can be adapted to a weak foundation, provided proper consideration is given to thorough foundation exploration, testing, and design.

Phreatic Line and Horizontal Drain In Earth fill Dams

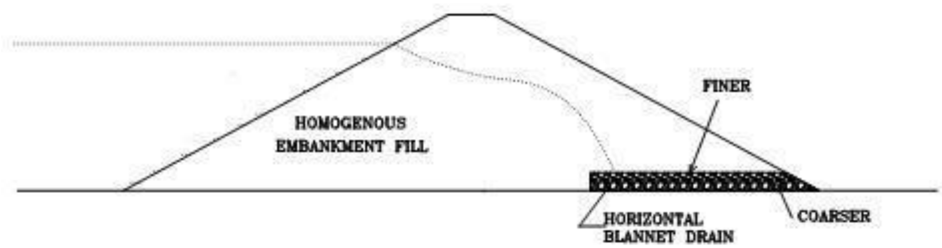
Earth dams are generally built of locally available materials in their natural state with a minimum of processing. Homogeneous earth dams are built whenever only one type of material is economically available.

The material must be sufficiently impervious to provide an adequate water barrier and slopes must be relatively flat to make it safe against piping and sloughing.

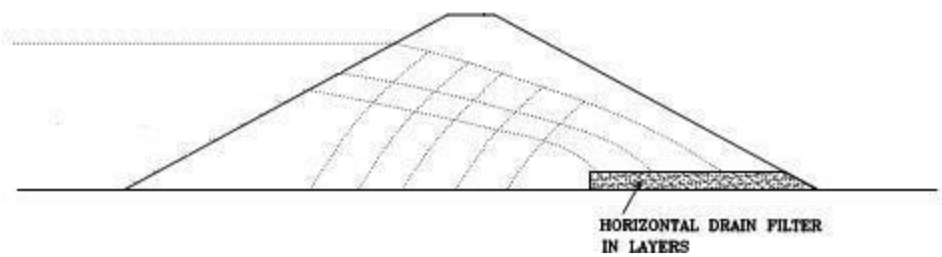
The general design procedure is to make a first estimate on the basis of experience with similar dams and then to modify the estimate as required after conducting a stability analysis except where there is a surplus of material.



Rock Toe



Horizontal Fillter



The upstream slopes of most of the earth dams in actual practice usually vary **from 2.0 (horizontal):1 (vertical) to 4:1** and the downstream slopes are generally between **2:1 and 3:1** (USBR 2003). Free board depends on the height and action of waves. USBR (2003) recommends normal free-board about 1.5 to 3 m depending on the fetch. The width of the dam crest is determined by considering the nature of embankment materials, height and importance of structure, possible roadways requirements, and practicability of construction. A majority of dams have the crest widths varying between 5 and 12 m.

About 30% of dams had failed due to the seepage failure, viz piping and sloughing. Recent comprehensive reviews by Foster et al. (2000a,b) and Fell et al. (2003) show that internal erosion and piping are the main causes of failure and accidents affecting embankment dams; and the proportion of their failures by piping increased from 43% before 1950 to 54% after 1950. The sloughing of the downstream face of a homogeneous earth dam occurs under the steady-state seepage condition due to the softening and weakening of the soil mass when the top flow line or phreatic line intersects it. Regardless of flatness of the downstream slope and impermeability of soil, the phreatic line intersects the downstream face to a height of roughly one-third the depth of water . It is usual practice to use a modified homogeneous section in which an internal drainage system in the form of a horizontal blanket drain or a rock toe or a combination of the two is provided. The drainage system keeps the phreatic line well within the body of the dam. Horizontal filtered drainage blankets are widely used for dams of moderate height.USBR constructed the 50 m high Vega dam, which is one of the highest with a homogenous section and a horizontal downstream drain.

The minimum length of the horizontal blanket drain required to keep the phreatic line within the body of the dam by a specified depth and also equations for maximum downstream slope cover and minimum and maximum effective lengths of the downstream filtered drainage system.

The position of the phreatic line influences the stability of the earth dam because of potential piping due to excessive exit gradient and sloughing due to the softening and weakening of the soil mass as if it touches the downstream slope or intersects it. When the dam embankment is homogeneous or when the downstream zone is of questionable permeability, a horizontal drainage blanket is provided to keep the phreatic line well within the dam body, to allow adequate embankment and foundation drainage, and to eliminate piping from the foundation and the embankment.

As the dams are made of fine-grained soil, saturation may occur due to the capillary rise above the phreatic surface so it is necessary to account for capillary rise while calculating the minimum length of the downstream filtered drainage. Though the suction head in the soil matrix above the phreatic surface within the dam body due to capillary rise generally improves the stability of the downstream slope, once the capillary fringe intersects the downstream slope the pressure changes from negative (suction) to atmospheric and the downstream face may become a seepage face leading to its failure. Hence the phreatic line should not intersect the downstream slope and it should be a distance greater than capillary rise below the sloping face so that the chances of the sloughing or piping may be nullified.

Stability and Failure of Earth Filled Dams

Failure of earth dams may be:

1. Hydraulic Failure
2. Seepage Failure
3. Structural Failure

1. Hydraulic Failure:

1. Overtopping of dams
2. Erosion of the Upstream Surface
3. Erosion of the Downstream Surface
4. Erosion of the Downstream toe

i. Overtopping of dams:

This type of dam is made up of only one type of material. Usually porous materials is used. These dams are easy and cheap to construct but cannot be used to make multipurpose large dams. For large multipurpose dams zoned type method is used. Over topping failures result from the erosive action of water on the embankment. Erosion is due to un-controlled flow of water over, around, and adjacent to the dam. Earth embankments are not designed to be over-topped and therefore are particularly susceptible to erosion. Once erosion has begun during over-topping, it is almost impossible to stop. A well vegetated earth embankment may withstand limited over topping if its crest is level and water flows over the crest and down the face as an evenly distributed sheet without becoming concentrated. The owner should closely monitor the reservoir pool level during severe storms.

ii. Erosion of the Upstream Surface:

Here zones of different materials are made.

Shell is used to give support and stability to the structure of dam. It is made of coarse materials and is pervious in nature.

Core is used to make the dam water tight and to reduce the seepage. Fine material is used here.

Used in large dams.

iii. Erosion of the Downstream Surface:

Due to rainfall, snow and winds the downstream surface of the dam also erodes. By providing a section of coarse materials here, this erosion can be reduced or prevented.

2. Seepage Failure:

All earth dams have seepage resulting from water permeating slowly through the dam and its foundation. Seepage must be controlled in both velocity and quantity. If uncontrolled, it can progressively erode soil from the embankment or its foundation, resulting in rapid failure of the dam. Erosion of the soil begins at the downstream side of the embankment, either in the dam proper or the foundation, progressively works toward the reservoir, and eventually develops a direct connection to the reservoir. This phenomenon is known as "piping." Piping action can be recognized by an increased seepage flow rate, the discharge of muddy or discolored water, sinkholes on or near the embankment, or a whirlpool in the reservoir. Once a whirlpool (eddy) is observed on the reservoir surface, complete failure of the dam will probably follow in a matter of minutes. As with over topping, fully developed piping is virtually impossible to control and will likely cause failure. Seepage can cause slope failure by creating high pressures in the soil pores or by saturating the slope. The pressure of seepage within an embankment is difficult to determine without proper instrumentation. A slope which becomes saturated and develops slides may be showing signs of excessive seepage pressure.

Seepage failure of the dams is of the following types

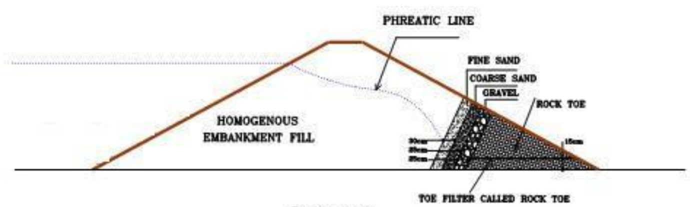
1. Piping through the dam
2. Piping through the foundation
3. Conduit Leakage

1. **Piping thorugh the dam:** There are two kinds of forces acting on the downstream face of the dam:
    1. Weight of the material
    2. Seepage Force

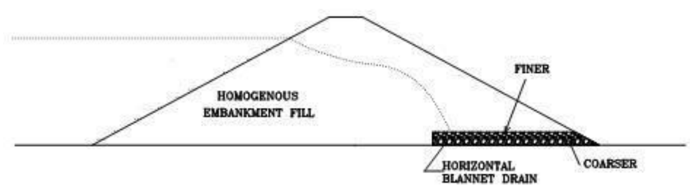
If the seepage force exceeds the weight of the material the water washes away the soil from the plate and creates a hole in the ground. This hole deepens as more and more mateial is taken away from it and extends longitudinally, making a pipe hole called "Piping in the dam".

3. Structural Failure:

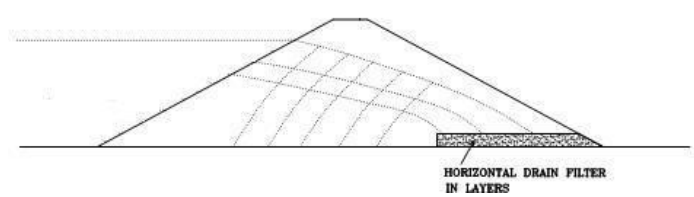
Structural failures can occur in either the embankment or the appurtenances. Structural failure of a spillway, lake drain, or other appurtenance may lead to failure of the embankment. Cracking, settlement, and slides are the more common signs of structural failure of embankments. Large cracks in either an appurtenance or the embankment, major settlement, and major slides will require emergency measures to ensure safety, especially if these problems occur suddenly. If this type of situation occurs, the lake level should be lowered, the appropriate state and local authorities notified, and professional advice sought. If the observer is uncertain as to the seriousness of the problem, the Division of Water should be contacted immediately. The three types of failure previously described are often interrelated in a complex manner. For example, uncontrolled seepage may weaken the soil and lead to a structural failure. A structural failure may shorten the seepage path and lead to a piping failure. Surface erosion may result in structural failure.



Rock Toe



Horizontal Fillter



1. Failure of downstream face during steady seepage conditions
2. Failure of upstream face during sudden draw down
3. Failure due to sliding of foundation
4. damage due to burrowing animals
5. Failure of dam due to earthquake

1. Usually upper part of the dam is dry and the lower is saturated with water which gives rise to pore water pressure within the voids. Dam body is saturated - All pores / voids are filled with water, pore water pressure is induced. Effective pressure reduces and shear strength of soil decreases
2. When water is suddenly withdrawn or in other words if the level of water in the reservoir reduces suddenly, the soil on the upstream face of the dam body may be highly saturated and has pore water pressure that tries to destabilise the dam and if this force is high enough, it can fail the dam.
3. If the shear strength of the soil on which the foundation is built is weak though the foundation itself may be strong but due to weakness of the soil foundation may slide on the sides and in some cases the foundation itself may be not able to resist the shear force that may have increased from normal due to any reason.
4. Burrowing animals - Small animals living in the holes and pits may have dug theri holes anywhere in te dam body which may widen with the passage of time and can be dangerous.
5. Earthquake

Minor defects such as cracks in the embankment may be the first visual sign of a major problem which could lead to failure of the structure. The seriousness of all deficiencies should be evaluated by someone experienced in dam design and construction. A qualified professional engineer can recommend appropriate permanent remedial measures.

Spill ways and energy dissipaters

Instructional objectives

On completion of this lesson, the student shall learn:

1. The functions of a spillways and energy dissipators in projects involving diversion and storage projects
2. Different types of spillways
3. How to determine the shape of an ogee-crested spillway and compute its discharge
4. The spillway profile in the presence of a breast wall
5. Criteria for selecting a particular type of spillway
6. Different types of energy dissipators
7. Design procedure for hydraulic jump and bucket-type energy dissipators
8. Protection measures against science downstream of energy dissipators

Introduction

The previous lessons dealt with storage reservoirs built by impounding a river with a dam and the common types of dams constructed by engineers. However, in rare cases only it is economical or practical for the reservoir to store the entire volume of the design flood within the reservoir without overtopping of dam. Hence, a dam may be constructed to that height which is permissible within the given topography of the location or limited by the expenditure that may be possible for investment. The excess flood water, therefore, has to be removed from the reservoir before it overtops the dam. Passages constructed either within a dam or in the periphery of the reservoir to safely pass this excess of the river during flood flows are called Spillways.

Ordinarily, the excess water is drawn from the top of the reservoir created by the dam and conveyed through an artificially created waterway back to the river. In some cases, the water may be diverted to an adjacent river valley. In addition to providing sufficient capacity, the spillway must be hydraulically adequate and structurally safe and must be located in such a way that the out-falling discharges back into the river do not erode or undermine the downstream toe of the dam. The surface of the spillway should also be such that it is able to withstand erosion or scouring due to the very high velocities generated during the passage of a flood through the spillway.

The flood water discharging through the spillway has to flow down from a higher elevation at the reservoir surface level to a lower elevation at the natural river level on the downstream through a passage, which is also considered a part of the spillway. At the bottom of the channel, where the water rushes out to meet the natural river, is usually provided with an energy dissipation device that kills most of the energy of the flowing water. These devices, commonly called as Energy Dissipators, are required to prevent the river surface from getting dangerously scoured by the impact of the outfalling water. In some cases, the water from the spillway may be allowed to drop over a free overfall, as in Kariba Dam on Zambezi River in Africa, where the free fall is over 100m.

In some projects, like the Indira Sagar Dam on River Narmada, two sets of spillways are provided – Main and Auxillary. The main spillway, also known as the service spillway is the one which is generally put into operation in passing most of the design flood. The crest levels of the auxillary spillways are usually higher and thus the discharge capacities are also small and are put into operation when the discharge in the river is higher than the capacity of the main spillway. Sometimes, an Emergency or Fuse Plug types of spillway is provided in the periphery of the reservoir which operates only when there is very high flood in the river higher than the design discharge or during the malfunctioning of normal spillways due to which there is a danger of the dam getting overtopped.

Usually, spillways are provided with gates, which provides a better control on the discharges passing through. However, in remote areas, where access to the gates by personnel may not be possible during all times as during the rainy season or in the night ungated spillways may have to be provided.

The capacity of a spillway is usually worked out on the basis of a flood routing study, explained in lesson 4.5. As such, the capacity of a spillway is seen to depend upon the following major factors:

- The inflow flood
- The volume of storage provided by the reservoir
- Crest height of the spillway
- Gated or ungated

According to the Bureau of Indian Standards guideline IS: 11223-1985 "Guidelines for fixing spillway capacity", the following values of inflow design floods (IDF) should be taken for the design of spillway:

- For large dams (defined as those with gross storage capacity greater than 60 million m^3 or hydraulic head greater than 60 million m^3 or hydraulic head between (2m and 30m), IDF should be based on the Standard Project Flood (SPF).
- For intermediate dams those with gross storage between 10 and 60 million m^3 or hydraulic head between (2m and 30m), IDF should be based on the Standard Project Flood (SPF).
- For small dams (gross storage between 0.5 to 10 million m^3 or hydraulic head between 7.5m to 12m), IDF may be taken as the 100 year return period flood.

The volume of the reservoir corresponding to various elevation levels as well as the elevation of the crest also affects the spillway capacity, as may be obvious from the flood routing procedure shown in Lesson 4.5.

If the spillway is gated, then the discharging water (Q) is controlled by the gate opening and hence the relation of Q to reservoir water level would be different from that of an ungated spillway. In the example of Lesson 4.5, an ungated spillway considered. Where as, in most practical cases, spillways are provided with gates and the gate operation is guided by a certain predetermined sequence which depends upon the inflow discharge. Hence, for an actual spillway capacity design, one has to consider not only the inflow hydrograph, but also the gate operation sequence.

Apart from spillways, which safely discharge the excess flood flows, outlets are provided in the body of the dam to provide water for various demands, like irrigation, power generation, etc. Hence, ordinarily riverflows are usually stored in the reservoir or released through the outlets, and the spillway is not required to function. Spillway flows will result during floods or periods of sustained high runoff when the capacities of other facilities are exceeded. Where large reservoir storage is provided, or where large outlet or diversion capacity is available, the spillway will be utilized infrequently. This feature may be contrasted with that of a diversion structure-like a barrage-where the storage is almost nil, and hence, the spillway there is in almost continuous operation.

Spillways are ordinarily classified according to their most prominent feature, either as it pertains to the control, to the discharge channel, or to some other component. The common types of spillway in use are the following:

1. Free Overfall (Straight Drop) Spillway
2. Overflow (Ogee) Spillway
3. Chute (Open Channel/Trough) Spillway
4. Side Channel Spillway
5. Shaft (Drop Inlet/Morning Glory) spillway
6. Tunnel (Conduit) spillway
7. Siphon spillway

These spillways are individually treated in the subsequent sections.

The water flowing down from the spillways possess a large amount of kinetic energy that is generated by virtue of its losing the potential head from the reservoir level to the level of the river on the downstream of the spillway. If this energy is not reduced, there are danger of scour to the riverbed which may threaten the stability of the dam or the neighbouring river valley slopes. The various arrangements for suppressing or killing of the high energy water at the downstream toe of the spillways are called Energy Dissipators. These are discussed at the end of this lesson.

Free Overfall Spillway

In this type of spillway, the water freely drops down from the crest, as for an arch dam (Figure 1). It can also be provided for a decked over flow dam with a vertical or adverse inclined downstream face (Figure 2). Flows may be free discharging, as will be the case with a sharp-crested weir or they may be supported along a narrow section of the crest. Occasionally, the crest is extended in the form of an overhanging lip (Figure 3) to direct small discharges away from the face of the overfall section. In free falling water is ventilated sufficiently to prevent a pulsating, fluctuating jet.

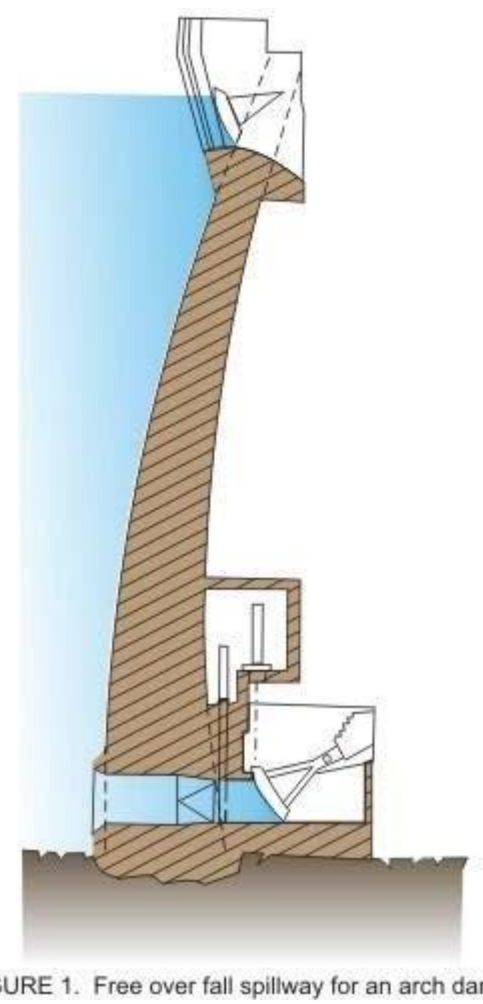
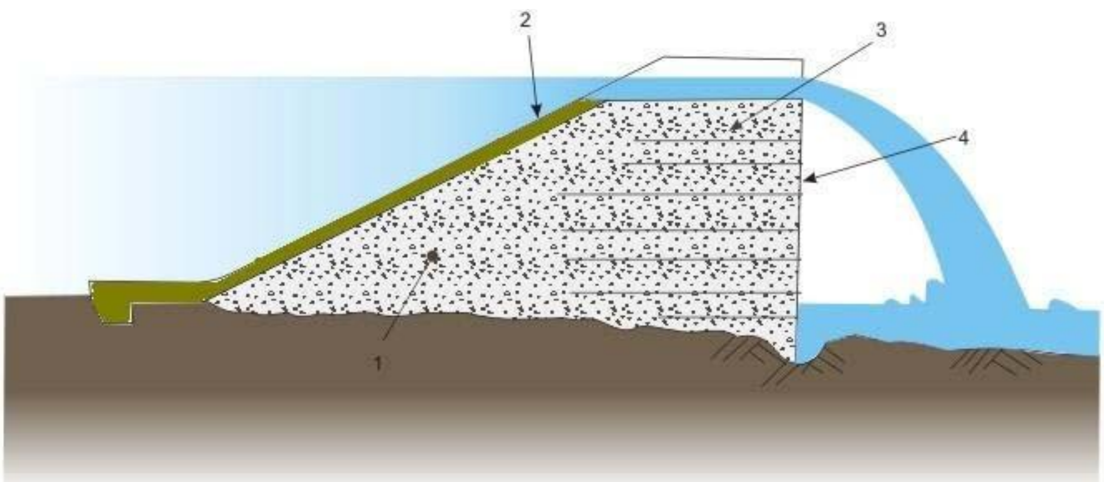


FIGURE 1. Free over fall spillway for an arch dam



LEGEND

1. RANDOM FILL 2. WATERTIGHT MEMBRANE 3. STEEL TENDONS 4. CONCRETE SLABS (1.5 M X 1.5 M).

FIGURE 2. Free over fall spillway for a decked embankment dam

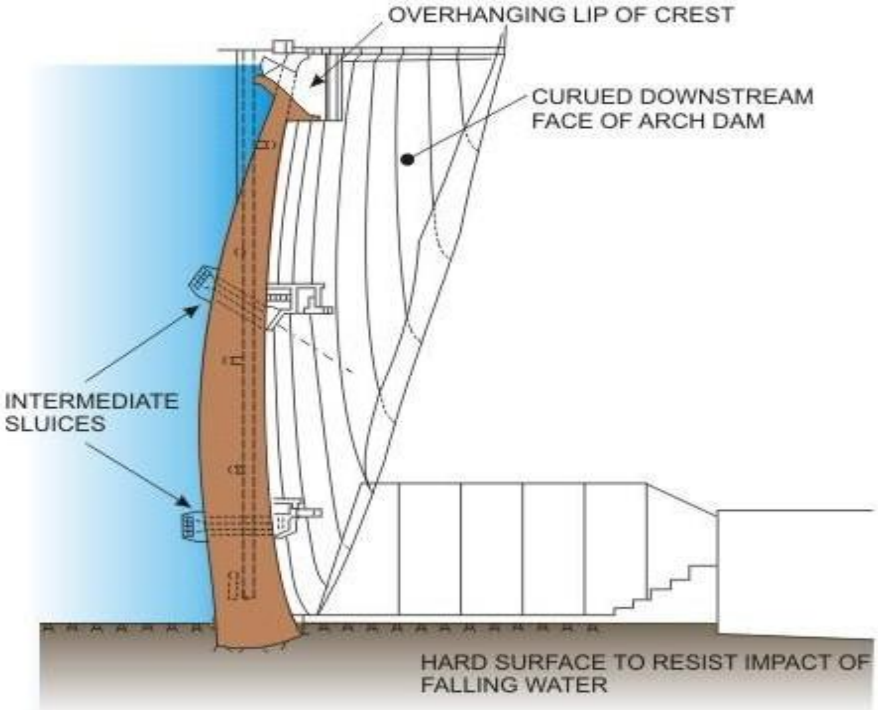


FIGURE 3. Short lip provided for overfall spilling of an arch dam

Where artificial protection is provided at the loose, as in Figure 3, the bottom may not scour but scour may occur for unprotected streambeds which will form deep plunge pool (Figure 4). The volume and the depth of the scour hole are related to the range of discharges, the height of the drop, and the depth of tail water. Where erosion cannot be tolerated an artificial pool can be created by constructing an auxiliary dam downstream of the main structure, or by excavating a basin which is then provided with a concrete apron or bucket.

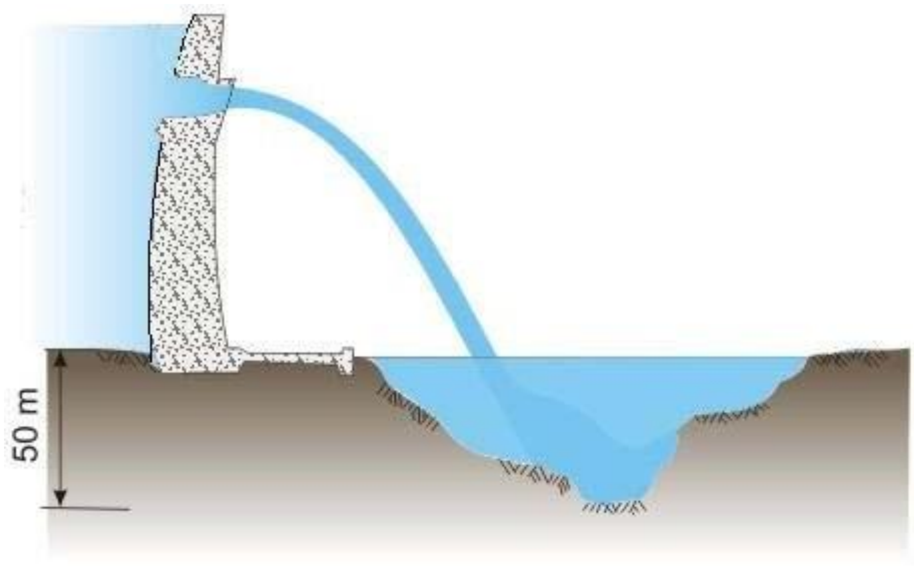


FIGURE 4. Scour below Kariba Dam Spillway , Zimbabwe

Overflow Spillway

The overflow type spillway has a crest shaped in the form of an ogee or S-shape (Figure 5). The upper curve of the ogee is made to conform closely to the profile of the lower nappe of a ventilated sheet of water falling from a sharp crested weir (Figure 6). Flow over the crest of an overflow spillway is made to adhere to the face of the profile by preventing access of air to the underside of the sheet of flowing water. Naturally, the shape of the overflow spillway is designed according to the shape of the lower nappe of a free flowing weir conveying the discharge flood. Hence, any discharge higher than the design flood passing through the overflow spillway would try to shoot forward and get detached from the spillway surface, which reduces the efficiency of the spillway due to the presence of negative pressure between the sheet of water and spillway surface. For discharges at designed head, the spillway attains near-maximum efficiency. The profile of the spillway surface is continued in a tangent along a slope to support the sheet of flow on the face of the overflow. A reverse curve at the bottom of the slope turns the flow in to the apron of a sliding basis or in to the spillway discharge channel.

An ogee crest apron may comprise an entire spillway such as the overflow of a concrete gravity dam (Figure 7), or the ogee crest may only be the control structure for some other type of spillway (Figure 8). Details of computing crest shape and discharges of ogee shaped crest is provided in Section 4.8.9.

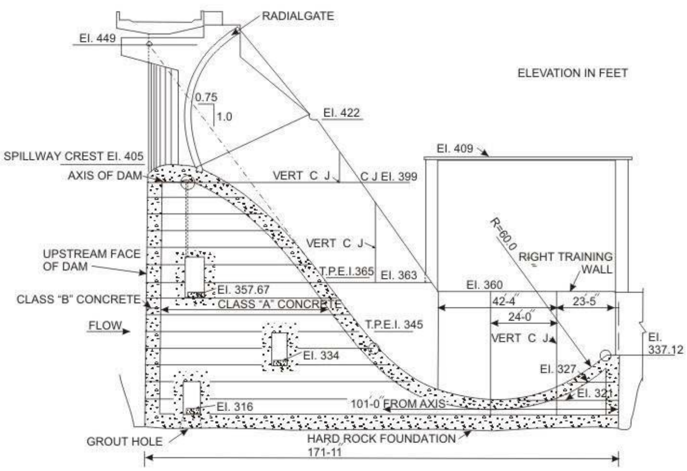


FIGURE 5. Typical overflow (ogee) spillway .Example of Panchet Dam on River Damodar

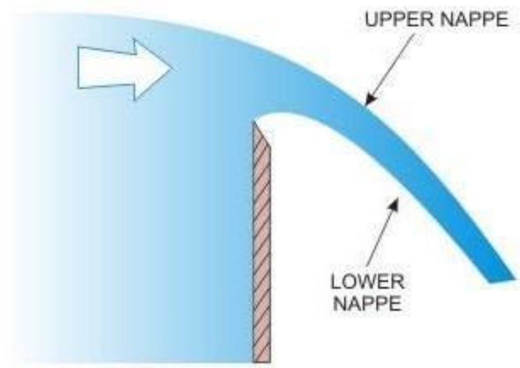


FIGURE 6. Outflow from a free-falling weir , properly ventilated from below

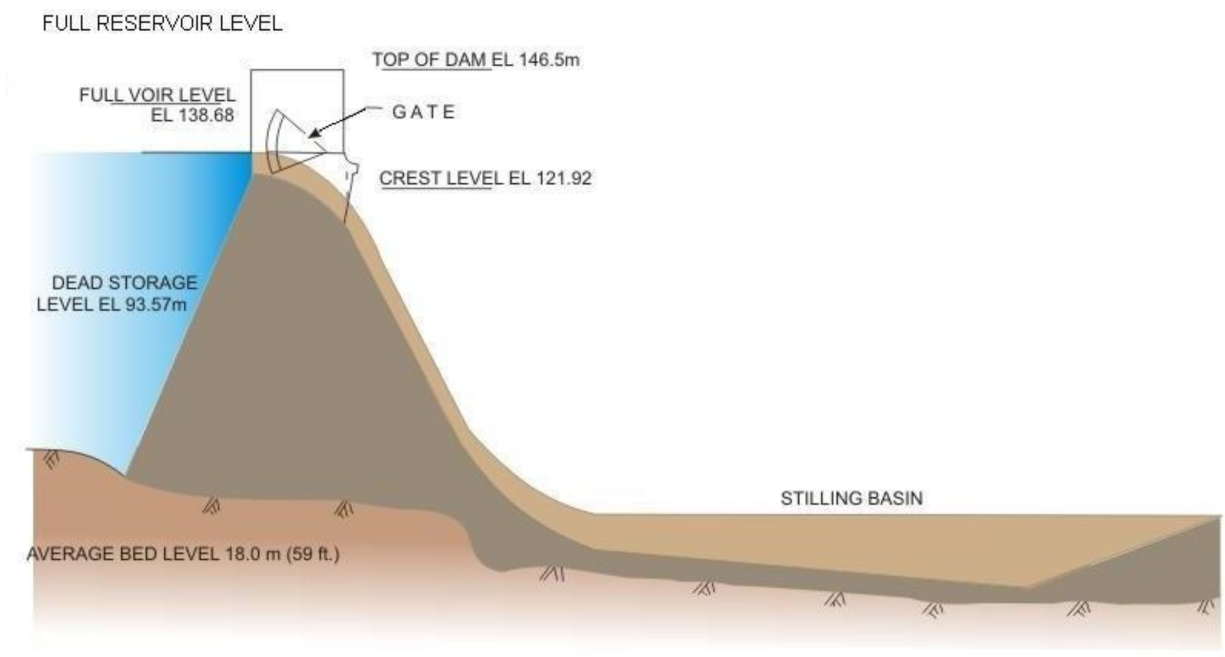


FIGURE 7. Ogee spillway & apron of Sardar Sarovar Dam spillway

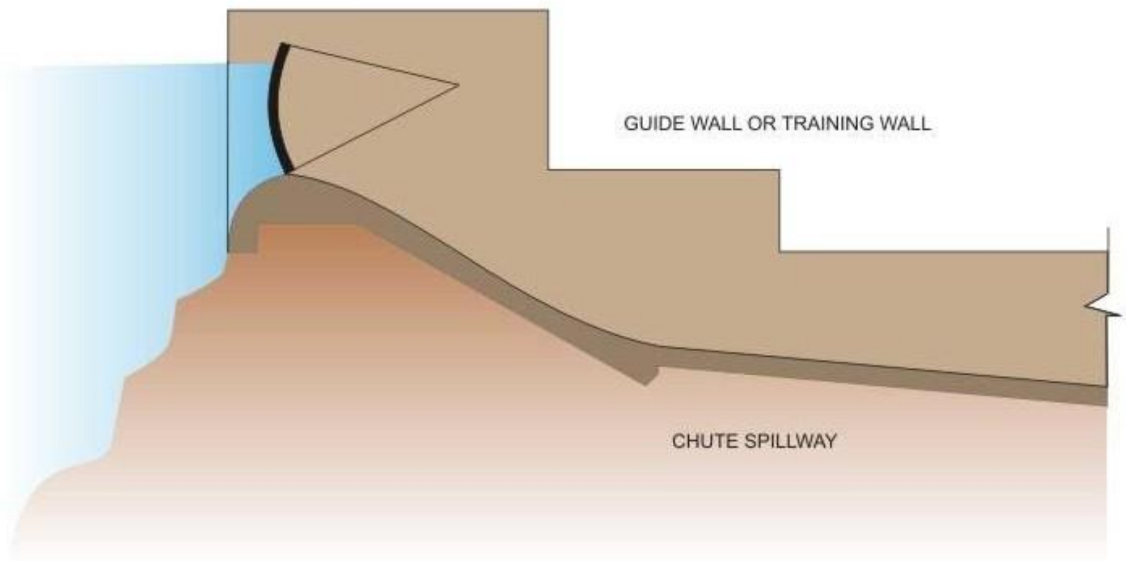


FIGURE 8. Ogee spillway for controlling flow into a chute-type spillway

Chute Spillway

A chute spillway, variously called as open channel or trough spillway, is one whose discharge is conveyed from the reservoir to the downstream river level through an open channel, placed either along a dam abutment or through a saddle (Figure 9). The control structure for the chute spillway need not necessarily be an overflow crest, and may be of the side-channel type (discussed in Section 4.9.4), as has been shown in Figure 10. However, the name is most often applied when the spillway control is placed normal or nearly normal to the axis of the open channel, and where the streamlines of flow both above and below the control crest follow in the direction of the axis.

Generally, the chute spillway has been mostly used in conjunction with embankment dams, like the Tehri dam, for example. Chute spillways are simple to design and construct and have been constructed successfully on all types of foundation materials, ranging from solid rock to soft clay.

Chute spillways ordinarily consist of an entrance channel, a control structure, a discharge channel, a terminal structure, and an outlet channel. Often, the axis of the entrance channel or that of the discharge channel must be curved to fit the topography. For further details, one may refer to the Bureau of Indian Standards Code IS: 5186- 1994 “Design of chute and side channel spillways-criteria”.

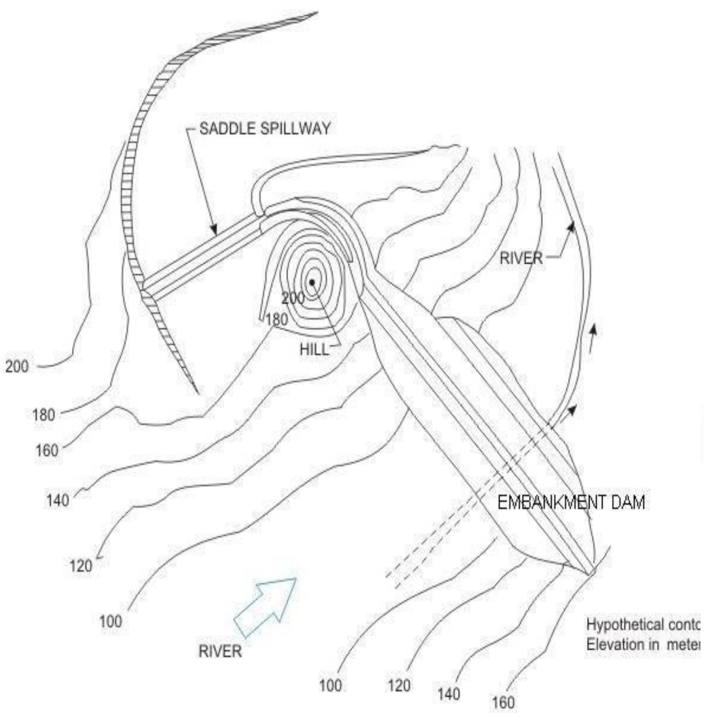


FIGURE 9. Saddle spillway

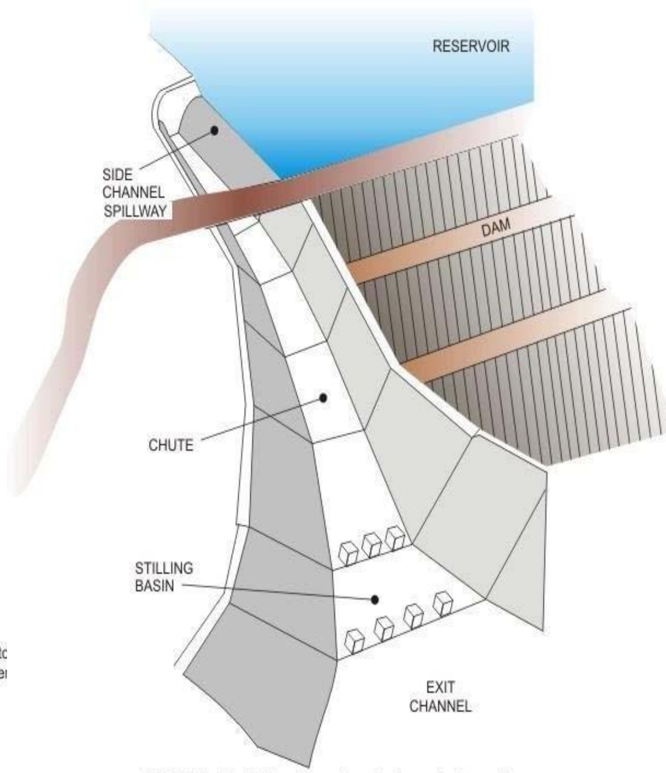


FIGURE 10. Side channel entry to a chute spillway

Side channel Spillway

A side channel spillway is one in which the control weir is placed approximately parallel to the upper portion of the discharge channel, as may be seen from Figure 10. When seen in plan with reference to the dam, the reservoir and the discharge channel, the side channel spillway would look typically as in Figure 11 and its sectional view in Figure 12. The flow over the crest falls into a narrow trough opposite to the weir, turns an approximate right angle, and then continues into the main discharge channel. The side channel design is concerned only with the hydraulic action in the upstream reach of the discharge channel and is more or less independent of the details selected for the other spillway components. Flow from the side channel can be directed into an open discharge channel, as in Figure 10 or 11 showing a chute channel, or in to a closed conduit which may run under pressure or inclined tunnel. Flow into the side channel might enter on only one side of the trough in the case of a steep hill side location or on both sides and over the end of the trough if it is located on a knoll or gently sloping abutment.

Discharge characteristics of a side channel spillway are similar to those of an ordinary overflow spillway and are dependent on the selected profile of the weir crest. Although the side channel is not hydraulically efficient, nor inexpensive, it has advantages which make it adoptable to spillways where a long overflow crest is required in order to limit the afflux (surcharge held to cause flow) and the abutments are steep andprecipitous.

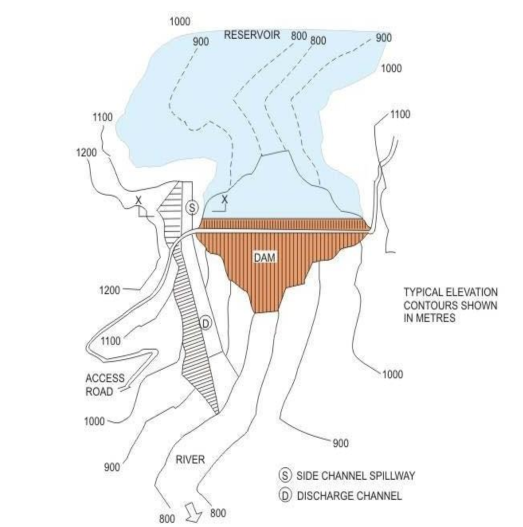


FIGURE 11. Plan of an embankment dam showing side channel spillway and chute channel

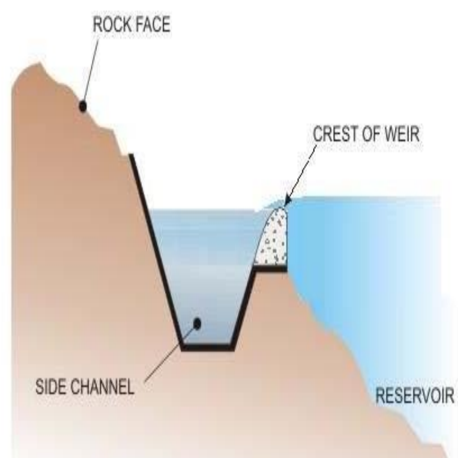


FIGURE 12. Magnified sectional view X-X through the side channel spillway shown in Figure 11

Shaft Spillway

A Shaft Spillway is one where water enters over a horizontally positioned lip, drops through a vertical or sloping shaft, and then flows to the downstream river channel through a horizontal or nearly horizontal conduit or tunnel (Figure 13). The structure may be considered as being made up of three elements, namely, an overflow control weir, a vertical transition, and a closed discharge channel. When the inlet is funnel shaped, the structure is called a Morning Glory Spillway. The name is derived from the flower by the same name, which it closely resembles especially when fitted with anti- vortex piers (Figure 14). These piers or guide vanes are often necessary to minimize vortex action in the reservoir, if air is admitted to the shaft or bend it may cause troubles of explosive violence in the discharge tunnel-unless it is amply designed for freeflow.

Discharge characteristics of the drop inlet spillway may vary with the range of head. As the head increases, the flow pattern would change from the initial weir flow over crest to tube flow and then finally to pipe flow in the tunnel. This type of spillway attains maximum discharging capacity at relatively low heads. However, there is little increase in capacity beyond the designed head, should a flood larger than the selected inflow design flood occur.

A drop inlet spillway can be used advantageously at dam sites that are located in narrow gorges where the abutments rise steeply. It may also be installed at projects where a diversion tunnel or conduit is available for use.

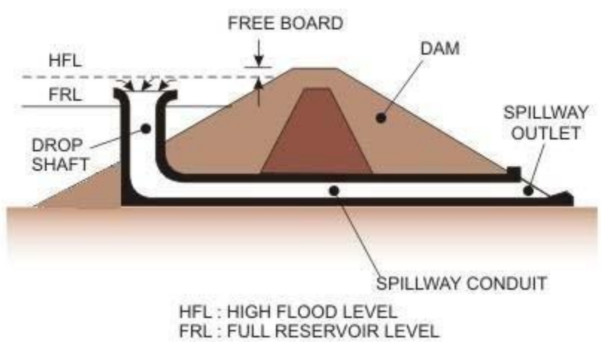


FIGURE 13. Section through a shaft spillway

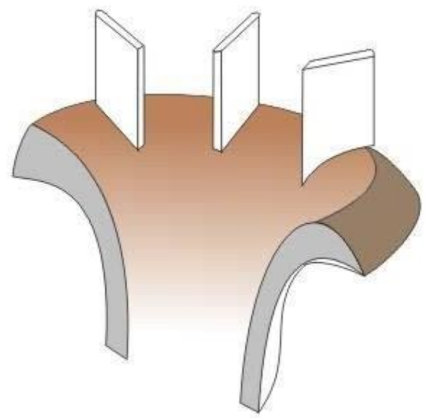


FIGURE 14. Morning glory spillway with anti-vortex piers

Tunnel Spillway

Where a closed channel is used to convey the discharge around a dam through the adjoining hill sides, the spillway is often called a tunnel or conduit spillway. The closed channel may take the form of a vertical or inclined shaft, a horizontal tunnel through earth or rock, or a conduit constructed in open cut and backfilled with earth materials. Most forms of control structures, including overflow crests, vertical or inclined orifice entrances, drop inlet entrances, and side channel crests, can be used with tunnel spillways. Two such examples have been shown in Figs. 15 and 16. When the closed channel is carried under a dam, as in Figure 13, it is known as a conduit spillway.

With the exception of those with orifice or shaft type entrances, tunnel spillways are designed to flow partly full throughout their length. With morning glory or orifice type control, the tunnel size is selected so that it flows full for only a short section at the control and thence partly full for its remaining length. Ample aeration must be provided in a tunnel spillway in order to prevent a fluctuating siphonic action which would result if some part of exhaution of air caused by surging of the water jet, or wave action or backwater.

Tunnel spillways are advantageous for dam sites in narrow gorges with steep abutments or at sites where there is danger to open channels from rock slides from the hills adjoining the reservoir.

Conduit spillways are generally most suited to dams in wide valleys as in such cases the use of this types of spillway would enable the spillway to be located under the dam very close to the stream bed.

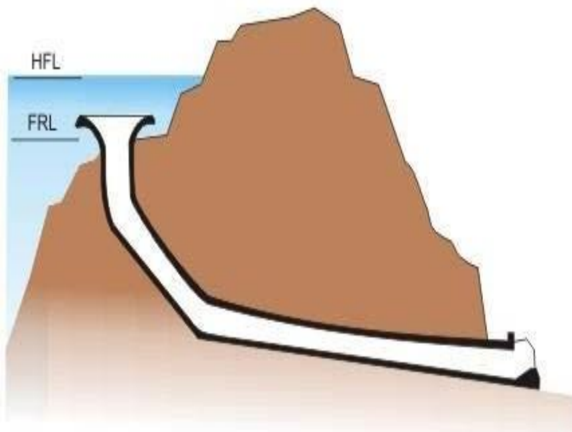


Figure 15. Tunnel spillway with a morning glory entrance.

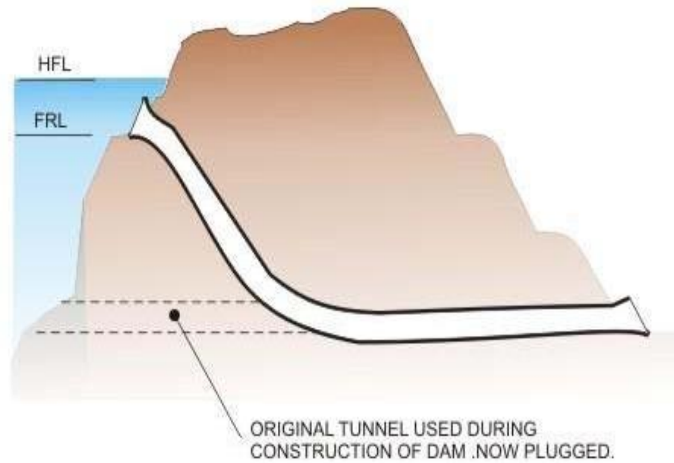


FIGURE 16. Bell mouth entry to a tunnel spillway (Gate control at entrance not shown)

Tunnel Spillway

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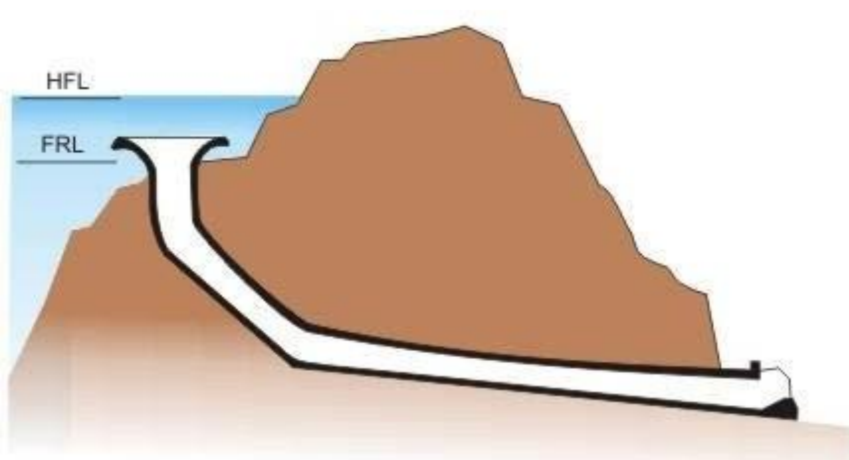


Figure 15. Tunnel spillway with a morning glory entrance.

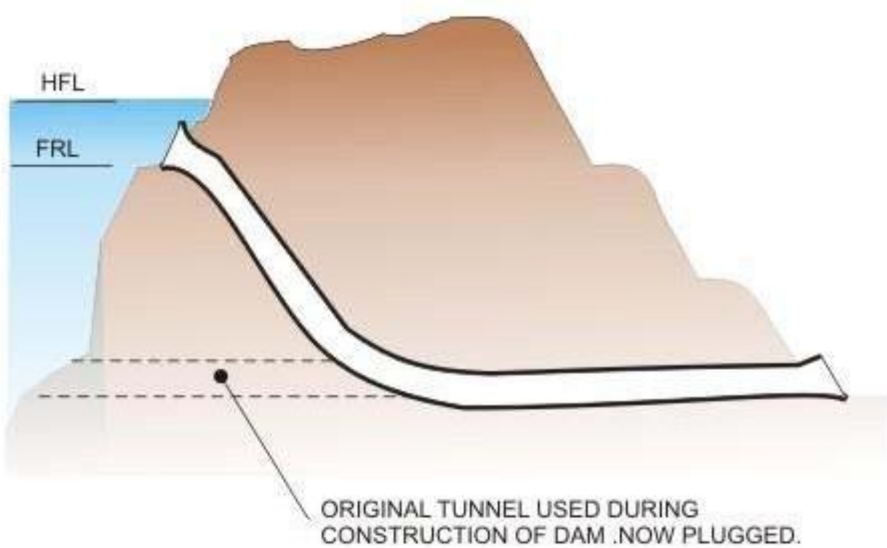


FIGURE 16. Bell mouth entry to a tunnel spillway (Gate control at entrance not shown)

Siphon Spillway

A siphon spillway is a closed conduit system formed in the shape of an inverted U, positioned so that the inside of the bend of the upper passageway is at normal reservoir storage level (Figure 17). This type of siphon is also called a Saddle siphon spillway. The initial discharges of the spillway, as the reservoir level rises above normal, are similar to flow over a weir. Siphonic action takes place after the air in the bend over the crest has been exhausted. Continuous flow is maintained by the suction effect due to the gravity pull of the water in the lower leg of the siphon.

Siphon spillways comprise usually of five components, which include an inlet, an upper leg, a throat or control section, a lower leg and an outlet. A siphon breaker air vent is also provided to control the siphonic action of the spillway so that it will cease operation when the reservoir water surface is drawn down to normal level. Otherwise the siphon would continue to operate until air entered the inlet. The inlet is generally placed well below the Full Reservoir Level to prevent entrance of drifting materials and to avoid the formation of vortices and draw downs which might break siphonic action.

Another type of siphon spillway (Figure 18) designed by Ganesh Iyer has been named after him. It consists of a vertical pipe or shaft which opens out in the form of a funnel at the top and at the bottom it is connected by a right angle bend to a horizontal outlet conduit. The top or lip of the funnel is kept at the Full Reservoir Level. On the surface of the funnel are attached curved vanes or projections called the volutes.

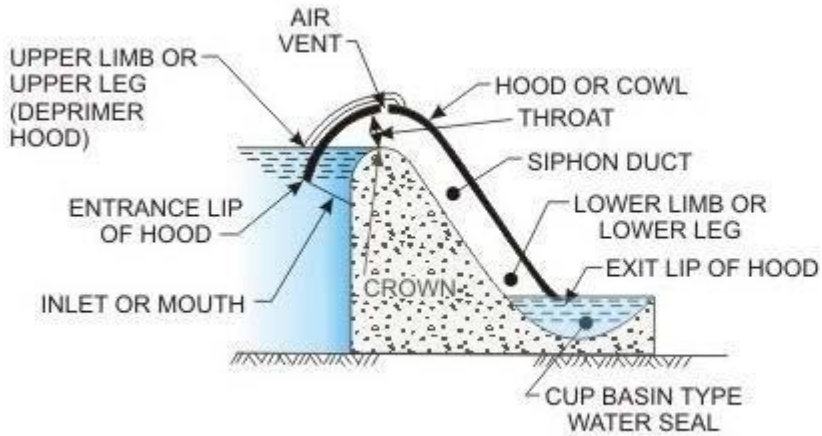


FIGURE 17. Saddle Siphon

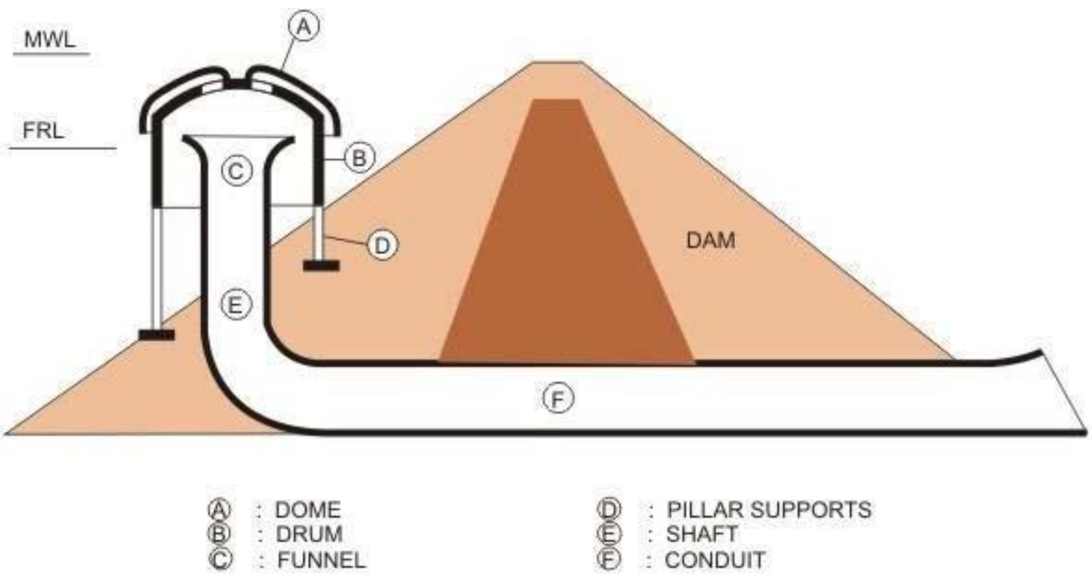


FIGURE 18. Volute of siphon spillway components

Special types of spillways

Apart from the commonly used spillways, a few other types of spillways are used sometimes for projects, which are explained below.

Saddle Spillway

In some basins formed by a dam, there may be one or more natural depressions for providing spillway. They are sometimes preferred for locating main spillway or emergency or auxiliary spillways. A site which has a saddle is very desirable and economical, if the saddle is suitable for locating the spillway. An example for such a spillway may be seen in Figure 9.

Fuse plug

It may be a simple earth bank, flash board or other device designed to fail when overtopped. Such plugs may be used where the sudden release of a considerable volume of water is both safe and not over destructive to the environment. “For example, the saddle spillway of Figure 9 may be constructed as an earthen embankment dam, with its crest at a slightly higher elevation than the High Flood Level (HFL) of the reservoir. In the occurrence of a flood greater than the design flood which may cause rise in the reservoir water would overtop the earthen embankment dam and cause its collapse and allow the flood water to safely pass through the saddle spillway.

Sluice Spillway

The use of large bottom openings as spillways is a relatively modern innovation following the greater reliance on the safety and operation of modern control gates under high pressure. A distinct advantage of this type of spillway is that provision can usually be made for its use for the passage of floods during construction. One disadvantage is that, once built, its capacity is definite whereas the forecasting of floods is still indefinite. A second disadvantage is that a single outlet may be blocked by flood debris, especially where in flow timber does not float. Figure 20 shows an example of a sluicespillway.

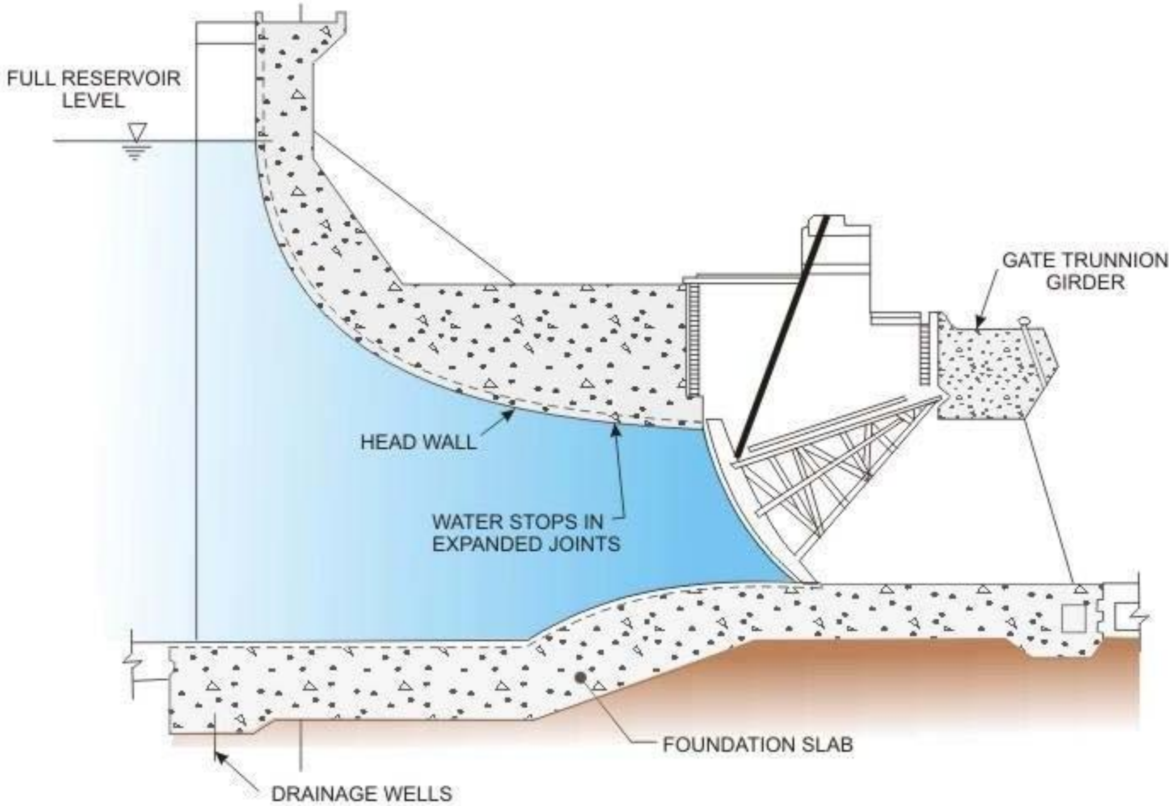


FIGURE 19. Sluice spillway

Duck-bill Spillway

This is a spillway with a rectangular layout projections into the reservoir comprising three straight overflow lengths intersecting at right angles. The layout could be trapezoidal in which case the corner angles will be other than 90 degree. The flow from the three reaches of the spillway interacts in the trough portion and is further conveyed through a discharge carrier on to a terminal structure to provide for energy dissipation. An example of this type of spillway is shown in Figure 21.

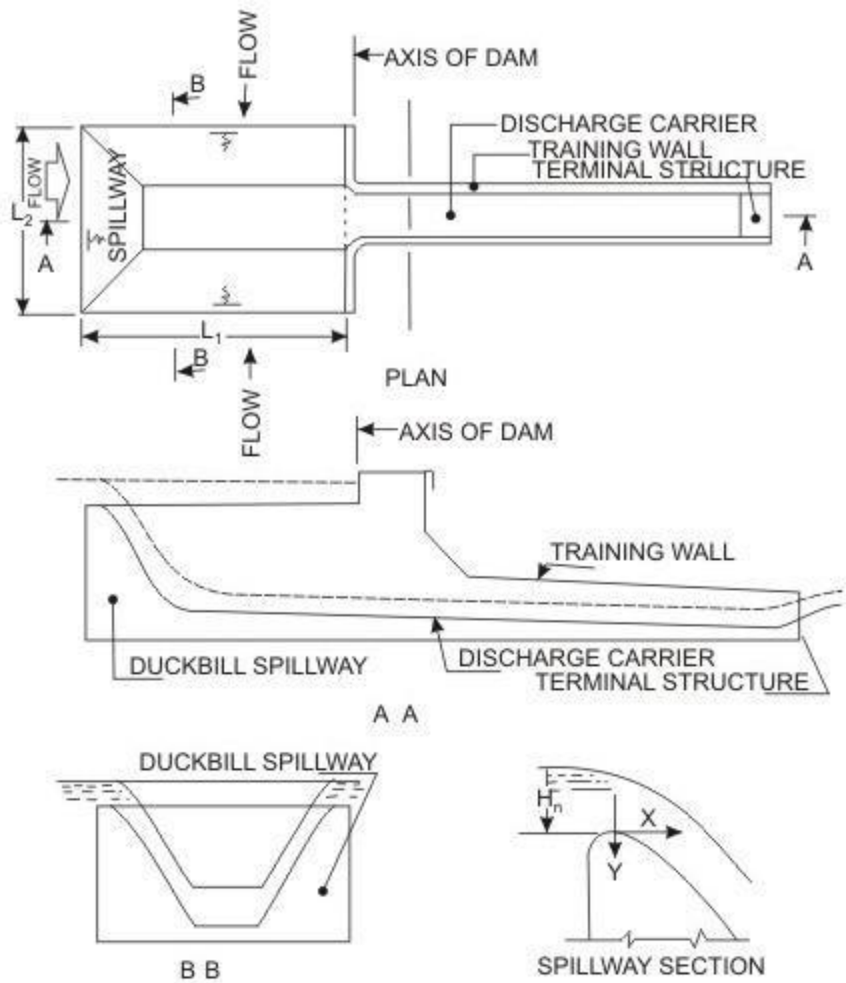


FIGURE 20. Duckbill Spillway

Shape and Hydraulics of Ogee-Crest

Crest shape

The ogee shaped crest is commonly used as a control weir for many types of spillways- Overflow (Figure 5), Chute (Figure 8), Side Channel (Figure 12) etc. The ogee shape which approximates the profile of the lower nappe of a sheet of water flowing over a sharp-crested weir provides the ideal form for obtaining optimum discharges. The shape of such a profile depends upon the head, the inclination of the upstream face of the flow section, and the height of the overflow section above the floor of the entrance channel (which influences the velocity of approach to the crest). The ogee profile to be acceptable should provide maximum possible hydraulic efficiency, structural stability and economy and also avoid the formation of sub atmospheric pressures at the surface (which may induce cavitations).

Ogee crested control structures are also sensitive to the upstream shape and hence, three types of ogee crests are commonly used and shown in Figure 21. These are as follows:

1. Ogee crests having vertical upstream face
2. Ogee crests having inclined upstream face
3. Ogee crests having over hang on up stream face

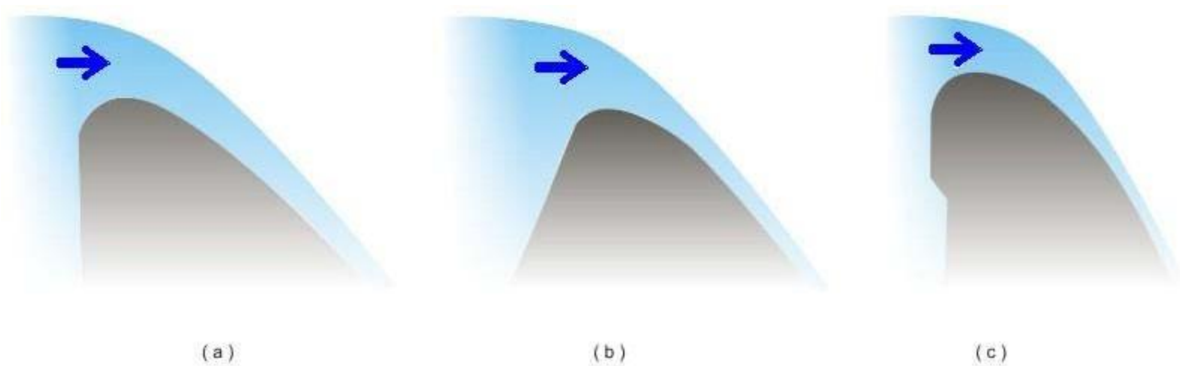


FIGURE 21. Ogee crest control weirs with
(a) Vertical upstream face
(b) Inclined upstream face
(c) Overhangs on the upstream

However, the same general equations for the up stream and down stream quadrants are applicable to all the three cases, as recommended by the Bureau of Indian Standards code IS: 6934-1998 “Hydraulic design of high ogee over flow spillways- recommendations” and are outlined in the following paragraphs.

1. *Ogee crests with vertical upstream face*

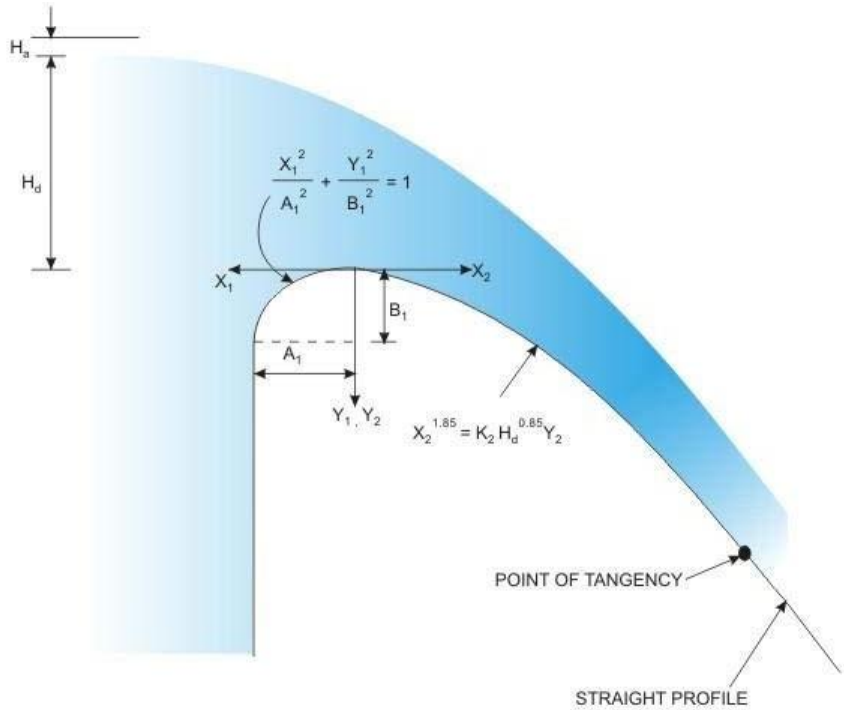


FIGURE 22. Ogee crest shape with vertical upstream wall . Coefficients to be determined from Figure 21

The upstream quadrant of the crest (Figure 22) may confirm to the equation of an ellipse as given below:

$$\frac{x_1^2}{A_1^2} + \frac{y_1^2}{B_1^2} = 1 \qquad (1)$$

Where the values of A_1 and B_1 may be determined from the graphs given in (Figure 23).

The downstream profile of the ogee crest may confirm to the following equation:

$$x_2^{1.85} = K_2 H_d^{0.85} Y_2 \qquad (2)$$

Where the magnitude of K_2 may be read from the relevant graph shown in Figure 23.

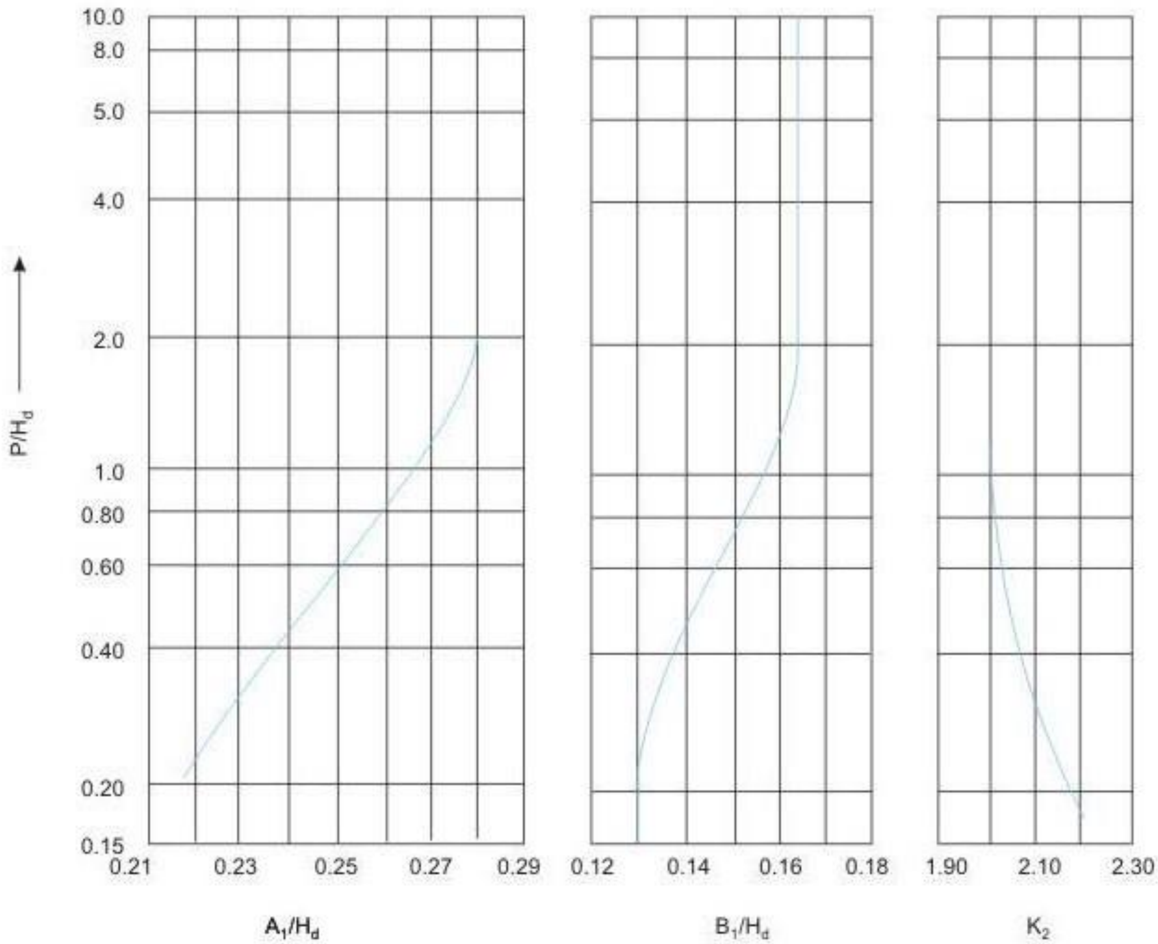


FIGURE 23. Coefficients for Figure 22

2. *Ogee crests with sloping up stream face*

In this case, the desired inclination of the upstream face is made tangential to the same elliptical profile as provided for a crest with a vertical face. The down stream face equation remains unchanged.

3. Ogee crests with overhang

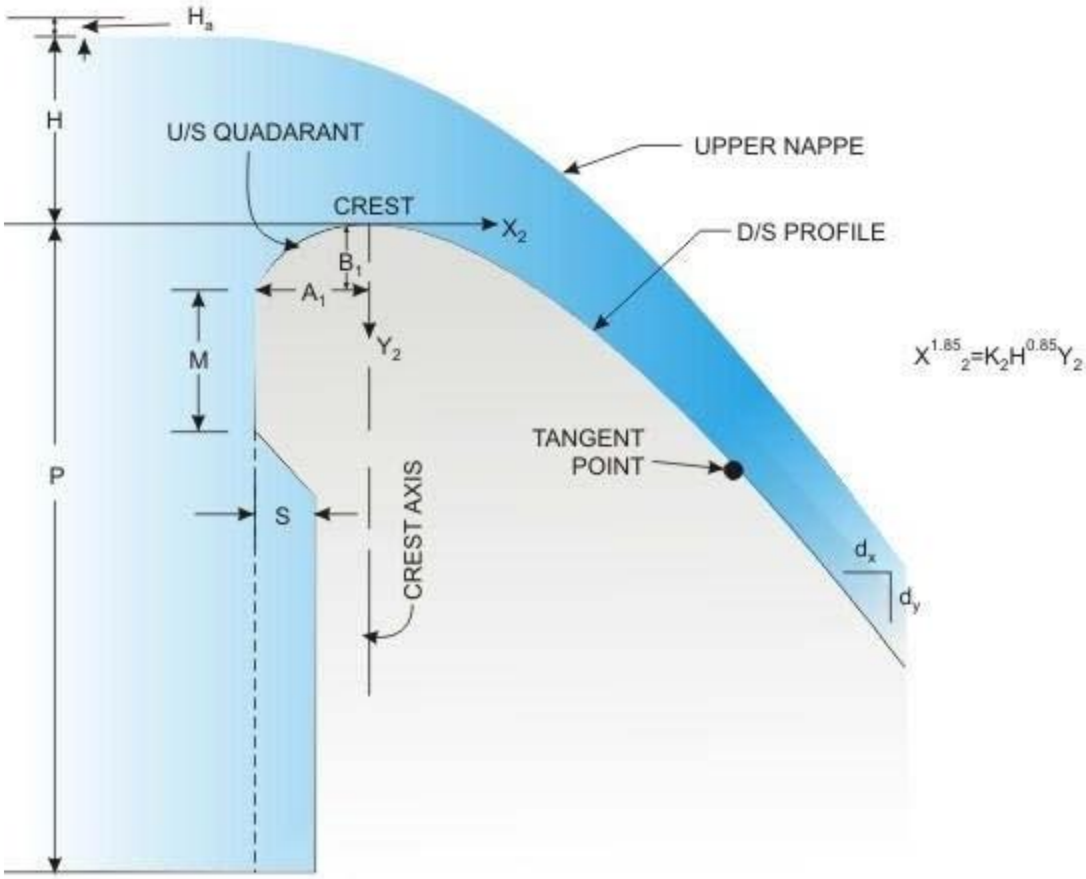


FIGURE 24. Overhang details of ogee crest

Whenever structural requirements permit, the upstream vertical face of an ogee crested spillway (Figure 22) may be offset inside, (Figure 24). It is recommended that the ratio of the rises M to the design head H_d, should be at least 0.6 or greater for flow conditions to be stable. The crest shapes on the up stream and downstream may be provided the same as for an ogee crest with vertical up stream wall if the condition M/ H_d >0.6 is satisfied.

Discharge characteristics of ogee crests-uncontrolled flow

For an ogee crested control weir for a spillway without any control with a gate, the free flow discharge equation is given as

$$Q = C_d L_e H_e^{3/2} \qquad (3)$$

Where Q is the discharge (in m^3/s), C_d is the coefficient of discharge, L_e is the effective length of crest (in m), including velocity of approach head. The discharge coefficient, $C_{d,}$ is influenced by a number of factors, such as:

1. Depth of approach
2. Relation of the actual crest shape to the ideal nappe shape
3. Upstream face slope
4. Downstream apron interference, and
5. Downstream submergence

The effect of the above mentioned factors on the variation of discharge and calculation for effective length are mentioned in the following paragraphs.

1. Effect of depth of approach

For a high sharp-crested ogee shaped weir, as that of a Overfall spillway of a large dam, the velocity of approach is small and the lower nappe flowing over the weir attains maximum vertical contraction. As the approach depth is decreased, the velocity of approach increases and the vertical contraction diminishes. For sharp-crested weirs whose heights are not less than about one-fifth of the head producing the flow, the coefficient of discharge remains fairly constant with a value of about 1.82 although the contraction diminishes. For weir heights less than about one-fifth the head, the contraction of the flow becomes increasingly suppressed and the crest coefficient decreases. This is the case of an ogee crested chute spillway control section. When the weir height becomes zero, the contraction is entirely suppressed and the weir turns into a broad crested one, for which the theoretical coefficient of discharge is 1.70. The relationship of the ogee crest coefficient of discharge C_d for various values of P/H_d where P is the height of the weir above base and H_d is the design head, is given in Figure 25. The coefficients are valid only when ogee is formed to the ideal nappe shape.

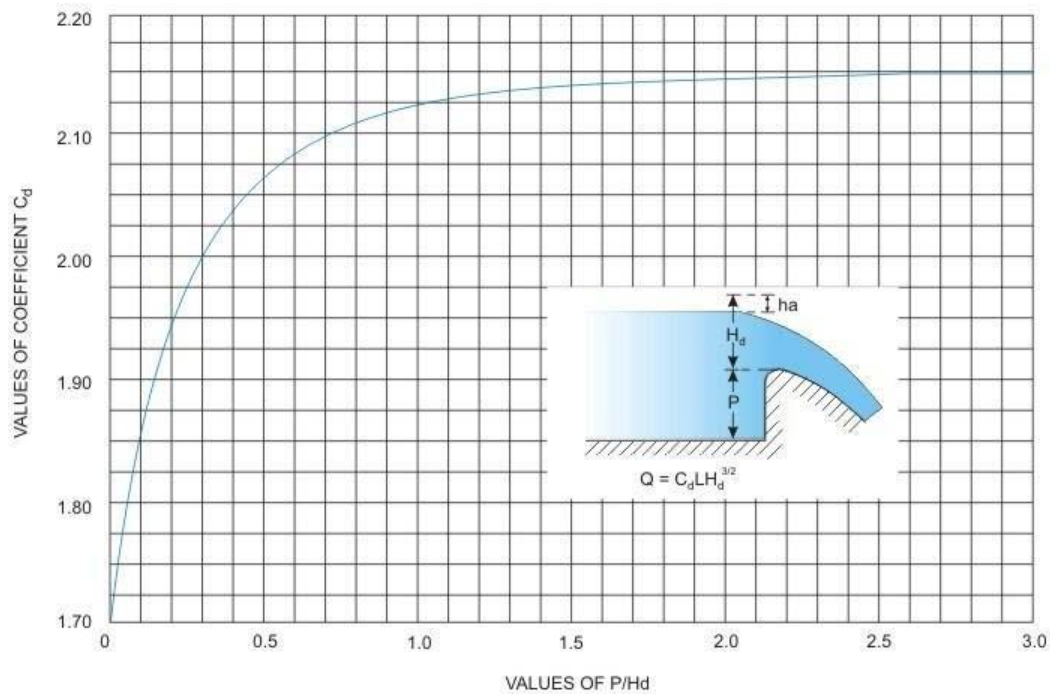


FIGURE 25. Coefficient of discharge (C_d) variation due to height of a vertical faced ogee crest

2. Effect of the crest shape differing from the ideal nappe shape

When the ogee crest is formed to a shape differing from the ideal nappe shape or when the crest has been shaped for a head larger or smaller than the one under consideration, the coefficient of discharge will differ from that given in the previous section. A wider crest shape will reduce the coefficient of discharge while a narrower Crest Shape will reduce the coefficient. The application of this concept is required to deduce the discharge flowing over a spillway when the flow is less or more than the design discharge. The variation of the coefficient of discharge in relation to H/H_d, where H is the actual head and H_d is the design head, is shown in Figure 26.

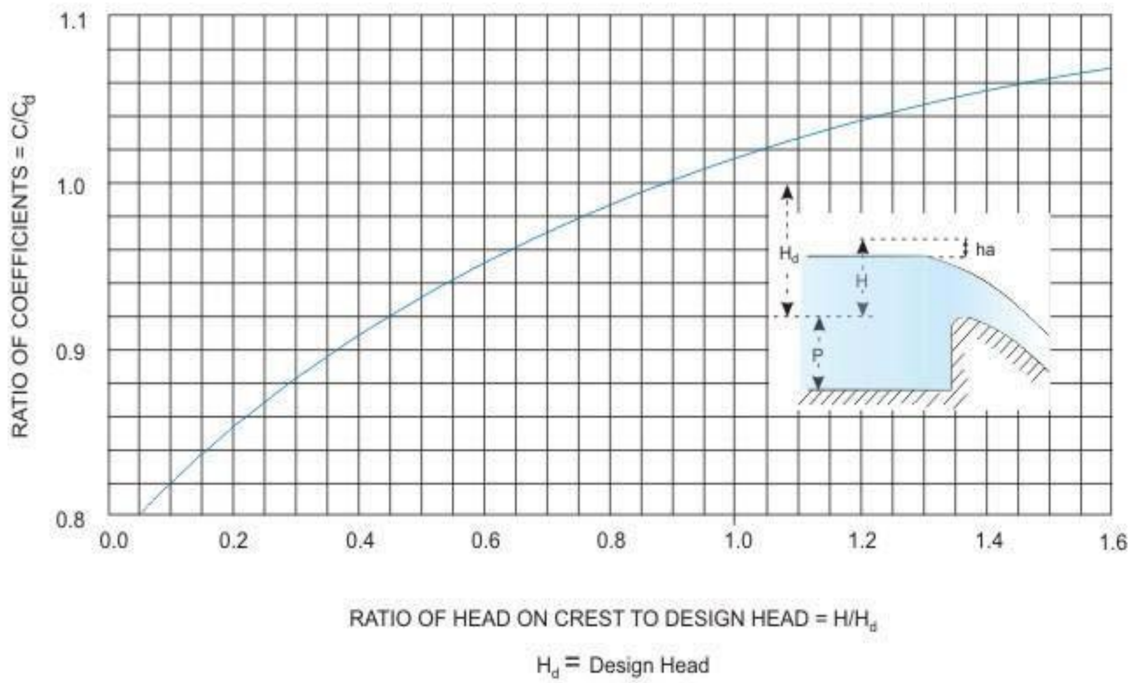


FIGURE 26. Coefficient of discharge other than the design head

3. Effect of upstream face slope

For small ratio of P/H_d where P is the height of the weir and H_d the design head, as for the approach to a chute spillway, increase of the slope of upstream face tends to increase the coefficient of discharge, as shown in Figure 27. This figure shows the ratio of the coefficient for ogee crest with a sloping face to that with vertical face. For large ratios of P/H_d, the effect is a decrease of the coefficient. The coefficient of discharge is reduced for large ratios P/H_d only for relatively flat upstream slopes.

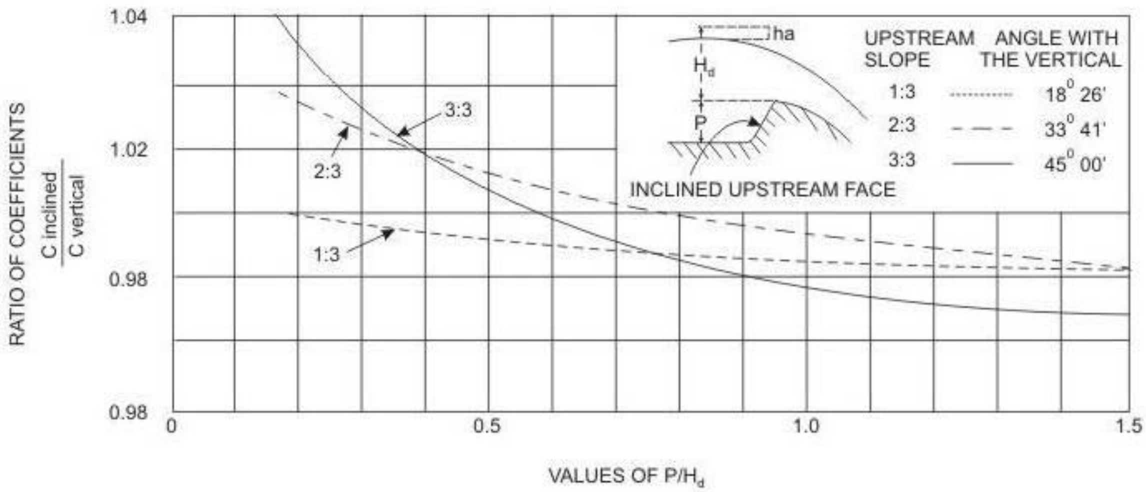


Figure 27. Coefficient of discharge variation with upstream face inclination.

4. Effect of downstream apron interference and downstream submergence

This condition is possible for dams of relatively small heights compared to the natural depth of the river, when the water level downstream of the weir crest is high enough to affect the discharge, the condition being termed as submerged. The conditions that after the coefficient of discharge in this case are the vertical distance from the crest of the over flow to the downstream apron and the depth of flow in the downstream channel, measured above the apron.

Five distinct characteristic flow conditions can occur below an overflow crest, depending on the relative positions of the apron and the downstream water surface:

A. The flow will continue at supercritical stage
B. A partial or incomplete hydraulic jump will occur immediately downstream from the crest
C. A true hydraulic jump will occur
D. A drowned jump will occur in which the high-velocity jet will follow the face of the overflow and then continue in an erratic and fluctuating path for a considerable distance under and through the slower water, and
E. No jump will occur - the jet will break away from the face of the overflow and ride along the surface for a short distance and then erratically intermingle with the slow moving water underneath.

According to USBR (1987), the relationship of the floor positions and downstream submergences which produce these distinctive flows can be shown in a graph as in Figure 28.

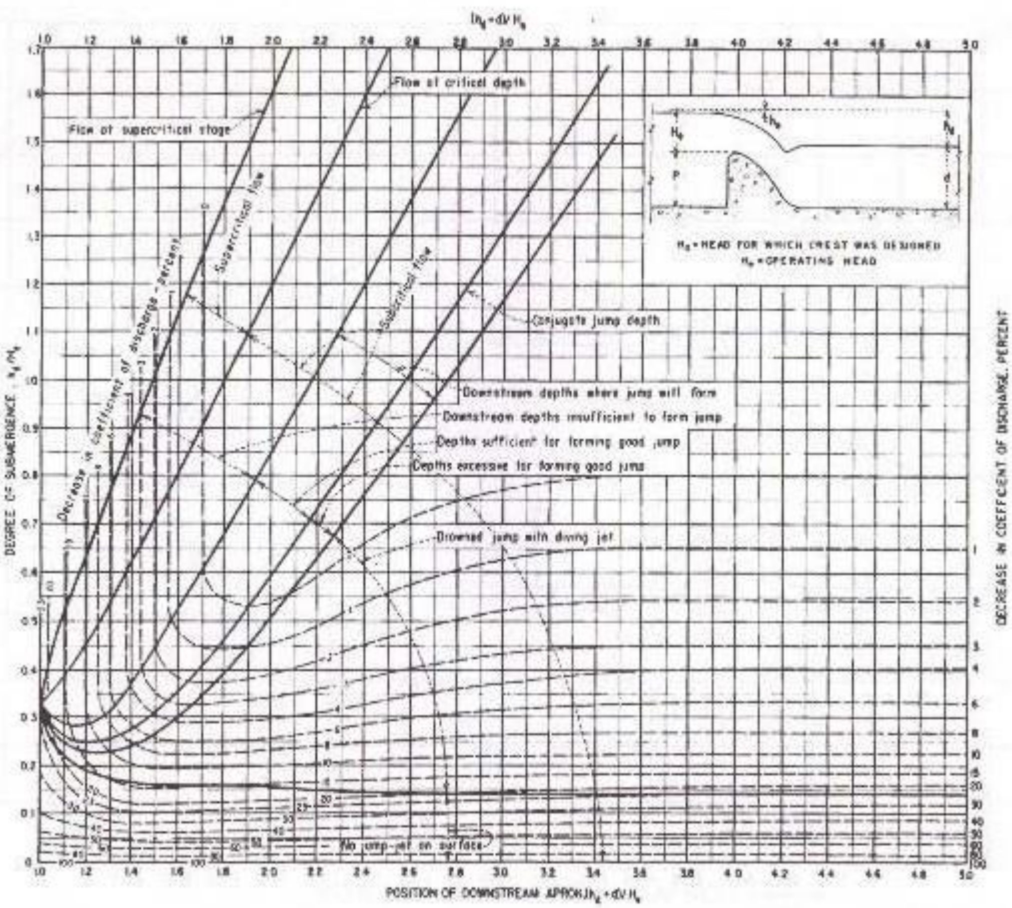


FIGURE 28. Effects of downstream influences on flow over weir crests

(Courtesy: United States Bureau of Reclamation, Design of Small Dams)

Usually for large dams the cases A,B or C with dominate and the decrease in the coefficient of discharge is due principally to the back pressure effect of the downstream apron and is independent of any submergence effect due to tail water. Cases D and E can be expected t o be found in low-height dams like small height diversion or navigation dam. Figure 29, adapted from USBR (1987), shows the effect of downstream apron conditions on the coefficient of discharge. It may be noted that this curve plots the same data represented by the vertical dashed lines of Figure 28 in a slightly different form. As the downstream apron level nears the crest of the overflow ($\frac{h_d + d}{H_e}$ approaches 1.0), where h_d is the difference of total energy on upstream and the water level downstream, d is the downstream water depth and H_e is the total energy upstream measured above the crest of the weir, the coefficient of discharge is about 77 percent of that for un-retarded flow. From Figure 29 it can be seen that when the ratio of $\frac{h_d + d}{H_e}$ values exceed about 1.7, the downstream floor position has little effect on the coefficient, but there is a decrease in the coefficient caused by tail water submergence. Figure 30 shows the ratios of the coefficient of discharge where affected by tailwater conditions, to that coefficient for free flow conditions. This curve plots the data represented by the horizontal dashed lines on Figure 28 in a slightly different form. Where the dashed lines of Figure 28 are curved, the decrease in the coefficient is the result of a combination of tail-water effects and downstream apron position.

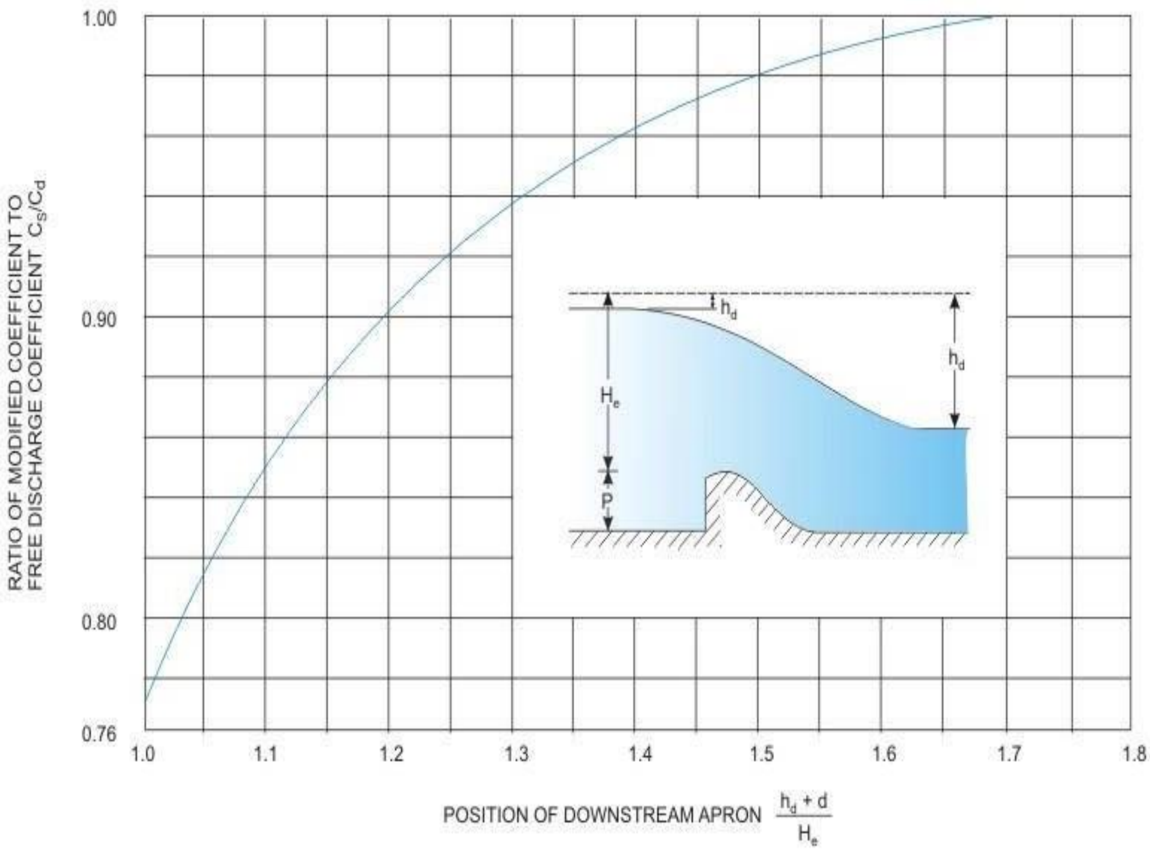


FIGURE 29. Ratio of discharge coefficients resulting from the effect of the apron on flow

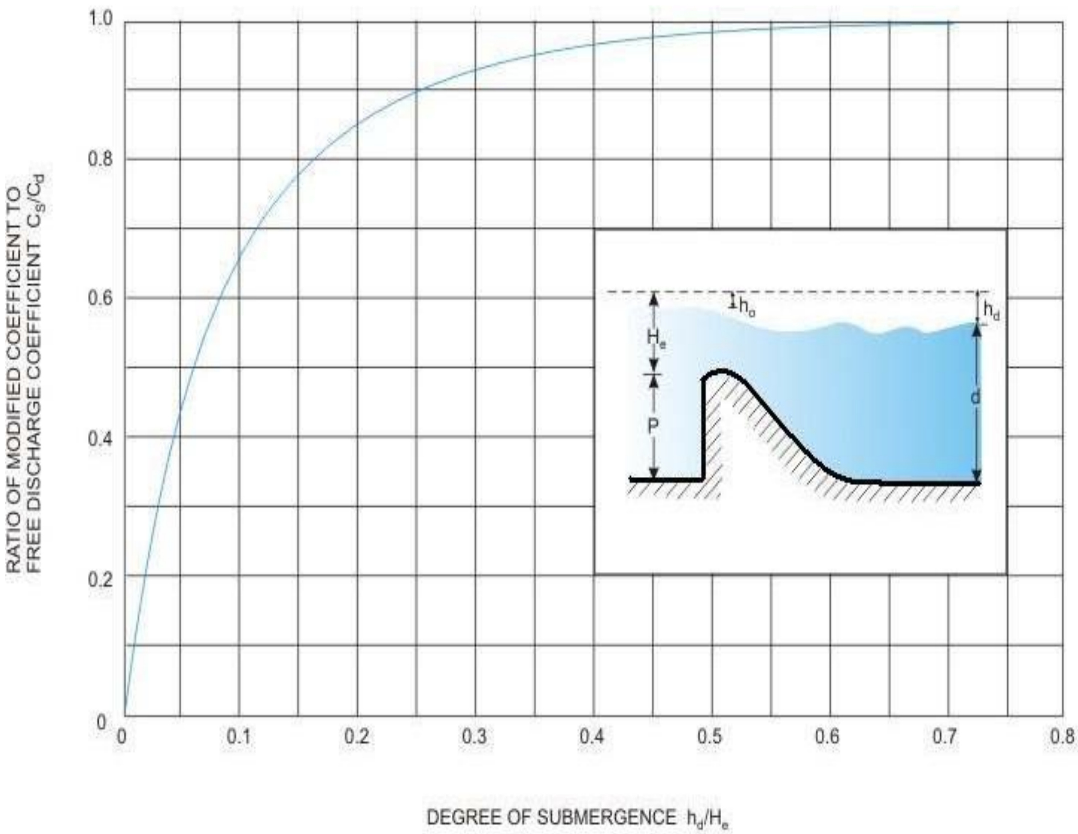


FIGURE 30. Ratio of discharge coefficients caused by tailwater effects

If the ordinate of Figure 30 is changed from $\frac{h_d}{H_e}$ to $1 - \frac{h_d}{H_e}$, that is, equal to $\frac{H_e - h_d}{H_e} = \frac{h}{H_e}$ where *h* is the downstream water depth measured above crest, then the curve of Figure 30 may be transposed as in Figure 31.

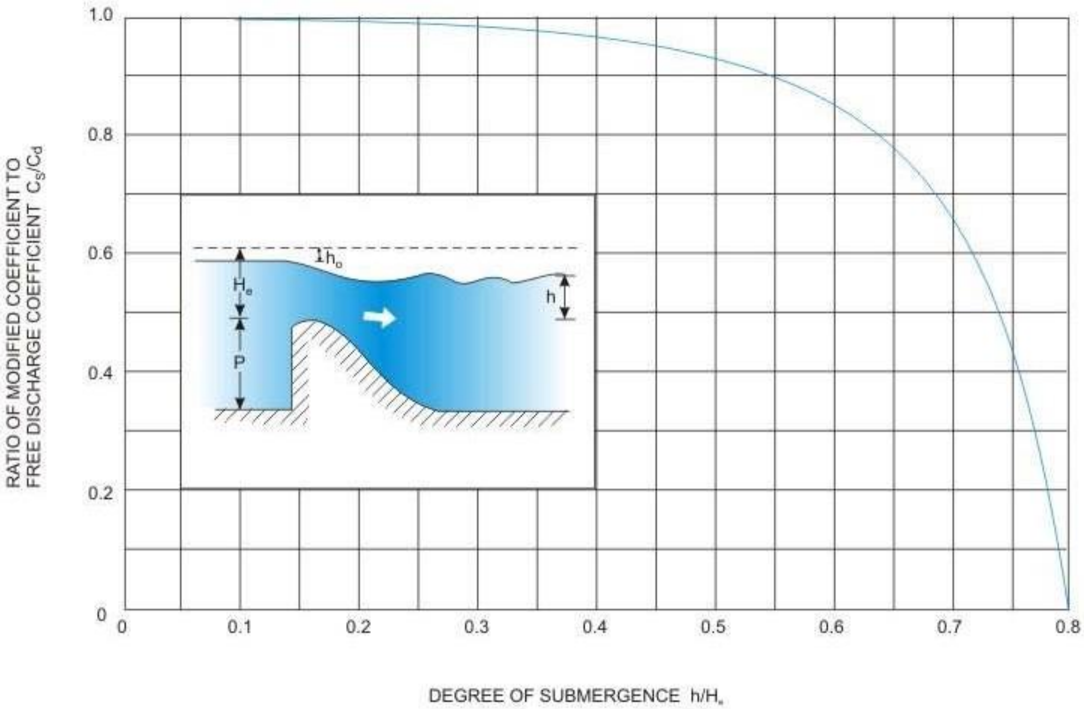


FIGURE 31. Ratio of submerged discharge coefficient to that without submergence effect
This figure is similar to that given in FIGURE 30, but with different ordinate value

The total head on the crest He, does not include allownces for approach channel friction losses due to curvature into the inlet section, and inlet or transition losses. Where the design of the approach channel results in appreciable losses, they must be added to H_e to determine reservoir elevations corresponding to the discharges given by the discharge equation.

Where the crest piers and abutments are shaped to cause side contractions of the overflow, the effective length, L_e, will be the net length of the crest, L. The effect of the end contractions may be taken into account by reducing the net length of crest as given below:

$$L_e = L - 2(NK_p + K_a)H_e$$

Where L, L_e and H have been explained before, N is the number of piers and K_p and K_a are the pier and abutment contraction coefficients. The reason for the reduction of the net length may be appreciated from Figure 32.

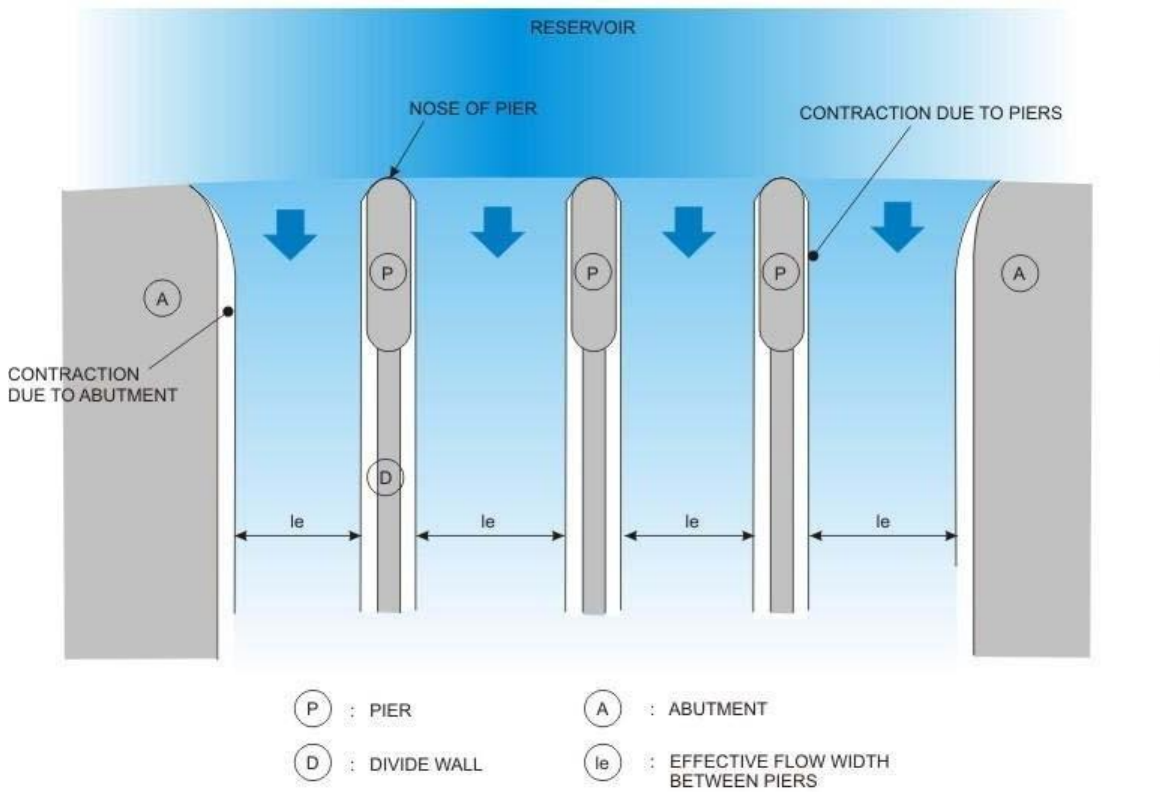


FIGURE 32. Abutment and pier contractions shown on a spillway plan

The pier contraction coefficient K_p depends upon the following factors:

1. Shape and location of the pier nose
2. Thickness of the pier
3. Head in relation to the design head
4. Approach velocity

For the condition of flow at the design head, the average values of pier contraction coefficients may be assumed as shown in Figure 33.

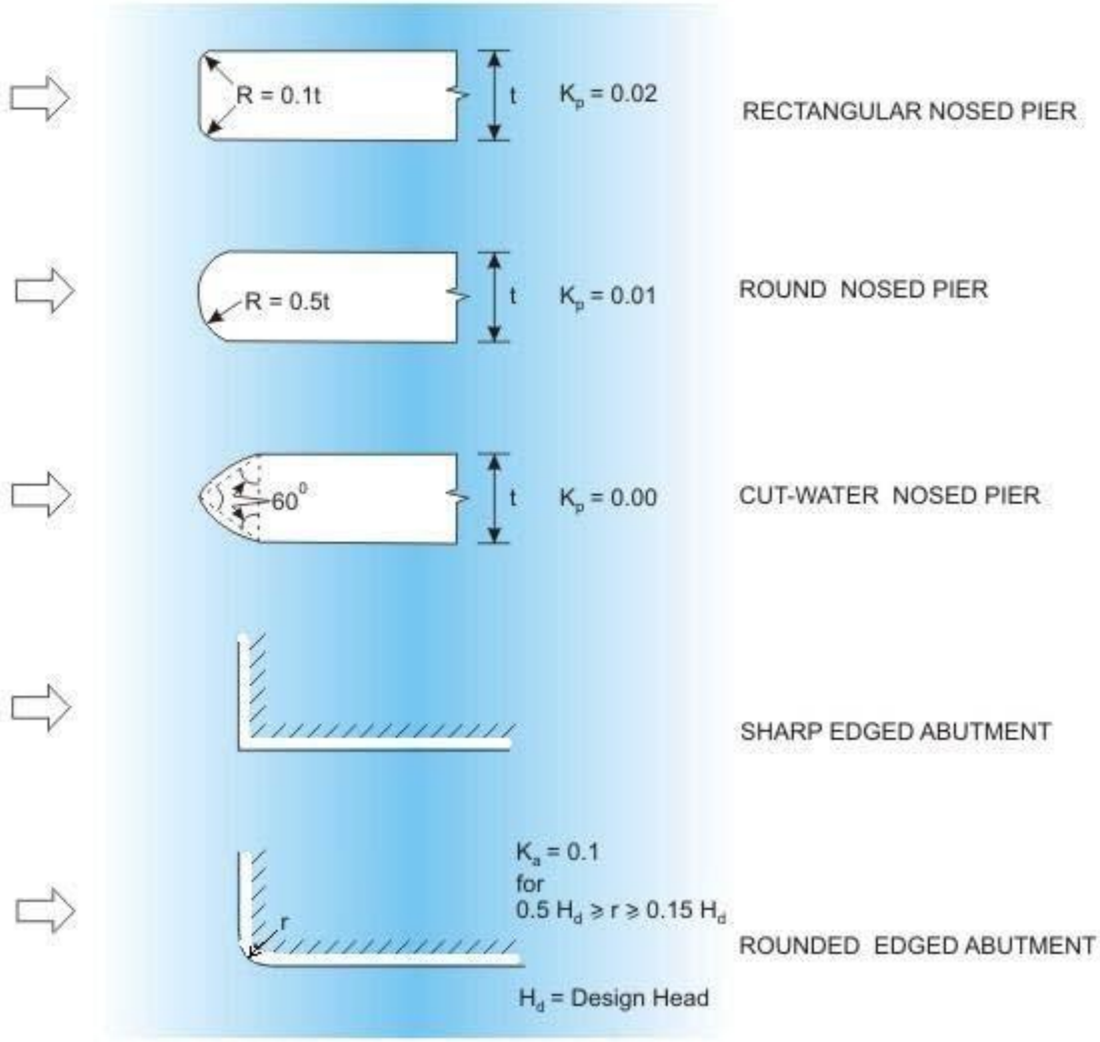


FIGURE 33. Recommended values of K_p and K_a

The abutment contraction coefficient is seen to depend upon the following factors:

1. Shape of abutment
2. Angle between upstream approach wall and the axis of flow
3. Head, in relation to the design head
4. Approach velocity

For the condition of flow at the design head, the average value of abutment contraction coefficients may be assumed as shown in Figure 33.

For flow at head other than design head, the values of K_p and K_a may be obtained from graphical plots given in IS: 6934-1973 "Recommendations for hydraulic design of high ogee overflow spillways".

Discharge characteristics of ogee crests-controlled spillway

The discharge for gated crests at partial gate opening is similar to flow through a low- head orifice and may be computed by the following equation recommended by the Bureau of Indian Standards code IS:6934-1998 "Hydraulic design of high ogee overflow spillways-recommendations".

$$Q = C_g \, G_0 \, L_e \sqrt{2g} \, H_e \qquad (4)$$

Where Q is the discharge (in m^3/s), C_g is the gated coefficient of discharge, G_0 is the gate opening (in m), L_e is the effective length of crest, g is the acceleration due to gravity, and H_e is the hydraulic head measured from the centre of the orifice (in m).

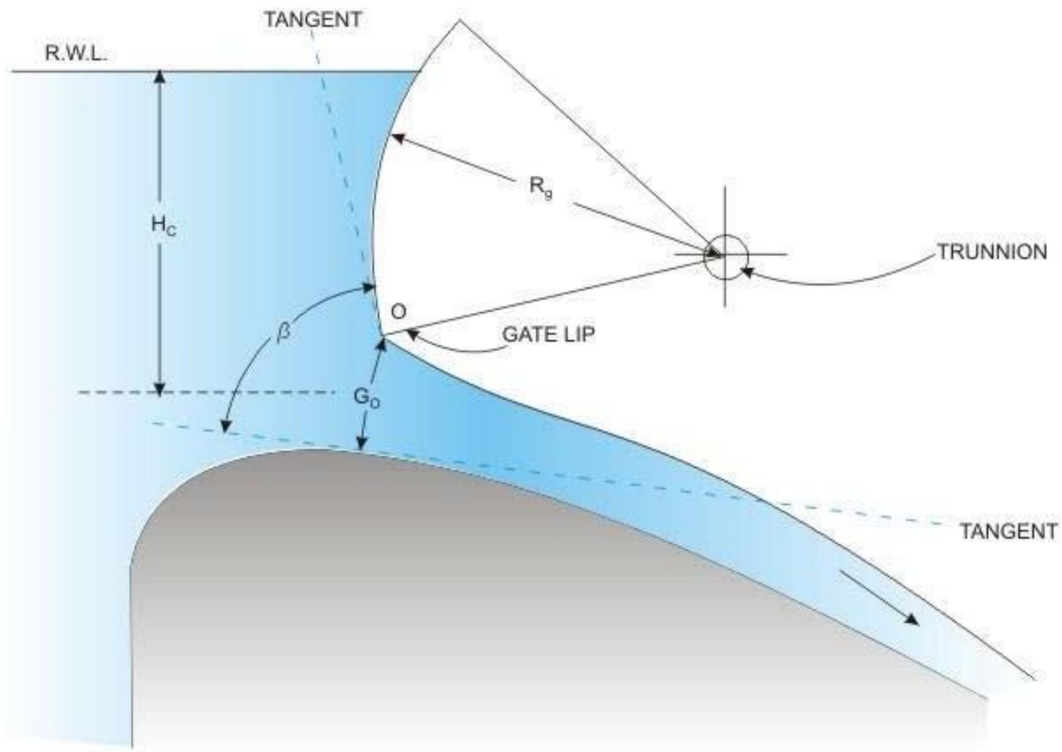


FIGURE 34. Partially opened radial gate discharging flow

Usually for high head spillways, radial gates are common and Figure 34 shows the position of a partially opened radial gate over an ogee-crested spillway. The gate opening G_0 may be seen to be measured as the shortest distance from the gate lip to the ogee crest profile meeting at G. The angle β is seen to be measured between the tangent at G and the tangent of the radial gate at gate lip. Figure 35 presents a curve relating the coefficient of gated discharge C_g with the angle β.

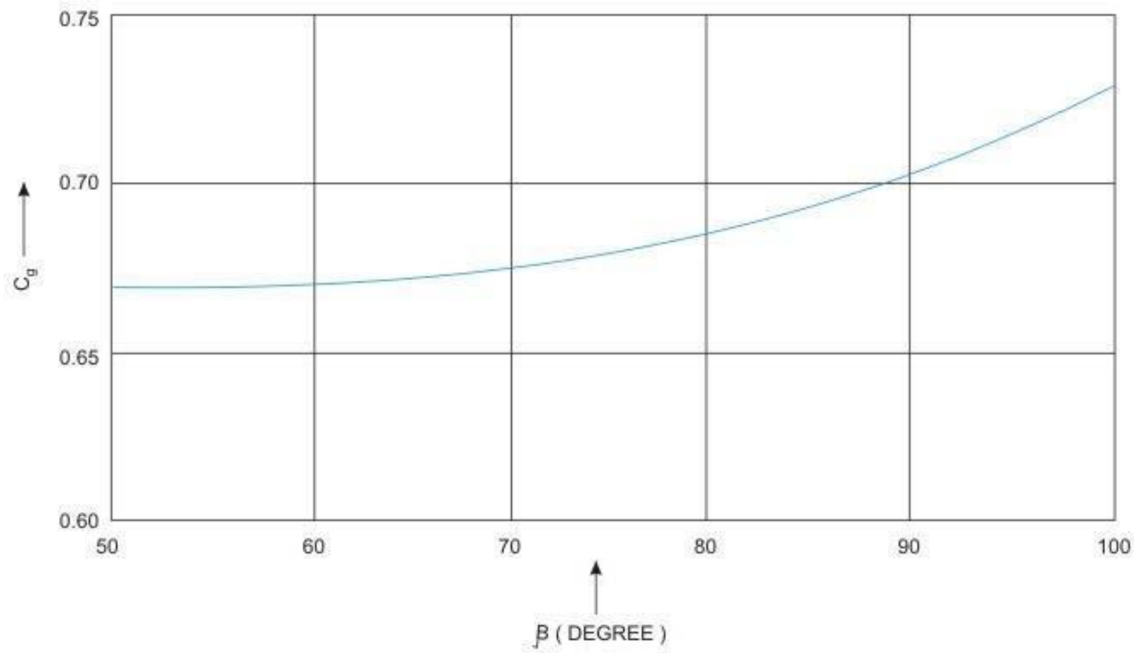


FIGURE 35. Coefficient of gated discharge Cg variation with lip angle β

The curve presents an average value of C_g determined for various approach and downstream conditions and may be used for preliminary design purposes. In fact, it may be noticed that the discharge equation mentioned above for calculating flow through a gated spillway as recommended in IS: 6934-1998 may not be strictly correct as the gate opening becomes larger, comparable to the hydraulic head H_c.

Spillway profile with breast wall

Spillways, generally the ogee-crested type, are sometimes provided with a breast wall from various considerations such as increasing the regulating storage of flood discharge, reducing the height of the gate, minimizing the cost of gate operating mechanism, etc.

For the spillways with breast wall, the following parameters are required to be determined:

a) Profile of the spillway crest including the upstream and downstream quadrants,
b) Profile of the bottom surface of the breast wall, and
c) Estimation of discharge efficiency of the spillway.

The flow through a spillway with breast wall has been idealised as two-dimensional flow through a sharp edged orifice in a large tank. The following guidelines for determining the parameters mentioned above may be used for preparing preliminary designs and studies on hydraulic model may be conducted for confirming or improving on the preliminary design. Figure 36 shows pertinent details of various profiles of the spillway with a breast wall.

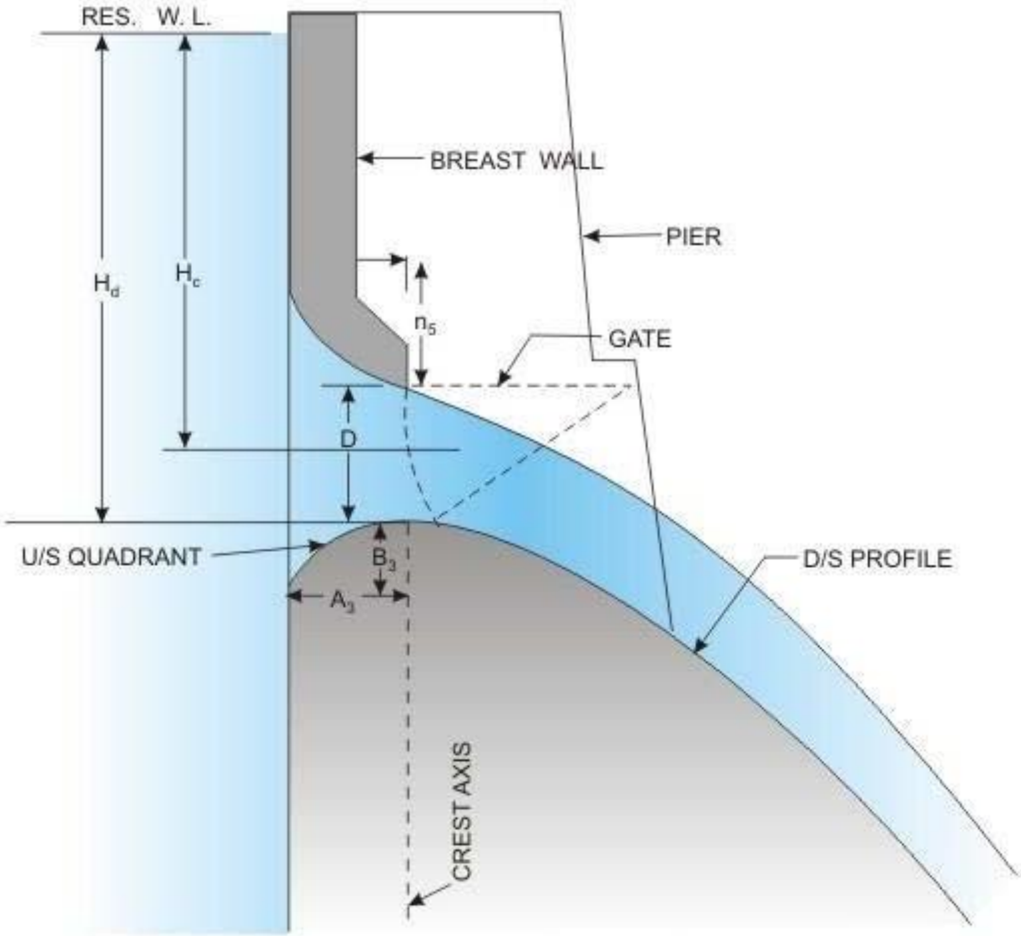


FIGURE 36. Spillway with breast wall

Ogee Profile - Upstream Quadrant'

The upstream quadrant may conform to an ellipse with the equation:

$$\frac{X^2}{A_3^2} + \frac{Y^2}{B_3^2} = 1 \qquad (5)$$

where

$A_3 = 0.541\, D\, (H_d/D)^{0.32}$ and

$B_3 = 0.3693\, D\, (H_d/D)^{0.04}$

Ogee Profile - Downstream profile

The downstream profile may conform to equation:

$$X_4^{n_4} = K_4 H_d^{n_4 - 1} Y_4 \tag{6}$$

where

$$K_4 = 0.04 + 0.025 \frac{H_d}{D} \tag{7}$$

and

$$n_4 = 1.782 - 0.0099 \left(\frac{H_d}{D} - 1 \right) \tag{8}$$

Bottom Profile of the Breast Wall

The bottom profile of the breast wall may conform to the equation:

$$Y_5 = \frac{K_5}{n_5^{2.4}} X_5^{2.4} \tag{9}$$

where

$$K_5 = 0.541 D \left(\frac{H_d}{D} \right)^{0.32} \tag{10}$$

$$n_5 = 0.4D \tag{11}$$

The upstream edge of the breast wall is in line with the upstream edge of the spillway and the downstream edge is in line with the spillway crest axis, as shown in Figure 36.

The details of the upstream curve of the crest and bottom profile of breast wall are shown in Figure 37.

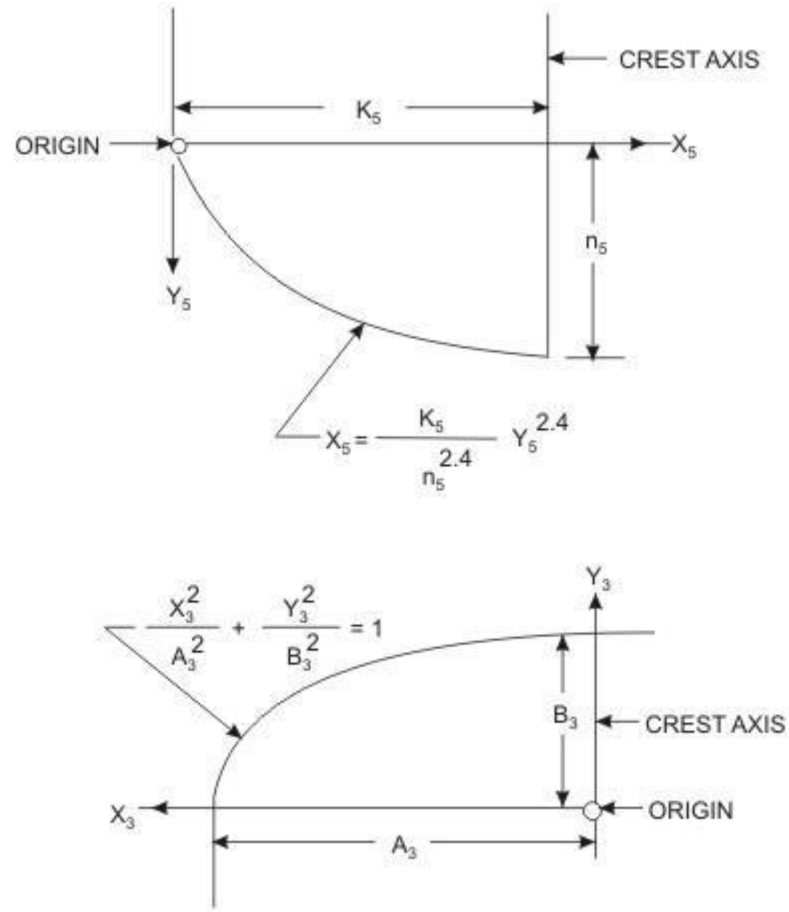


FIGURE 37. Upstream profile of ogee crest and bottom profile of breast wall details

Discharge Computation

The discharge through the breast wall spillway may be estimated by the equation:

$$Q = C_b \, L \, D \left\{ 2g \left(H_c - \frac{a}{2} + \frac{V_a^2}{2g} \right) \right\}^{0.5} \tag{12}$$

The following equation relates C_h with the parameter (H/H_d) in the range of H/H_d = 0.8 to 1.33.

$$C_b = 0.148631 + 0.945305(H/H_d) - 0.326238(H/H_d)^2 \tag{13}$$

Typical values of C_b are:

H/H_d	C_b

0.80	0.696
1.00	0.769
1.15	0.797
1.33	0.829

Selection of spillways

The Bureau of Indian Standards code IS: 10137-1982 "Guidelines for selection of spillways and energy dissipators" provide guidelines in choosing the appropriate type of spillway for the specific purpose of the project. The general considerations that provide the basic guidelines are as follows:

Safety Considerations Consistent with Economy

Spillway structures add substantially to the cost of a dam. In selecting a type of spillway for a dam, economy in cost should not be the only criterion. The cost of spillway must be weighed in the light of safety required below the dam.

Hydrological and Site Conditions

The type of spillway to be chosen shall depend on:

a) Inflow flood;
b) Availability of tail channel, its capacity and flow hydraulics;
c) Power house, tail race and other structures downstream; and
d) Topography

Type of Dam

This is one of the main factors in deciding the type of spillway. For earth and rockfill dams, chute and ogee spillways are commonly provided, whereas for an arch dam a free fall or morning glory or chute or tunnel spillway is more appropriate. Gravity dams are mostly provided with ogee spillways.

Purpose of Dam and Operating Conditions

The purpose of the dam mainly determines whether the dam is to be provided with a gated spillway or a non-gated one. A diversion dam can have a fixed level crest, that is, non-gated crest.

Conditions Downstream of a Dam

The rise in the downstream level in heavy floods and its consequences need careful consideration. Certain spillways alter greatly the shape of the hydrograph downstream of a dam. The discharges from a siphon spillway may have surges and break-ups as priming and depriming occurs. This gives rise to the wave travelling downstream in the river, which may be detrimental to navigation and fishing and may also cause damage to population and developed areas downstream.

Nature and Amount of Solid Materials Brought by the River

Trees, floating debris, sediment in suspension, etc, affect the type of spillway to be provided. A siphon spillway cannot be successful if the inflow brings too much of floating materials. Where big trees come as floating materials, the chute or ogee spillway remains the common choice.

Apart from the above, each spillway can be shown as having certain specific advantages under particular site conditions. These are listed below which might be helpful to decide which spillway to choose for a particular project.

Ogee Spillway

It is most commonly used with gravity dams. However, it is also used with earth and rockfill dams with a separate gravity structure; the ogee crest can be used as control in almost all types of spillways; and it has got the advantage over other spillways for its high discharging efficiency.

Chute Spillway

a) It can be provided on any type of foundation,
b) It is commonly used with the earth and rockfill dams, and
c) It becomes economical if earth received from spillway excavation is used in dam construction.

The following factors limit its adaption:
a) It should normally be avoided on embankments;
b) Availability of space is essential for keeping the spillway basins away from the dam paving; and
c) If it is necessary to provide too many bends in the chute because of the topography, its hydraulic performance can be adversely affected.

Side Channel Spillways

This type of spillway is preferred where a long overflow crest is desired in order to limit the intensity of discharge, It is useful where the abutments are steep, and it is useful where the control is desired by the narrow side channel.

The factor limiting its adoption is that this type of spillway is hydraulically less efficient.

Shaft Spillways (Morning Glory Spillway)

a) This can be adopted very advantageously in dam sites in narrow canyons, and
b) Minimum discharging capacity is attained at relatively low heads. This characteristic makes the spillway ideal where the maximum spillway outflow is to be limited. This characteristic

becomes undesirable where a discharge more than the design capacity is to be passed. So, it can be used as a service spillway in conjunction with an emergency spillway.

The factor limiting its adoption is the difficulty of air-entrainment in a shaft, which may escape in bursts causing an undesirable surging.

Siphon Spillway

Siphon spillways can be used to discharge full capacity discharges, at relatively low heads, and great advantage of this type of spillway is its positive and automatic operation without mechanical devices and moving parts.

The following factors limit the adoption of a siphon spillway:

It is difficult to handle flows materially greater than designed capacity, even if the reservoir head exceeds the design level; Siphon spillways cannot pass debris, ice, etc; There is possibility of clogging of the siphon passage way and breaking of siphon vents with logs and debris; In cold climates, there can be freezing inside the inlet and air vents of the siphon; When sudden surges occur and outflow stops; The structure is subject to heavy vibrations during its operation needing strong foundations; and Siphons cannot be normally used for vacuum heads higher than
8 m and there is danger of cavitation damage.

Overfall or Free Fall Spillway

This is suitable for arch dams or dams with downstream vertical faces; and this is suitable for small drops and for passing any occasional flood.

Tunnel or Conduit Spillway

This type is generally suitable for dams in narrow valleys, where overflow spillways cannot be located without risk and good sites are not available for a saddle spillway. In such cases, diversion tunnels used for construction can be modified to work as tunnel spillways. In case of embankment dams, diversion tunnels used during construction may usefully be adopted. Where there is danger to open channels from snow or rock slides, tunnel spillways are useful.

Energy dissipators

Different types of energy dissipators may be used along with a spillway, alone or in combination of more than one, depending upon the energy to be dissipated and erosion control required downstream of a dam. Broadly, the energy dissipators are classified under two categories – Stilling basins or Bucket Type. Each of these are further sub- categorized as given below.

Stilling basin type energy dissipators

They may fundamentally be divided into two types.

a) Hydraulic jump type stilling basins

1. Horizontal apron type (Figure 38)

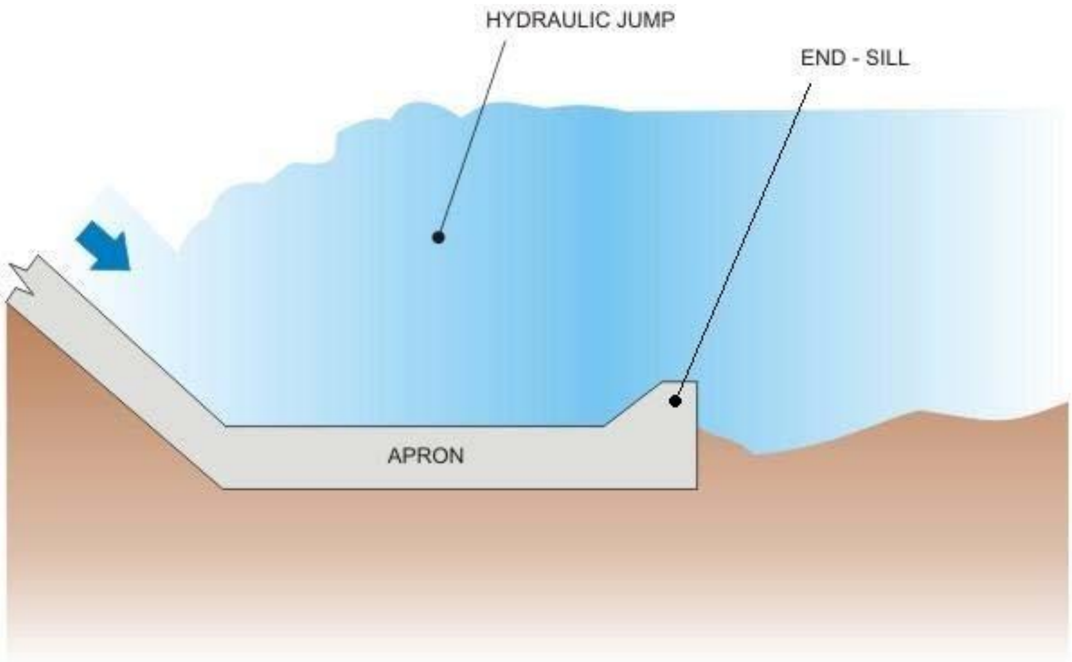


FIGURE 38. Horizontal apron stilling basin with end-sill

2. Sloping apron type (Figure 39)

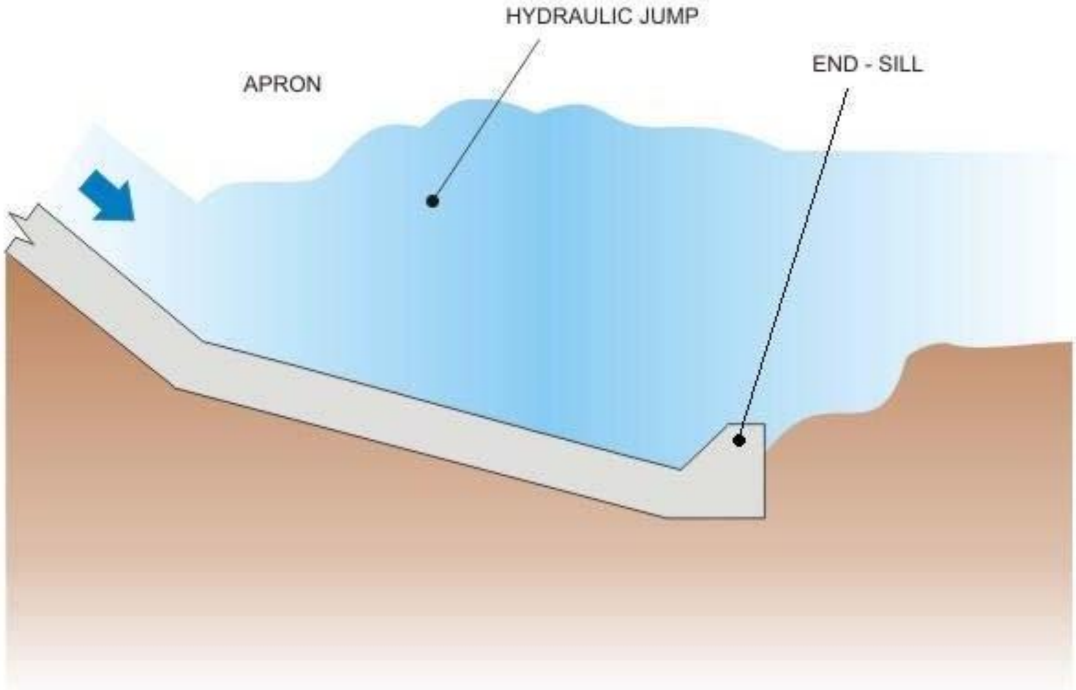


FIGURE 39. Sloping apron stilling basin with end-sill

b) Jet diffusion type stilling basins

1. Jet diffusion stilling basins (Figure 40)

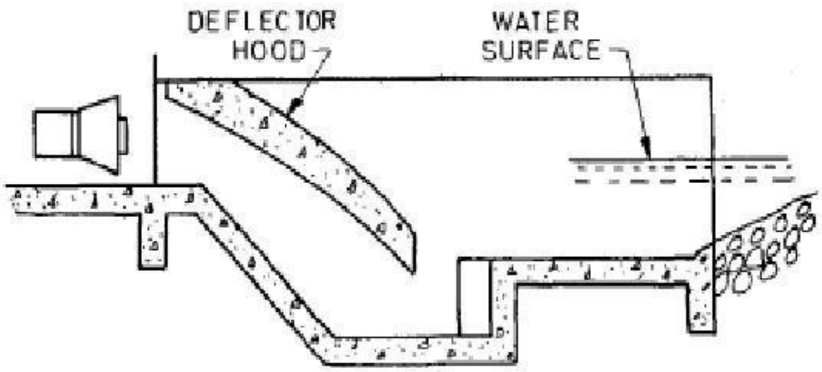


FIGURE 40. Jet diffusion stilling basin

(Image courtesy: IS 10137)

2. Interacting jet dissipators (Figure 41)

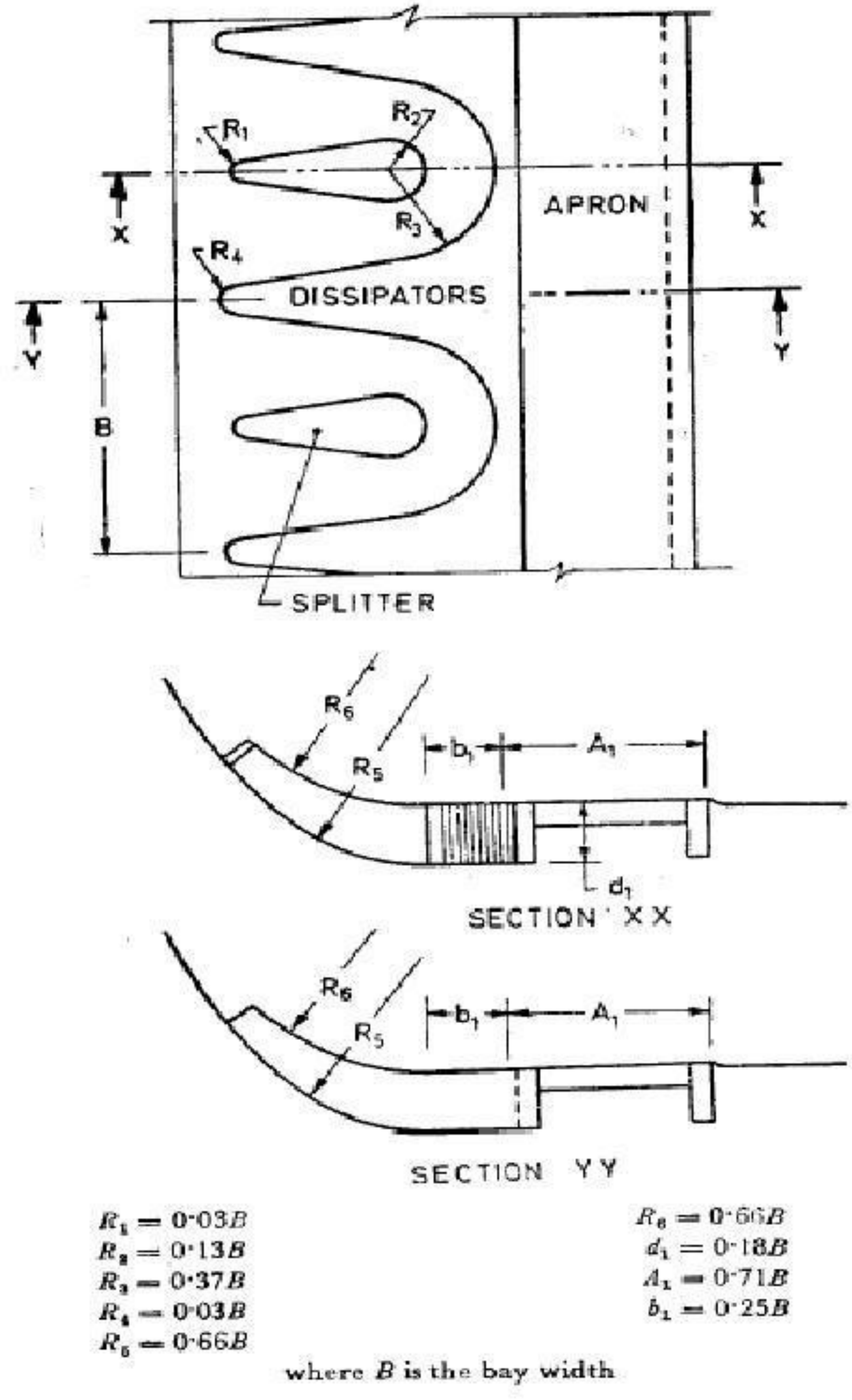


FIGURE 41. Interacting jet dissipators

(Image courtesy: IS 10137)

3. Free jet stilling basins (Figure 42)

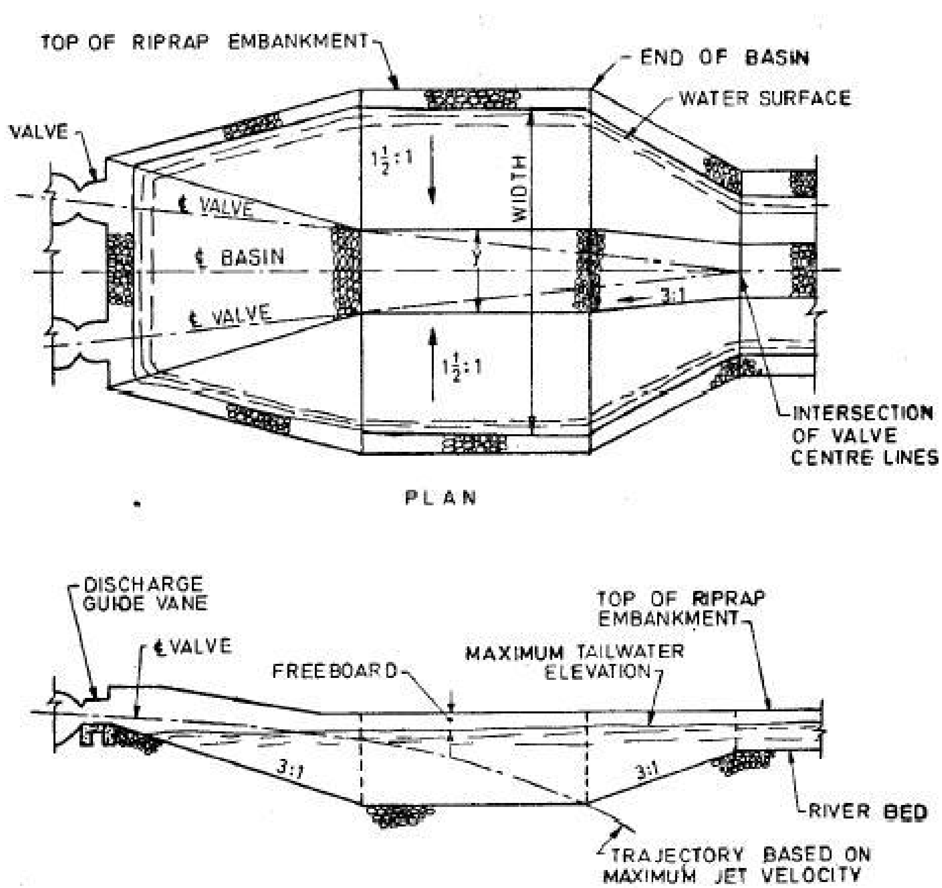


FIGURE 42. Free jet stilling basin

(Image courtesy: IS 10137)

4. Hump stilling basins (Figure 43)

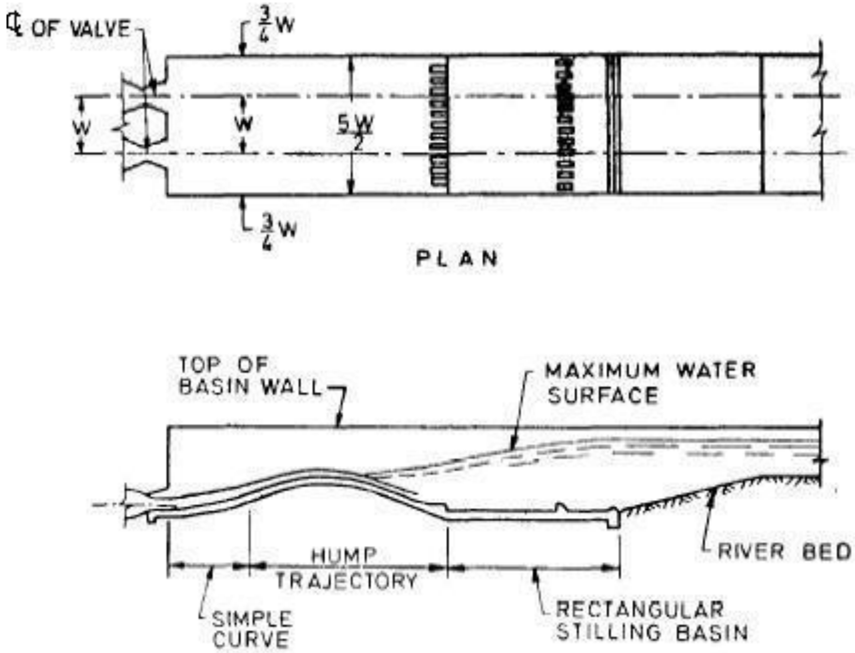


FIGURE 43. Hump stilling basin

(Image courtesy: IS 10137)

5. Impact stilling basins (Figure 44)

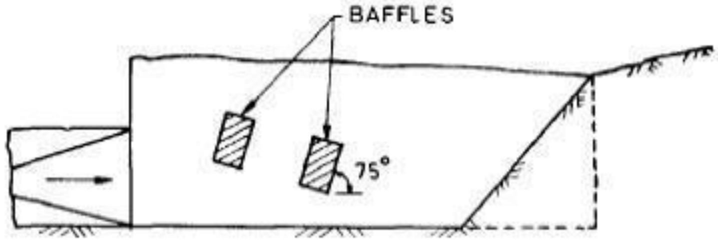


FIG. 15 IMPACT STILLING BASIN WITH INCLINED BAFFLES

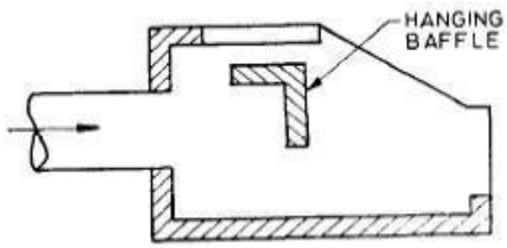


FIGURE 44. Impact stilling basin

(Image courtesy: IS 10137)

Bucket type energy dissipators

This type of energy dissipators includes the following:

1. Solid roller bucket
2. Slotted roller bucket
3. Ski jump (Flip/Trajectory) bucket

The shapes of the different types of bucket-type stilling basins have been given in section 4.8.14. Usually the hydraulic jump type stilling basins and the three types of bucket-type energy dissipators are commonly used in conjunction with spillways of major projects. The detailed designs of these are dealt in subsequent sections.

Since energy dissipators are an integral part of a dam’s spillway section, they have to be viewed in conjuction with the latter. Two typical examples have been shown in Figures 45 and 46, though it must be remembered that any type of energy dissipator may go with any type of spillway, depending on the specific site conditions.

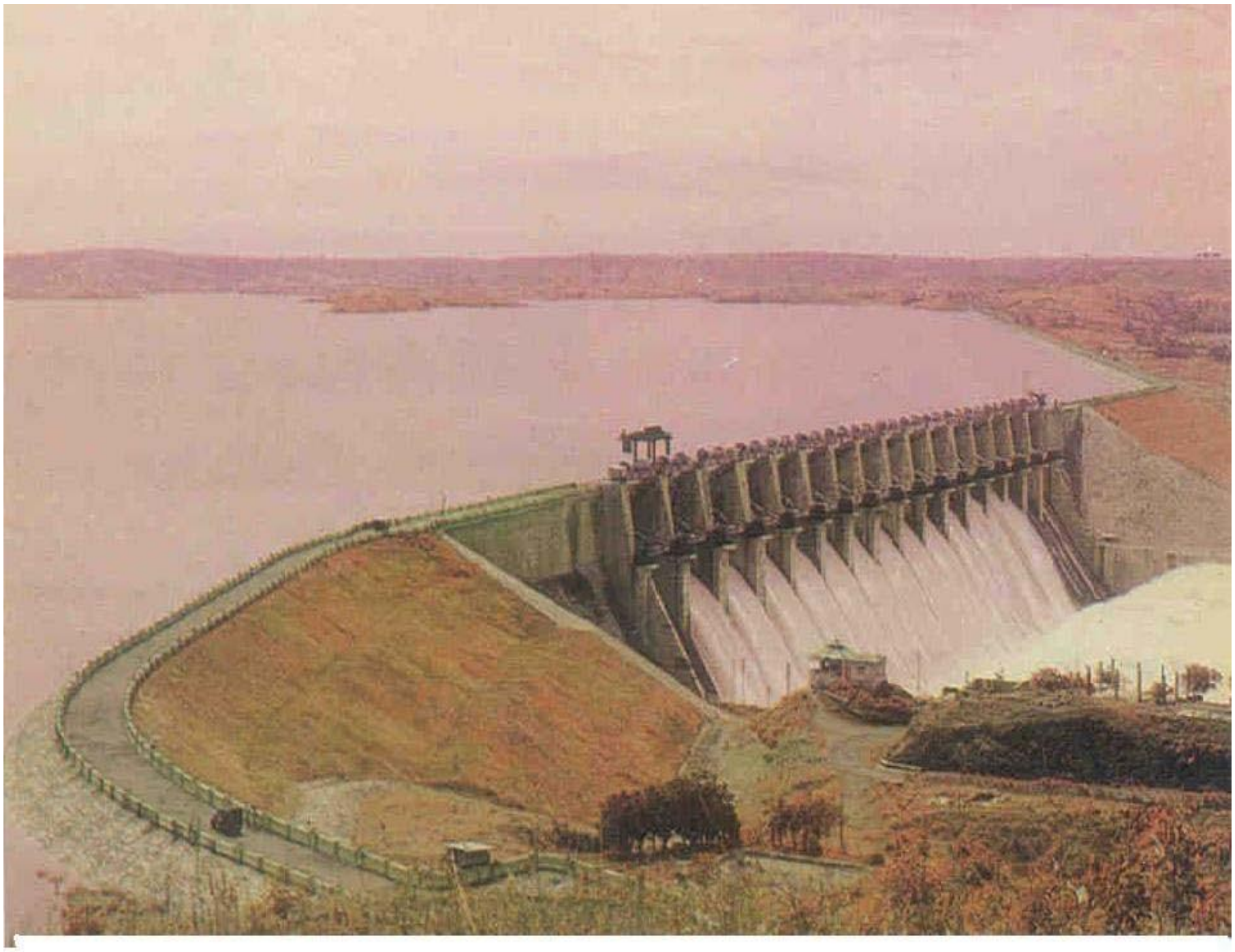


FIGURE 45. Mahi Bajaj Sagar Dam across river Mahi in Rajasthan showing ski-jump bucket energy dissipators in action

(Image courtesy: Web-site of Ministry of Water Resources, Government of India)

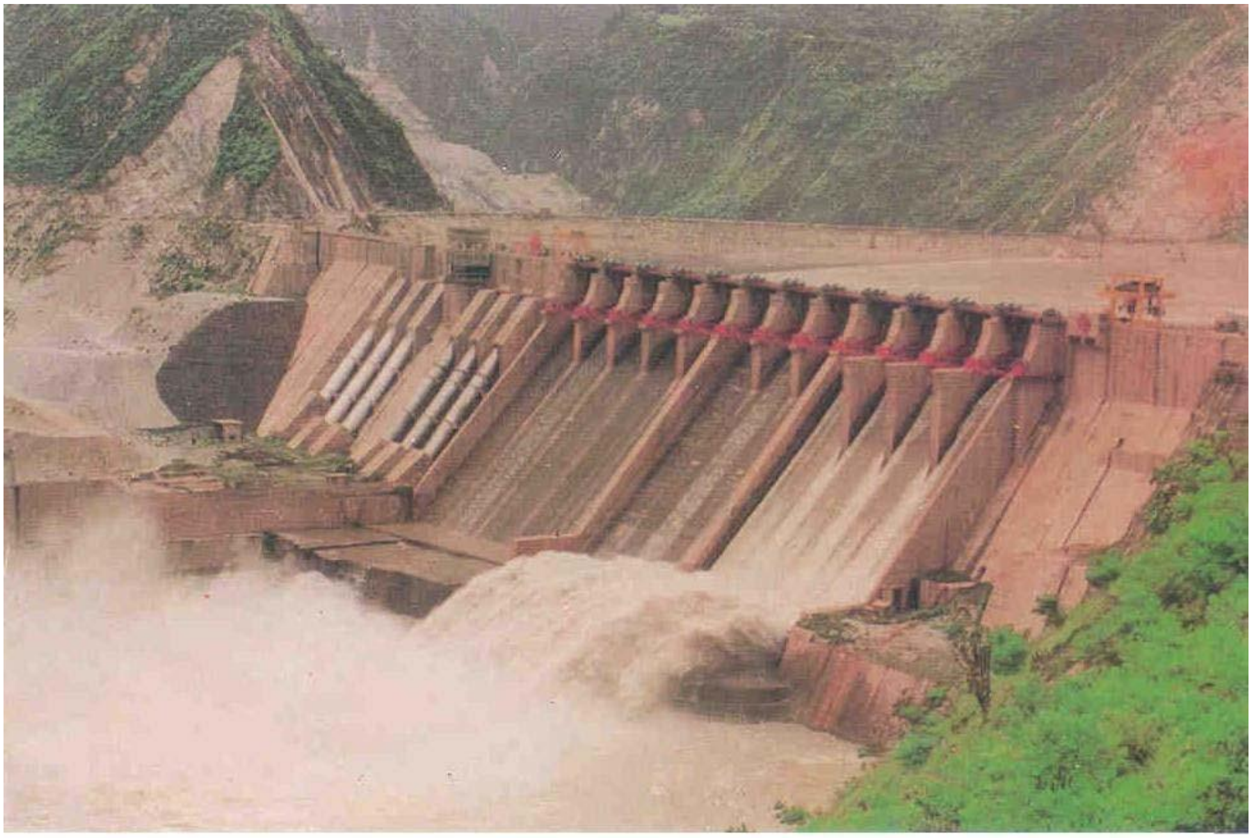
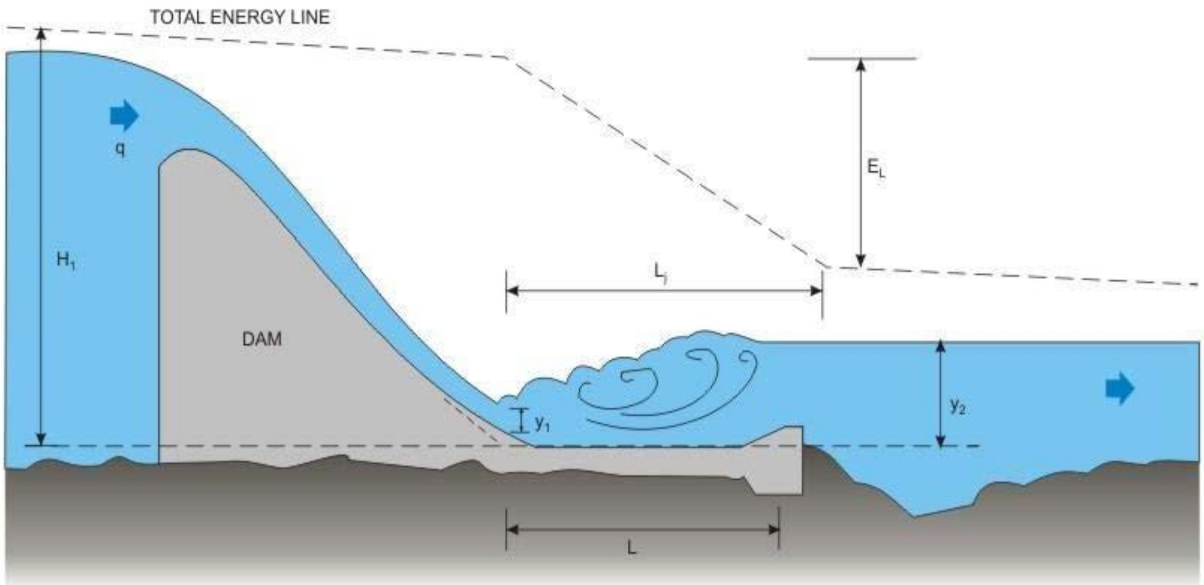


FIGURE 46. Salal project on river Chenab showing energy being dissipated by ski-jump bucket type energy dissipators

(Image courtesy: Web-site of Ministry of Water Resources, Government of India)

Design of Hydraulic Jump Stilling Basin type energy dissipators

A hydraulic jump is the sudden turbulent transition of supercritical flow to subcritical. This phenomena, which involves a loss of energy, is utilized at the bottom of a spillway as an energy dissipator by providing a floor for the hydraulic jump to take place (Figure 47). The amount of energy dissipated in a jump increases with the rise in Froude number of the supercritical flow.



L : LENGTH OF STILLING BASIN APRON

L_j : LENGTH OF HYDRAULIC JUMP

q : DISCHARGE PER UNIT WIDTH

H_1 : TOTAL ENERGY UPSTREAM OF JUMP

y_1 : PRE-JUMP (SUPER CRITICAL FLOW) DEPTH

y_2 : POST-JUMP (SUB CRITICAL FLOW) DEPTH

E_L : ENERGY LOST IN JUMP

FIGURE 47. Definition sketch of hydraulic jump & associated parameters

The two depths, one before (**y_1**) and one after (**y_2**) the jump are related by the following expression:

$$\frac{y_1}{y_2} = \frac{1}{2}\left(-1 + \sqrt{1+8F_1^2}\right) \tag{14}$$

Where F_1 is the incoming Froude number = $\frac{V_1}{\sqrt{gy_1}}$

Alternatively, the expression may be written in terms of the outgoing Froude number F_2 $\left(= \frac{V_2}{\sqrt{gy_2}}\right)$ as

$$\frac{y_2}{y_1} = \frac{1}{2}\left(-1 + \sqrt{1+8F_2^2}\right)$$

(15)

where $\mathbf{V_1}$ and $\mathbf{V_2}$ are the incoming and outgoing velocities and $\boldsymbol{g}$ is the acceleration due to gravity.

The energy lost in the hydraulic jump (E_L) is given as:

$$E_L = \frac{(y_2 - y_1)^3}{4 y_1 y_2} \tag{16}$$

In most cases, it is possible to find out the pre-jump depth ($\mathbf{y_1}$) and velocity ($\mathbf{V_1}$) from the given value of discharge per unit width (**q**) through the spillway. This is done by assuming the total energy is nearly constant right from the spillway entrance up to the beginning of the jump formation, as shown in Figure 47. $\mathbf{V_1}$ may be assumed to beequal to $\sqrt{2gH_1}$ **,** where $\mathbf{H_1}$ is the total energy upstream of the spillway, and neglecting friction losses in the spillway. The appropriate expressions may be solved to find out the post- jump depth ($\mathbf{y_2}$) and velocity ($\mathbf{V_2}$).

The length of the jump ($\mathbf{L_j}$) is an important parameter affecting the size of a stilling basin in which the jump is used. There have been many definitions of the length of the jump, but it is usual to take the length to be the horizontal distance between the toe of the jump upto a section where the water surface becomes quite level after reaching a maximum level. Because the water surface profile is very flat towards the end of the jump, large personal errors are introduced in the determination of the jump length.

Bradley and Peterka (1975) have experimentally found the length of hydraulic jumps and plotted them in terms of the incoming Froude number ($\mathbf{F_1}$), and post-jump depth ($\mathbf{y_2}$) as shown in Figure 48. It is evident that while $\mathbf{L_j/y_2}$ varies most for small values of $\mathbf{F}_1$, at higher values, say above 5 or so, $\mathbf{L_j /y_2}$ is practically constant at a value of about6.1.

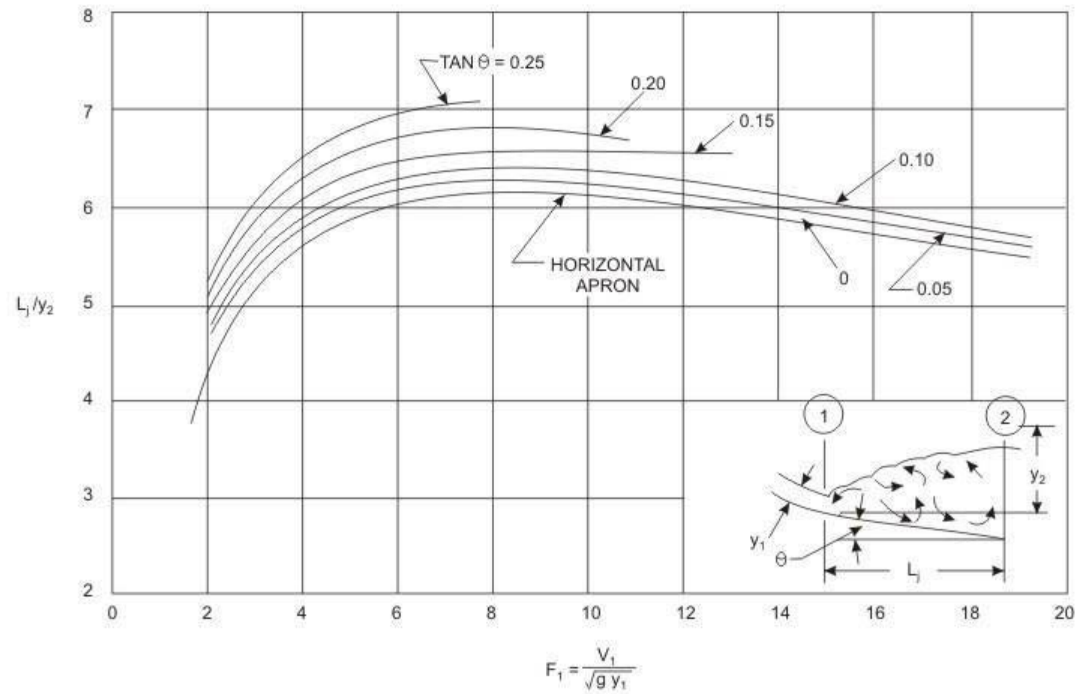


FIGURE 48. Length of hydraulic jump on a horizontal or inclined floor

The depth of water in the actual river downstream of the stilling basin (**y_2^***) is determined from the river flow observations that have been plotted as a stage-discharge curve (Figure 49).

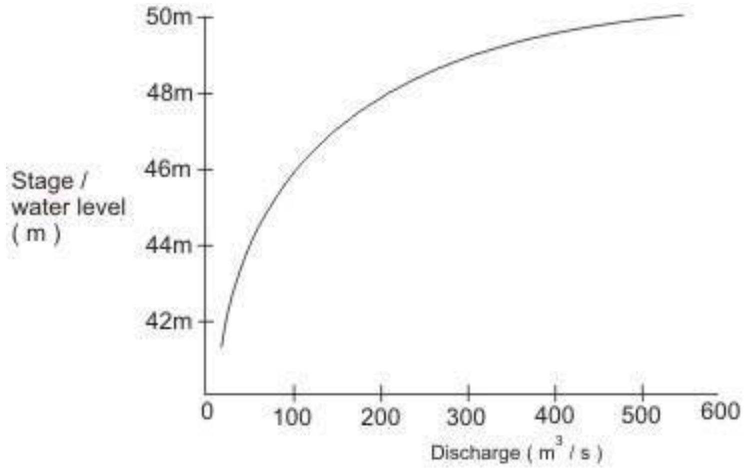


FIGURE 49. A typical stage-discharge curve for a river

Subtracting the stilling basing apron level from the stage or water level corresponding to the total discharge passing through the spillway gives the tail-water depth (**y_2^***). Since the stage-discharge curve gives indications about the tail-water of the spillway, it is called the Tail-Water Rating Curve (TRC), usually expressed as the water depth (**y_2^***) versus unit discharge (**q**), as shown in Figure 50(a).

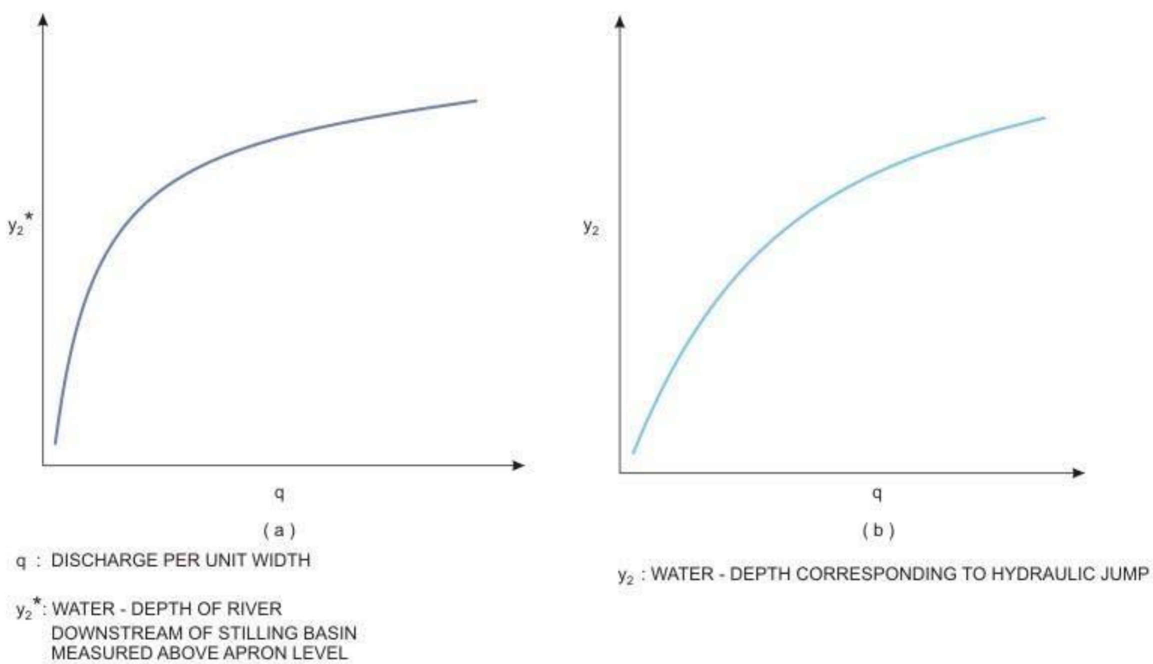


FIGURE 50. Water Level curves
(a) : Tail - water Rating Curve (TRC)
(b) : Jump Rating Curve (JRC)

At the same time, using the formula relating unit discharge (**q**) with the post-jump depth (**y_2**), a similar graph may be obtained, as shown in Figure 50(b). Since this graph gives indication about the variation of the post-jump depth, it is called the Jump Rating Curve (JRC).

In general, the JRC and TRC would rarely coincide, if plotted on the same graph, as shown in Figure 51.

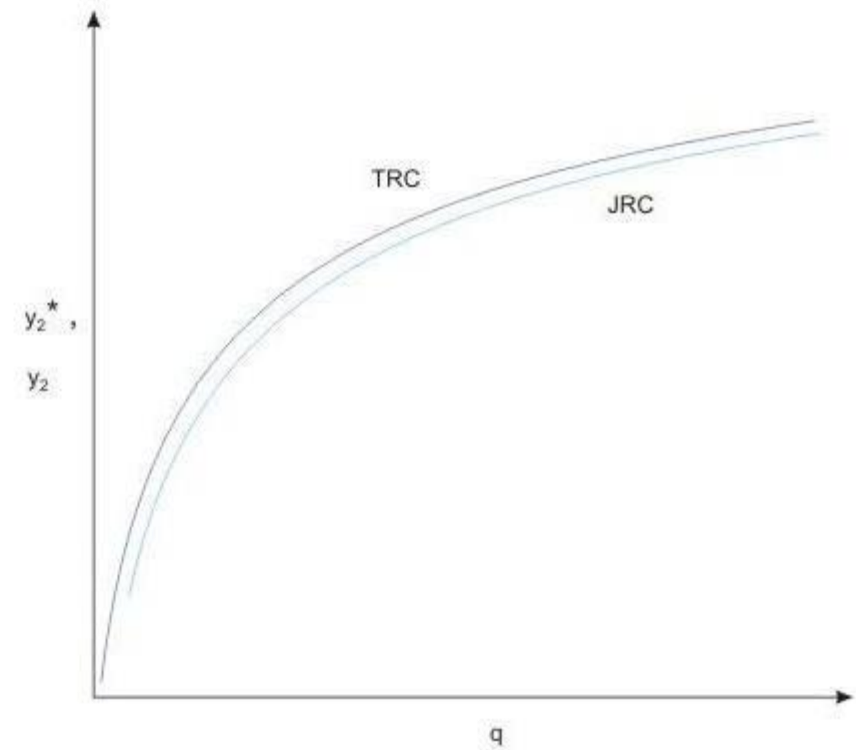


FIGURE 51. TRC & JRC coinciding

At times, the TRC may lie completely below the JRC (Figure 52), for all discharges, in which case the jump will be located away from the toe of the spillway resulting in possible erosion of the riverbed.

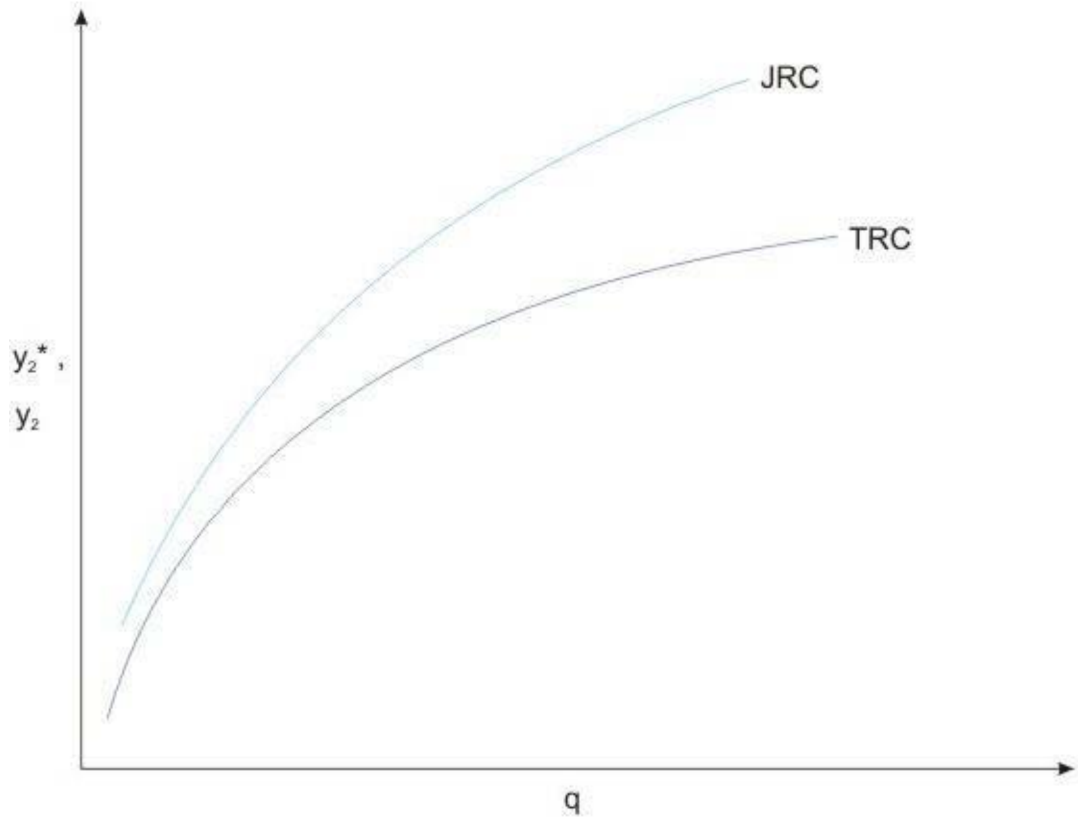


FIGURE 52. TRC below JRC for all discharges

If the TRC is completely above the jump would be located so close to the spillway to make it submerged which may not dissipate the energy completely. (Figure 53)

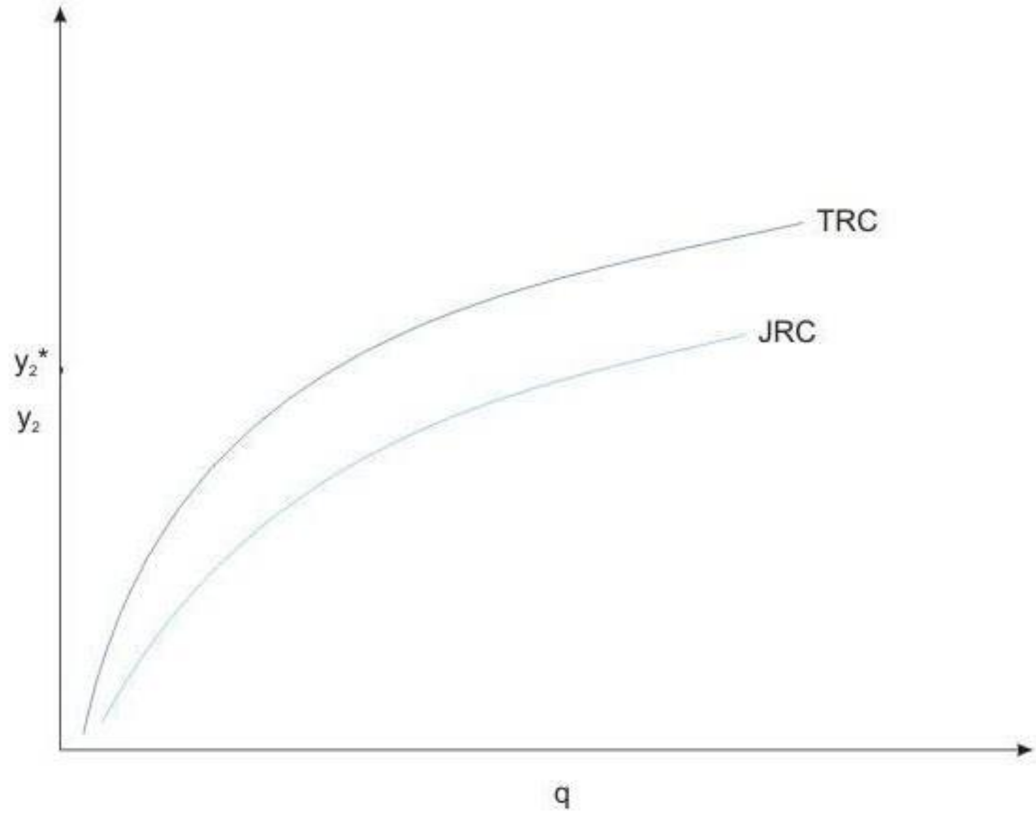


FIGURE 53. TRC above JRC for all discharges

It may also be possible in actual situations that the TRC may be below the JRC for some discharges above for the rest, as shown in Figs. 54 and 55.

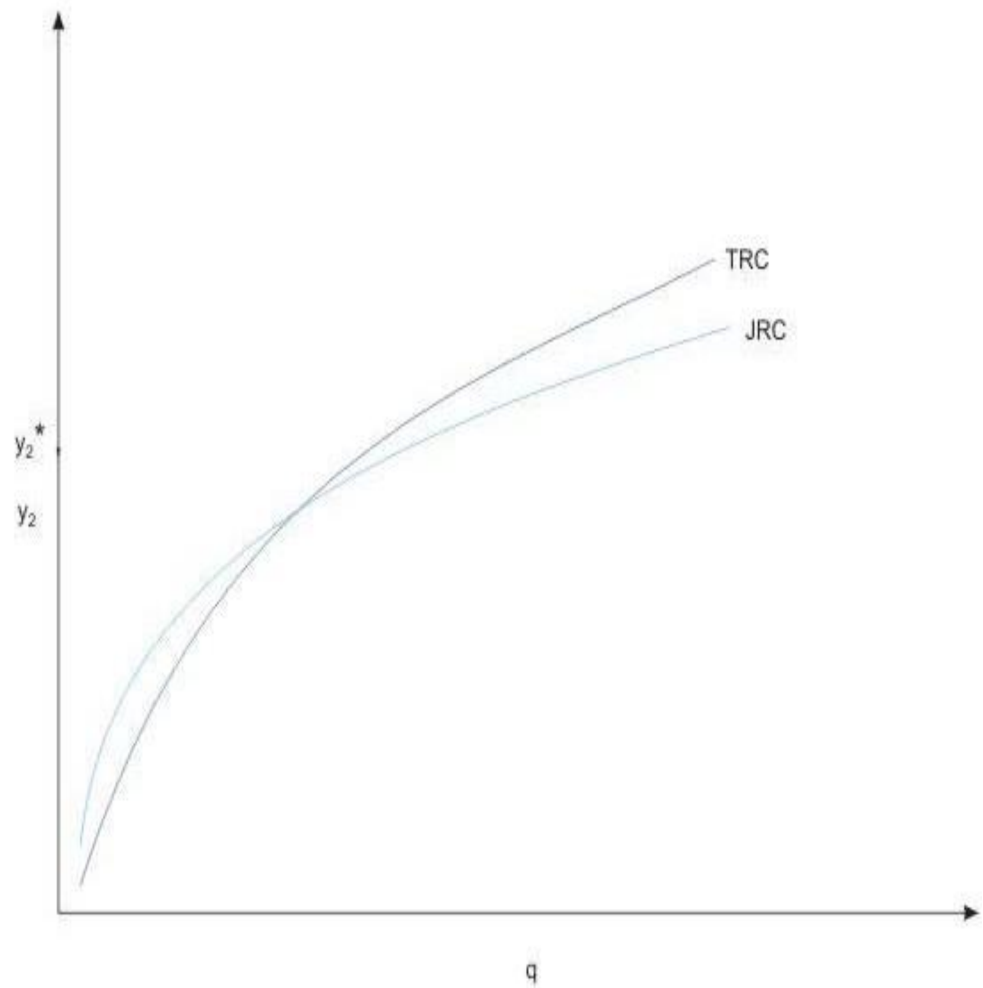


FIGURE 54. TRC below JRC for low discharges and above for high discharges

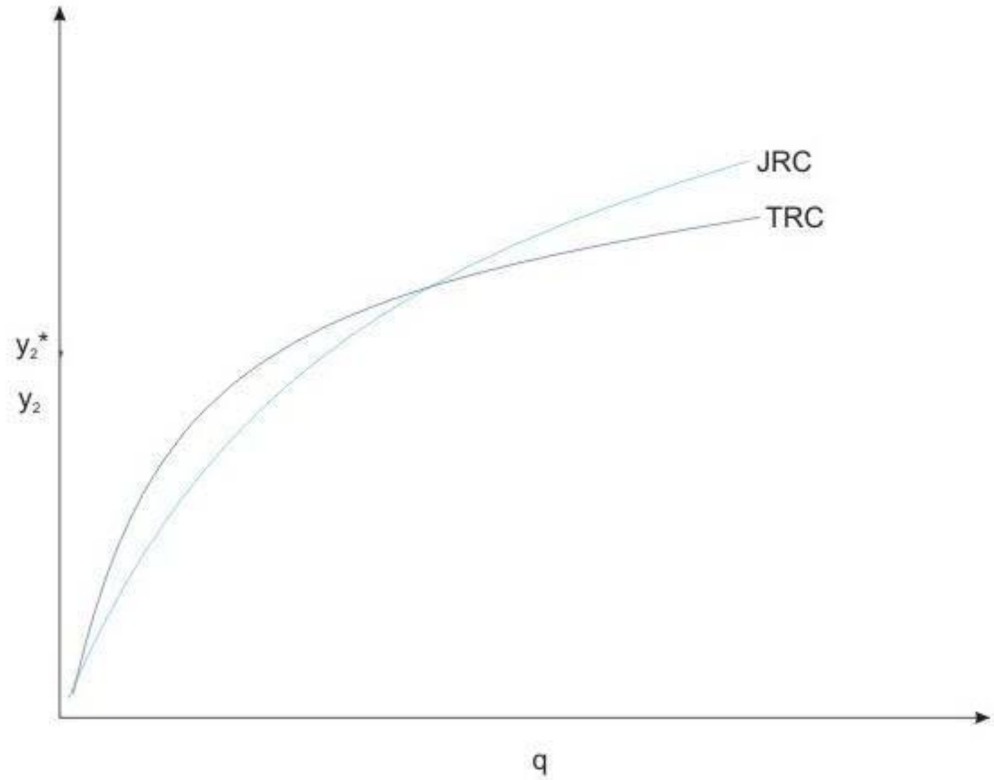


FIGURE 55. TRC above JRC for low discharges and below for high discharges

In these cases two, favourable location of jump may not be possible. In view of the above situations, the following recommendations have been made for satisfactory performance of the hydraulic jumps.

Case1 (Figure 51)
This is the ideal case in which the horizontal apron provided on the riverbed downstream from the toe of the spillway would suffice. The length of the apron should be equal to the length of the jump corresponding to the maximum discharge over the spillway.

Case2 (Figure 52)
It is apparent that the tail water depth as provided by the natural river is not sufficiently for the jump to form. This may be over come by providing a stilling basin apron that is depressed below the average riverbed level (Figure56) or by providing a sill or baffle of sufficient height at the end of the spillway (Figure 57)

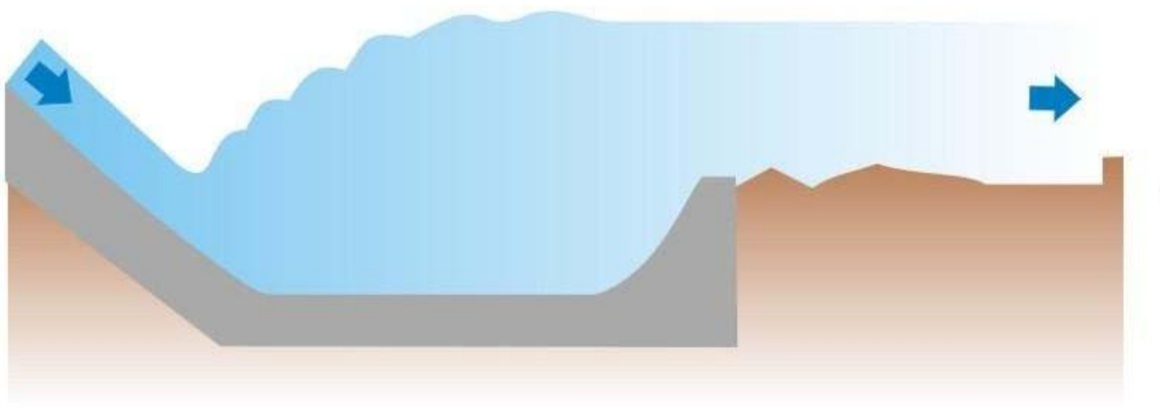


FIGURE 56. Depressed floor of stilling basin apron

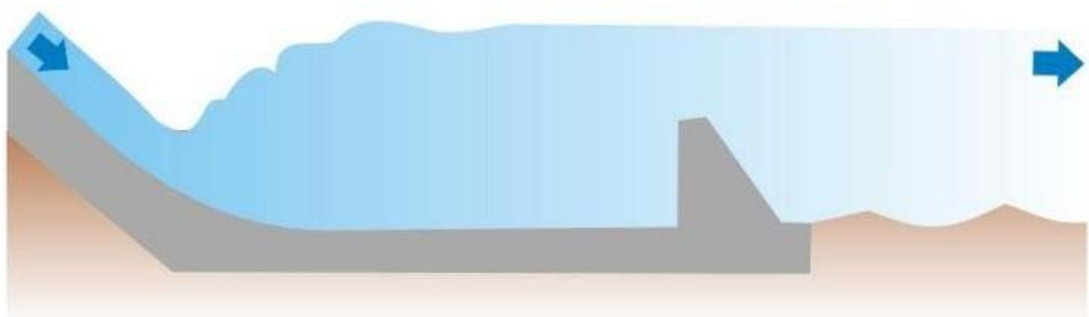


FIGURE 57. High end - sill or baffle at toe of stilling basin

Case 3 (Figure 53)
Since this situation results in submergence results in submergence of the jump, it is necessary to raise the floor in order to form a clear jump. In practice, it is done by providing an inclined apron of the stilling basin (Figure 58).

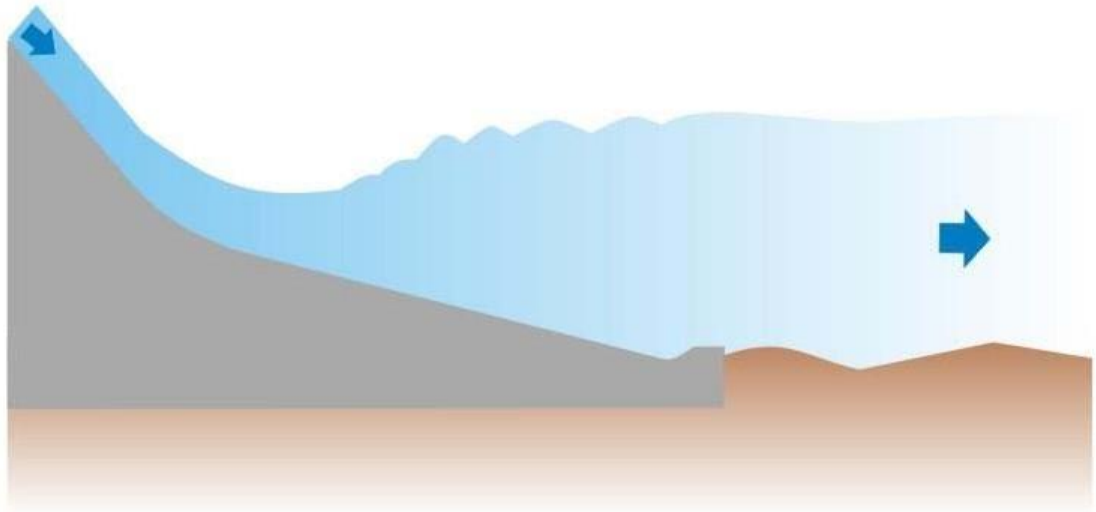


FIGURE 58. Inclined stilling basin

Case 4 (Figure 54)
This situation may be taken care of by providing an inclined floor in the upper portion of the stilling basin and providing either a depressed floor in the lower portion of the basin or provide a baffle at the end of the basin.

Case 5 (Figure 55)
In this case a sloping apron may be provided which lies partly above and partly below the riverbed. So that the jump will form on the higher slope at low discharges and on the lower slope at high discharges.

The type of Stilling Basins that may be provided under different situations is recommended by the Bureau of Indian Standards code IS: 4997-1968 “Criteria for design of hydraulic jump type stilling basins with horizontal and sloping aprons”. In all, these are four types of basin shapes recommended. Types I and II are meant for basins with horizontal floors and types III and IV for basins with inclined floors.

Design of bucket-type energy dissipators

Hydraulic behaviour of bucket type energy dissipator depends on dissipation of energy through:

a. Interaction of two rollers formed, one in the bucket, rolling anti-clockwise (if the flow is from the left to the right) and the other downstream of the bucket, rolling clockwise; or
b. Interaction of the jet of water, shooting out from the bucket lip, with the surrounding air and its impact on the channel bed downstream.

Bucket type energy dissipators can be either:

a) Roller bucket type energy dissipator; or
b) Trajectory bucket type energy dissipator.

The following two types of roller buckets are adopted on the basis of tailwater conditions and importance of the structure:

a) Solid roller bucket, and
b) Slotted roller bucket.

These are shown in Figure 59.

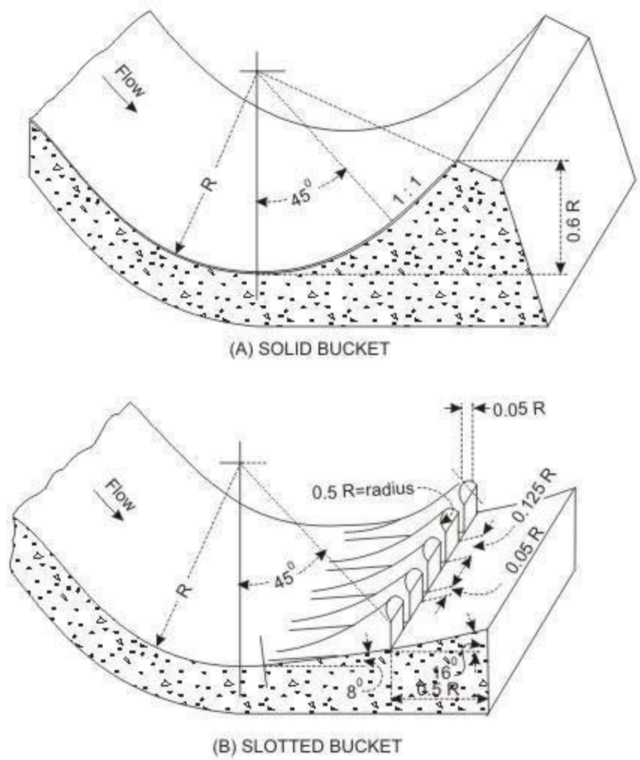


FIGURE 59. Roller buckets ; (A) Solid ; (b) Slotted

Roller bucket type energy dissipator is preferred when:

a) Tailwater depth is high (greater than 1.1 times sequent depth preferably 1.2 - times sequent depth), and
b) River bed rock is sound.

Trajectory bucket type energy dissipator is generally used when:

a) Tailwater depth is much lower than the sequent depth of hydraulic jump, thus preventing formation of the jump;
b) By locating at higher level it may be used in case of higher tailwater depths also, if economy permits; and

c) Bed of the river channel downstream is composed of sound rock.

This is shown in Figure 60.

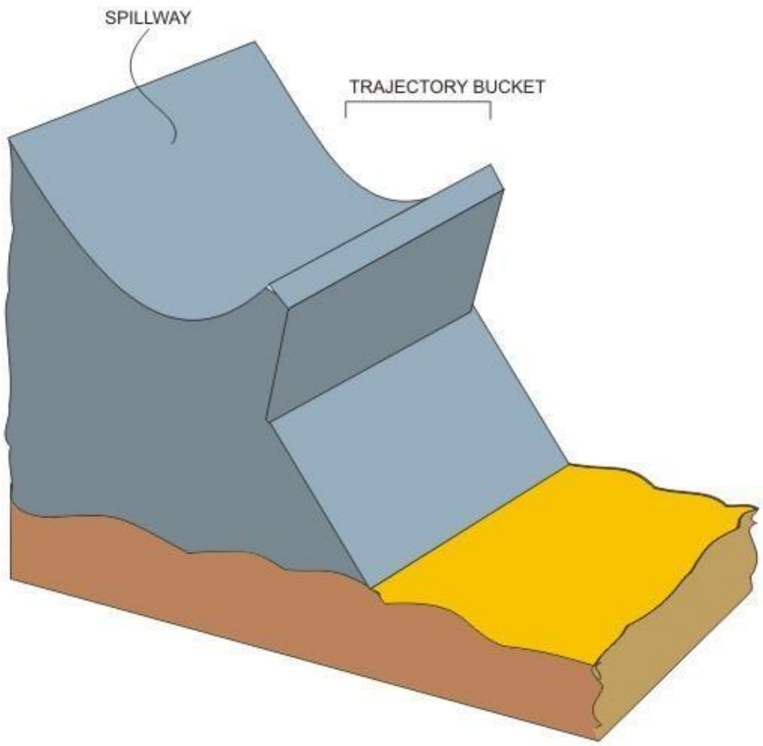


FIGURE 60. Trajectory bucket type energy dissipator

Action of the various types of bucket-type energy dissipators is given below:

Hydraulic Design of Solid Roller Bucket

An upturn solid bucket is used when the tailwater depth is much in excess of sequent depth and in which dissipation of considerable portion of energy occurs as a result of formation of two complementary elliptical rollers, one in bucket proper, called the surface roller, which is anticlockwise (if the flow is to the right) and the other downstream of the bucket, called the ground roller, which is clockwise.

In the case of solid roller bucket the ground roller is more pronounced and picks up material from downstream bend and carried it towards the bucket where it is partly deposited and partly carried away downstream by the residual jet from the lip. The deposition in roller bucket is more likely when the spillway spans are not operated equally, setting up horizontal eddies downstream of the bucket. The picked up material which is drawn into the bucket can cause abrasive damage to the bucket by churning action.

For effective energy dissipation in a solid roller bucket, both the surface or dissipating roller and the ground or stabilizing roller, should be well formed. Otherwise, hydraulic phenomenon of sweep out or heavy submergence occurs depending upon which of the rollers is inhibited.

Design Criteria - The principal features of hydraulic design of solid roller bucket consists of determining:

a) The bucket invert elevation,
b) The radius of the bucket, and
c) The slope of the bucket lip or the bucket lip angle.

The various parameters are shown in Figure 61 (a).

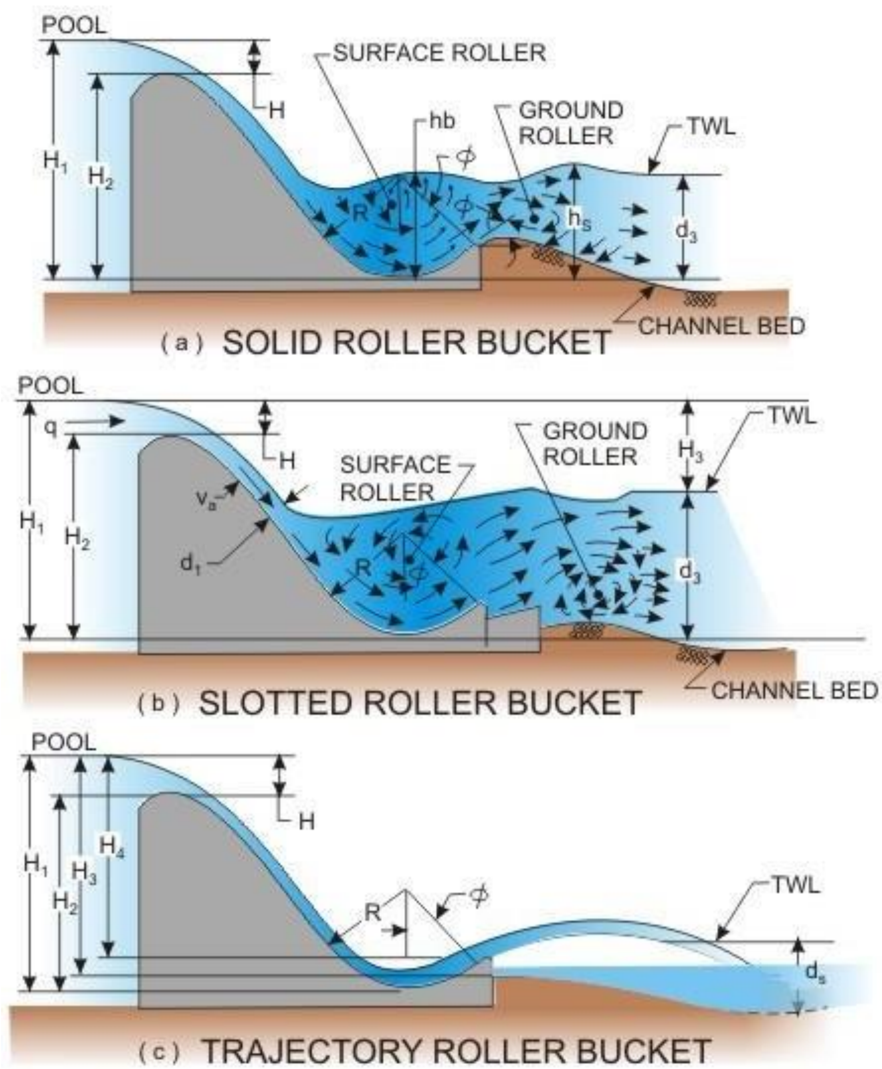


FIGURE 61. Sketches for bucket type energy dissipators

An example of the use of a solid roller bucket is the energy dissipator of the Maithan Dam Spillway (Figure 62)

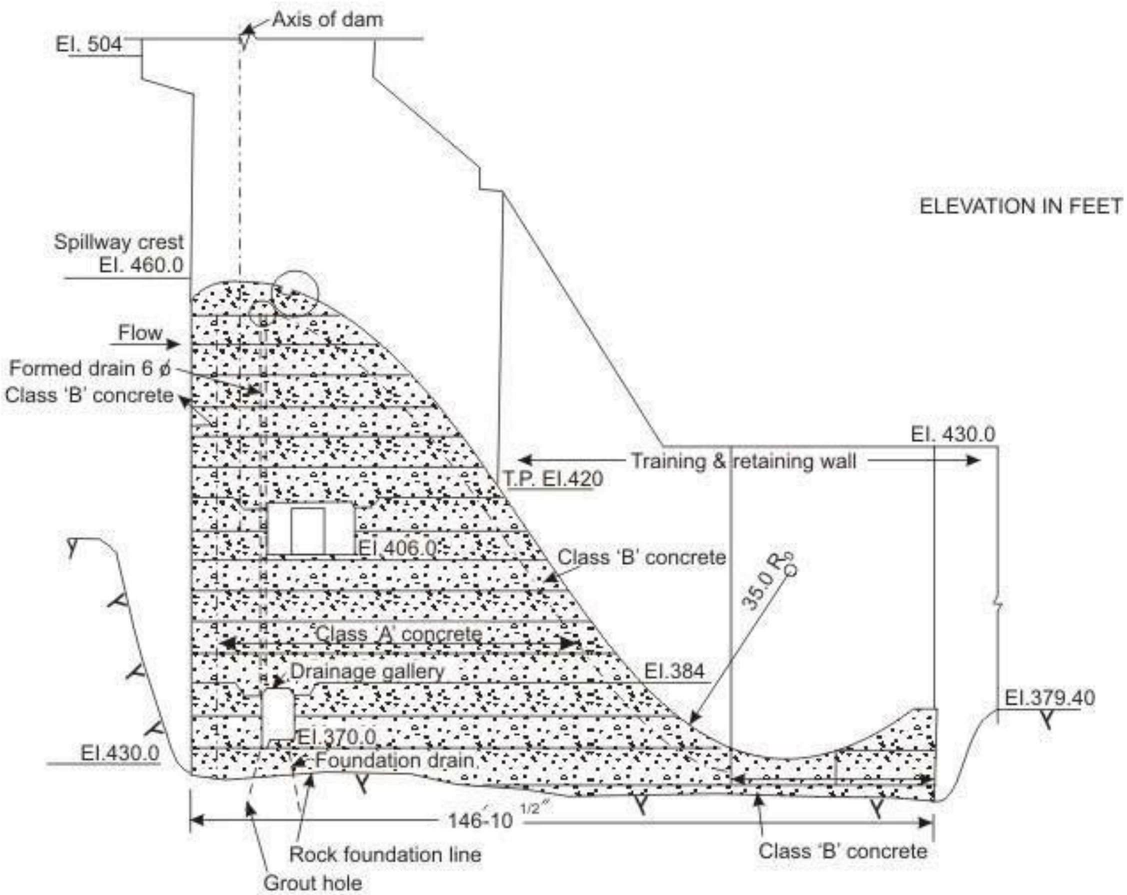


FIGURE 62. Maithon Dam spillway

Drawal of Bed Materials - A major problem with the solid roller bucket would be the damage due to churning action, caused to the bucket because of the downstream bed material brought into the bucket by the pronounced ground roller. Even in a slotted roller bucket downstream material might get drawn due to unequal operation of gates. The channel bed immediately downstream of the bucket shall be set at 1 to 1.5 m below the lip level to minimize the possibility of this condition. Where the invert of the bucket is required to be set below the channel general bed level the channel should be dressed down in one level to about 1 to 1.5 m below the lip level in about 15 m length downstream and then a recovery slope of about 3 (horizontal) to 1 (vertical) should be given to meet the general bed level as shown in Figure 62. Careful model studies should be done to check this tendency. If possible, even provision of solid apron or cement concrete blocks may be considered to avoid trapping of river bed material in the bucket as it may cause heavy erosion on the spillway face, bucket and side training wall.

In the case of slotted roller bucket a part of the flow passes through the slots, spreads laterally and is lifted away from the channel bottom by a short apron at the downstream end of the bucket. Thus the flow is dispersed and distributed over a greater area resulting in a less violent ground roller. The height of boil is also reduced in case of slotted roller bucket. The slotted bucket -provides a self-cleaning action to reduce abrasion in the bucket.

In general the slotted roller bucket is a improvement over the solid roller bucket for the range of tailwater depths under which it can operate without sweepout or diving. However, it is necessary that specific model experiments should be conducted to verify pressure on the teeth so as to avoid cavitation conditions. In case of hydraulic structures in boulder stages slotted roller buckets need not be provided. Heavy boulders rolling down the spillway face can cause heavy damage to the dents thereby making them ineffective and on the contrary, increasing the chances of damage by impact, cavitation and erosion.

Hydraulic Design of Slotted Roller Bucket

An upturned bucket with teeth in it used when the tailwater depth is much in excess of sequent depth and in which the dissipation of energy occurs by lateral spreading of jet passing through bucket slots in addition to the formation of two complementary rollers as in the solid bucket.

In the slotted roller bucket, a part of the flow passes through the slots, spreads laterally and is lifted away from the channel bottom by a short apron at the downstream end of the bucket. Thus the flow is dispersed and distributed over a greater area providing less violent flow concentrations compared to those in a solid roller bucket. The velocity distribution just downstream of the bucket is more akin to that in a natural stream, that is, higher velocities at the surface and lower velocities at the bottom. While designing a slotted roller bucket, for high head spillway exceeding the total head of 50 m or so, specific care should be taken especially for design of the teeth, to ensure that the teeth will perform cavitation free. Specific model tests should therefore be conducted to verify pressures on the teeth and the bucket invert should accordingly be fixed at such an elevation as to restrict the subatmospheric pressures to the permissible magnitude.

Design Criteria - The principal features of hydraulic design of the slotted roller bucket consists of determining in sequence:

a) bucket radius;
b) bucket invert elevation;
c) bucket lip angle; and
d) bucket and tooth dimensions, teeth spacing and dimensions and profile of short apron.

The various parameters are shown in Figure 61(b)

An example of the use of a slotted roller bucket is the energy dissipator provided in the Indira Sagar Dam Spillway (Figure 63).

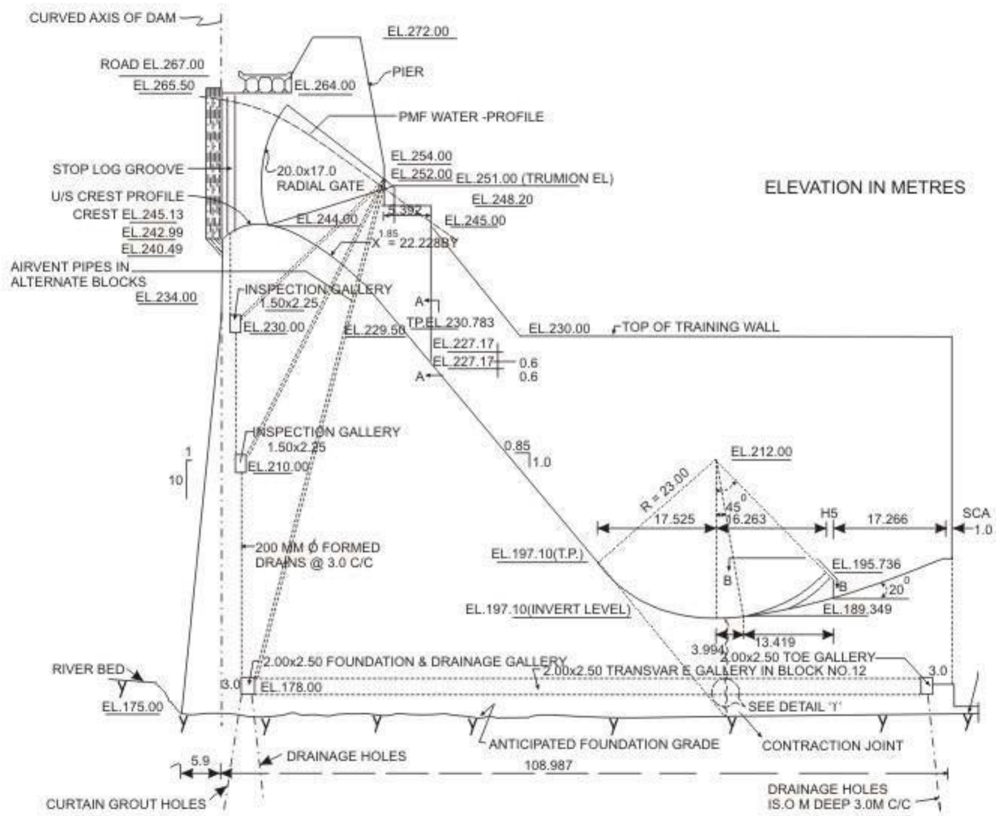


FIGURE 63. Indira Sagar Dam Spillway

Hydraulic Design of Trajectory Bucket Type Energy Dissipator

An upturn solid bucket used when the tailwater depth is insufficient for the formation of the hydraulic jump, the bed of the river channel downstream comprises sound rock and is capable of withstanding, without excessive scour, the impact of the high velocity jet. The flow coming down the spillway is thrown away from toe of the dam to a considerable distance downstream as a free discharging upturned jet which falls into the channel directly, thereby avoiding excessive scour immediately downstream of the spillway. There is hardly any energy dissipation within the bucket itself. The device is used mainly to increase the distance from the structure to the place where high velocity etc. hits the channel bed, thus avoiding the danger of excessive scour immediately downstream of the spillway. Due to the throw of the jet in the shape of a trajectory, energy dissipation takes place by

a) internal friction within the jet,
b) the interaction between the jet and surrounding air,
c) the diffusion of the jet in the tailwater, and
d) the impact on the channel bed.

When the tailwater depth is insufficient for the formation of the hydraulic jump and the bed of the channel downstream comprises sound rock which is capable of withstanding the impact of the high velocity jet, the provision of a trajectory bucket is considered more suitable as provision of conventional hydraulic jump type apron or a roller bucket involves considerable excavation in hard strata forming the bed. It is also necessary to have sufficient straight reach in the downstream of a skijump bucket. The flow coming down the spillway is thrown away in air from the toe of the structure to a considerable distance as a free discharging upturned jet which falls on the channel bed *d/s.* The hard bed can tolerate the spray from the jet and erosion by the plunging jet would not be a significant problem for the safety of the structure. Thus, although there is very little energy dissipation within the bucket itself, possible channel bed erosion close to the downstream toe of the dam is minimized. In the trajectory bucket, only part of the energy is dissipated through interaction of the jet with the surrounding air. The remaining energy is imparted to the channel bed below. The channel bed should consist of sound, hard strata and should be free from laminations, joints and weak pockets to withstand the impact of jet. The design of the trajectory bucket presupposes the formation of large craters or scour holes at the zone of impact of the jet during the initial years of operation and, therefore, the design shall be restricted to sites where generally sound rock is available in the river bed. Special care shall be taken to concrete weak pockets in the bed located in a length of

Design Criteria - The principal features of hydraulic design of tra jectory bucket consist of determining:

a) Bucket shape,
b) Bucket invert elevation, radius or principal geometrical parameters of the bucket, lip elevation and exit angle, trajectory length, and
c) Estimation of scour downstream of the spillway.

The various parameters are shown in Figure 61(c)

An example of the use of a trajectory bucket is the one provided in the Srisailam Dam Spillway (Figure 64).

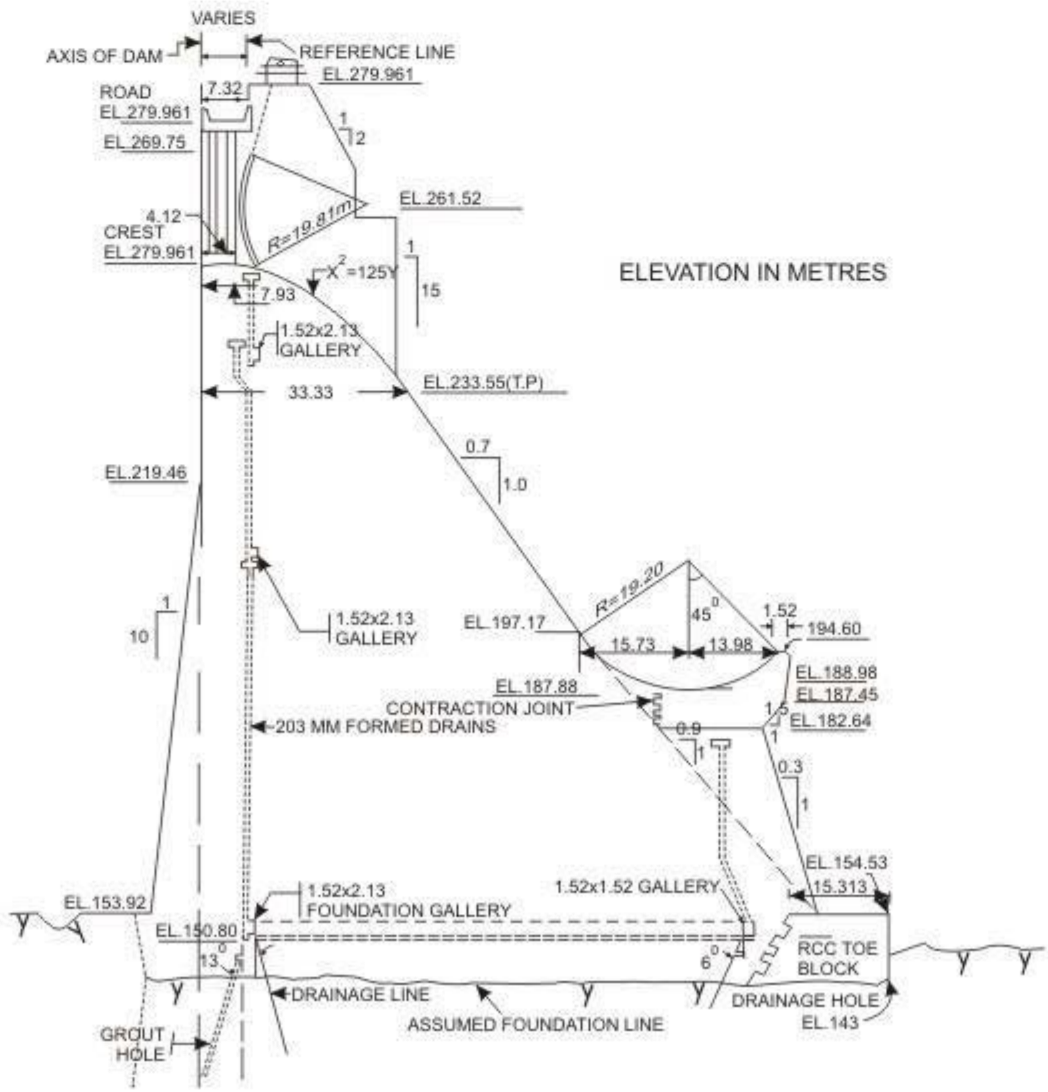


FIGURE 64. Srisailam Dam Spillway

Further details about the design of bucket type energy dissipators may be had from the Bureau of Indian Standards Code IS: 7365-1985 “Criteria for hydraulic design of bucket type energy dissipators”

Protection of downstream of spillways from scour

It may be noted that inspite of the provision of the best suited energy dissipator for a specific spillway under the prevailing site conditions, there may be still some energy is expected to be maximum for the trajectory type spillway, followed by the solid and slotted roller buckets and finally the hydraulic jump type stilling basins. In order to protect the downstream riverbed from these undesirable scour, the following types of protection works have been recommended by the Bureau of Indian Standards code IS:

13195-1991 “Preliminary design, operation and maintenance of protection works downstream of spillways-guidelines”.

1. *Training Walls at the Flanks of the Spillways-* Training walls extended beyond the end-sill of the stilling basins or buckets generally serve to guide the flow into the river channel, protect the wrap-rounds of the adjacent earth dams,
river banks or power house bays and tail race channels. To this extent, the training walls are -considered to be downstream protection works.

2 *Protective Aprons Downstream of Bucket Lips or End-sills of Stilling Basins- Protective* aprons of concrete laid on fresh rock or acceptable strata immediately downstream of bucket lips or end-sill of stilling basin, protect the energy dissipator against undermining due to excessive scour during or after construction of the spillways. A suitable concrete key is normally provided, at the downstream end of the apron. Where the normal river bed level is higher than the end-sill and a recovery slope is , provided, it sometimes becomes necessary to lay a concrete apron on such a recovery slope also for protection.

3. *Concrete Blocks or Concrete Filling on River Bed Downstream of Energy Dissipator* - Concrete blocks or concrete fillings are sometimes provided on the river bed downstream of energy dissipators to safeguard against excessive scour and prevent further scour.

4. *Protective Pitchings on Natural or Artificial Banks Downstream of Spillways-* Protective pitchings of stone rip rap, masonry or concrete blocks are provided on natural river banks or artificially constructed embankments of diversion channels, power house tail race channels or guide banks, for protecting them against high velocity flows or waves.

Figure 65 shows the various types of protection works that may typically be used downstream of a spillway.

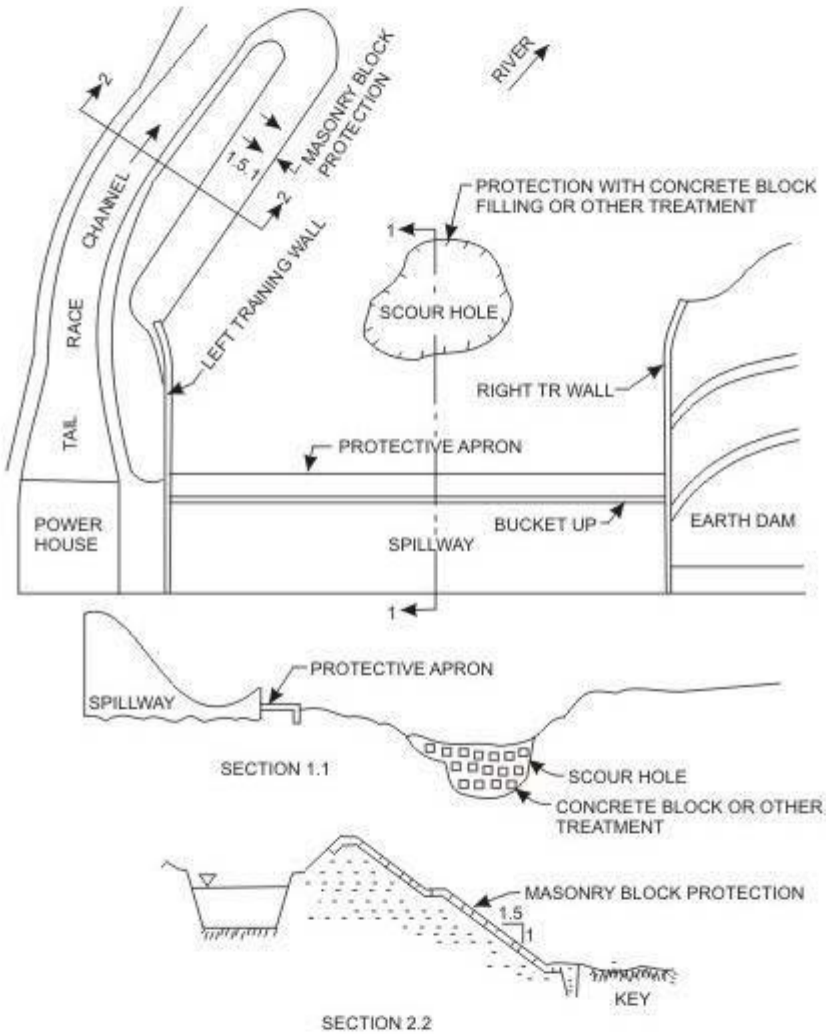


FIGURE 65. Different types of protection works downstream of a typical spillway project (Courtesy: IS 13195)

The importance of providing protection below a spillway, especially of the trajectory type may be noted from the incidence of deep scour on the downstream of the Srisailam dam spillway.

Case Study

Srisailam dam spillway (Figure 64) across river Krishna was constructed during 1977-83. It is a 137 m high concrete dam, with 12 spans of 18'3 m **x** 16'8 m. The river bed is composed of quartzites and shales. In the immediate downstream vicinity of the spillway, there were horizontal shear zones 0'2 m to 0'9 m thick, where the quartzites are crushed and sheared. During the monsoons of 1977 to 1980, the construction stage flood passed over the partially constructed spillway bays, spilling over 7 bays which were at different levels having a maximum difference of level of 23 m. The difference in level between the lip of the ski-jump bucket and downstream rock was about 44 m.

Shorter throw of the water spilling over the bucket lip, as a cascading flow caused deep scour in the immediate vicinity of the bucket lip. During subsequent floods, the scour holes were concreted and leveled as protective aprons in some part of the spillway. Such aprons were however, subjected to repeated damage and undermining. By April 1985, depth of scour below blocks 11 to 13 reached from 9 m to 22 m below the protective apron. Cavities of undermining below the apron were also present at a depth of 6 to 9 m.

The protection work consisted of providing an underwater massive concrete block touching the apron and filling the eroded cavities below the apron. The water level at downstream toe varied from the top of existing apron to about 1.5 m below it.

The scheme involved forming 4 cells with steel cylinder walls and filling concrete in each cell followed by concrete capping. Heavy concrete blocks (approximate 1 metre cube) were placed downstream of the cylinder watls to further protect the rock from the water jump damage.

Since the construction of the above protection works, the spillway was completed to final levels and crest gates have also been installed. Hydraulic model studies were conducted to evolve an operation of the spillway in such a way that the throw of the trajectory fall further away from the toe of the dam. This together with the protective measures already implemented is expected to prevent further erosion at the toe of the dam.

Module-3 Diversion head works

DIVERSION HEADWORKS:

Selection of site and layout, parts of diversion head-works, types of weirs and barrages, design of weirs on permeable foundations, control of silt entry into canal, silt excluders and different types of silt ejectors.

Definition

A structure constructed at the junction of the source (river, dam, canal) and the off taking canal.

Types of Headworks

The different types of headworks are as follows:

Diversion Headworks

- diverts the required supply from the source channel to the off taking channel
- water level in the source channel raised to the reqd. level
- reduces the need of excavation in the head reach
- command area is served better by flow irrigation
- should be capable of regulating the supplies into the off taking channel; all supplies when demand is keen & supplies are less
- control sediment entry

Storage Headworks

- fulfill requirements of the diversion headworks
- in addition, store excess water when available and release when demand exceeds supplies

Temporary Headworks

- bunds constructed across the river every year after floods
- replaced with permanent headworks when demand of water increases

Permanent Headworks - all important headworks

Location of Headworks on Rivers

Rivers have four stages: (i) rocky, (ii) boulder, (iii) trough (or alluvial), and (iv) delta

Rocky stage: far away from command arca; length and, therefore, cost of main canal increases, so is unsuitable for headworks

Delta Stage: irrigation demand is less and, also, nature of river (flat slopes, braiding) poses other problems, hence unsuitable

Boulder and alluvial stages - both suitable

The favorable features of boulder stage for headworks are:

- initial cost is less
- local availability of stones
- smaller width of river (therefore, weir)
- smaller scour depths (reduce the depth of cut-off)
- close proximity of higher banks (less training works)
- canal will have number of falls – can be utilized for power generation
- construction of temporary bunds (for initial period) possible
- will require large number of cross-drainage structures
- considerable loss of water through subsoil flow in the river bed and from the head reach of the main canal - crucial during periods of short supply

Hilly regions usually have wet climate and, therefore, irrigation demand is, generally, small to begin with and may increase later. In alluvial regions the demand for irrigation is high right from the beginning.

Site Selection for Headworks

River reach should be:

- straight and narrow
- well-defined and non-erodible high banks
- preferably deep channels on both banks and shallow channel at the centre

Based on considerations of suitability of site for different components of headworks following points are important:

- weir (barrage) - minimum length for economy, uniform flow for proper functioning
- under sluices - presence of deep channel to ensure adequate supply to the off-taking canal
- canal alignment - capable of serving its command area without much excavation
- sediment – off taking channel sited on the downstream end of the outer side of the bend

Diversion Headwork Components

- Weir (or barrage)
- Undersluices
- Divide Wall
- Fish Ladder
- Canal Head Regulator
- Sediment Excluder and Sediment Ejector
- River Training Works

Layout of Headworks

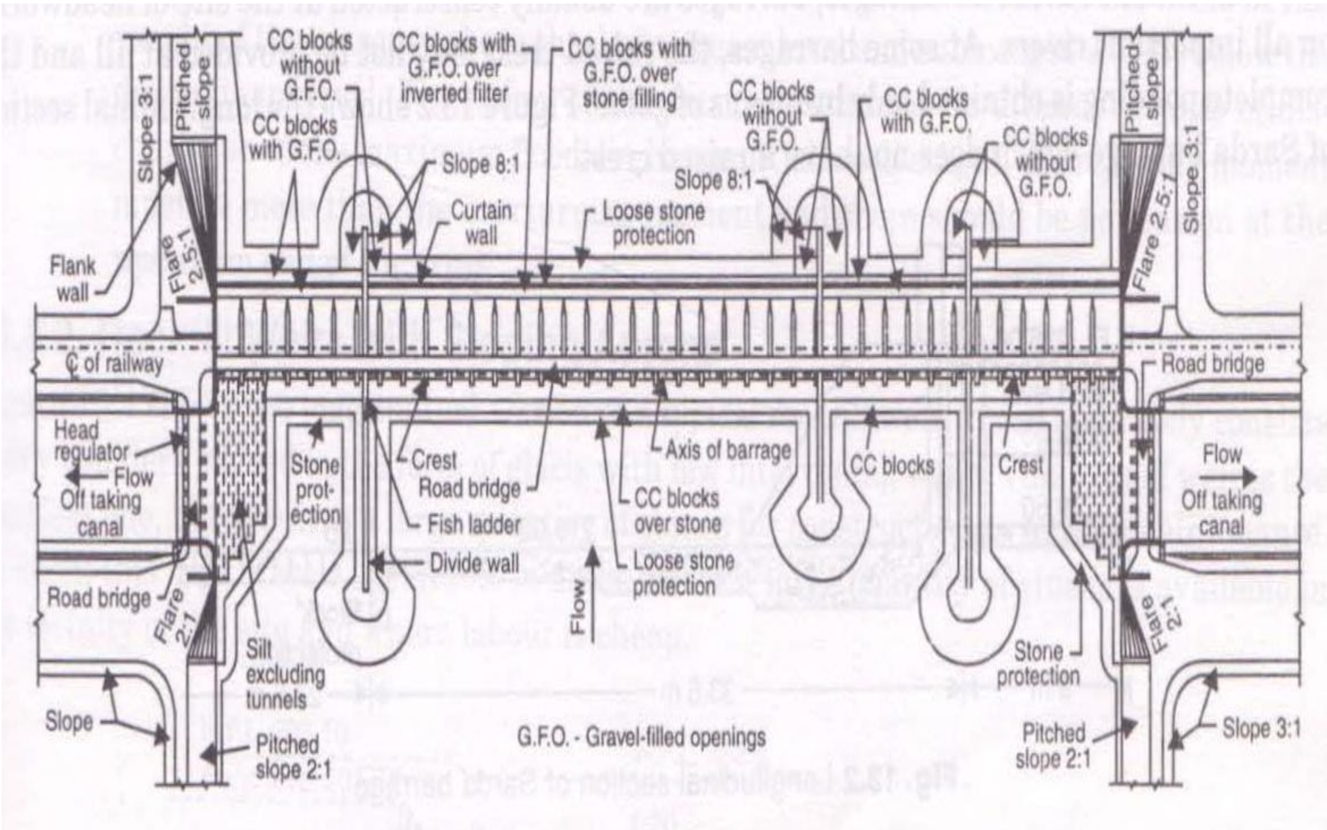


Figure 1. Typical layout of headworks

Weir (or Barrage)

Weir

- ungated barrier across a river
- raises water level in the river and diverts water into an off taking canal - on one or both banks of the river - just u/s of the weir
- usually aligned at right angle to the direction of flow - results in minimum length & normal uniform flow through all weir bays which minimizes the chances of shoal formation and oblique flow
- crest is raised above the river bed to raise the water level
- shutters at the top of the crest for further raising of water level and controlling pond level (difficult when pond level is higher than 2 m above the crest)
- provide gate-controlled weir - barrage

Barrage

- a gate-controlled weir with its crest at a lower level
- ponding up of the river for diversion is by means of gates
- offer better control on outflow and discharge in the offtaking canal
- afflux is small due to lower crest level of the barrage
- possible to provide a roadway across the river at small cost
- better control over sediment entry into canal
- Therefore, barrages are very common on all important headworks at times no raised crest as in Sarda barrage - ponding is by gates only design procedure is similar to that of weir

Types of Weirs

Masonry weirs with vertical d/s face

- masonry floor with a masonry crest on top of which shutters for ponding
- shutters dropped during floods to reduce afflux
- stability of crest examined for water level upto the top of shutter with no flow d/s when shutters dropped and water is on both sides of the crest

Rockfill weirs with sloping aprons

- simplest but requires large qty. of stones for constn. & maintenance

Concrete weirs (or barrages) with glacis

- on pervious foundation, only concrete weirs these days
- excess energy dissipated by means of hyd. Jump formed on glacis
- design based on Khosla's method and requires the knowledge of
- max. flood discharge & corresponding level around the weir site
- the stage - disch. Curve at the weir site
- the X-section of the river at the site

Based on the site conditions, general & economic considerations, decide

- afflux
- pond level
- min. waterway
- weir crest level

Undersluices (Sluice Ways or Scouring Sluices)

- Undersluices help in flushing the sediment deposited u/s of the canal head regulator on account of ponding up of water due to construction of weir across river
- gate-controlled openings in continuation of the weir with their crests at lower level

- useful for passing low floods after meeting the requirements of the offtaking canal
- shutters (or gates) operated only for passing high floods during monsoon
- located on the same side as the offtaking canal
- design procedure is similar to that of weir (use model analysis for major

headworks) Crest level of undersluice:

- generally coincides with the lowest cold weather level of the river bed at the weir site
- at least 1.2 m (2 m if sediment Excluder is provided) below that of the head regulator so that the sediment deposited u/s of the regulator does not enter the off taking canal. If needed, the crest level of the regulator is raised.

Discharge capacity of the undersluices is maximum of the following:

- twice the canal discharge - to ensure sufficient scouring capacity
- 10 - 20 % of the max. flood discharge - to reduce the length of the weir
- enough capacity to pass off low floods with w/s in the reservoir at pond level to avoid gate operation

Afflux & Waterway

- HFL u/s of weir rises due to construction of weir across river, this rise is afflux Afflux = u/s TEL - d/s TEL
- initially, the afflux is confined to a short reach u/s of weir but, gradually extends very far u/s in case of alluvial rivers - due to continued deposition
- afflux governs top levels of guide banks & marginal bunds & length of bunds
- waterway & afflux are interdependent

    larger afflux results in lesser waterway
    increases the discharge intensity q which, in turn,
    increases the scour &, hence, cost of protection works
    d/s
    higher afflux also increases the risk of failure of river training works
- For alluvial rivers:

  afflux = 1m in upper and middle reaches of river
  = 0.3m in lower reaches with flat gradients

waterway = 1.1 to 1.2 times Lacey's regime perimeter for the design discharge
= 0.8 to 0.9 times regime perimeter in rivers with coarser bed material

- lesser waterway increases afflux & cost of protection & river training works
- larger waterway is uneconomical & causes oblique flow and silting in part of waterways

Loosenes factor:

- overall length of weir /min. stable width from regime criterion

Pond level:

- water level which must be maintained in the under sluice pocket (i.e. u/s of the canal head regulator) so as to maintain FSL in the canal when full supply discharge is fed into it

= FSL + (1.0 to 1.2 m) so that sufficient working head is available even when head reach of canal has silted up or when canal is to be fed excess water

- limit on pond level : FSL = pond level - working head (1.0 to 1.2 m)
- maintained by keeping the weir crest at the pond level or by keeping the weir crest at lower level and provide shutters/gates

Retrogression:

- d/s of weir due to degradation; d/s HFL lowered; exit gradient increases during high floods : 0.3 to 0.5 m due to large qty. of sediment; during low floods : 1.25 to 2.25m due to relatively clear water for design flood retrogression is assumed 0.3 to 0.5 m

d/s TEL = u/s TEL - head loss (= afflux + retrogression)

Divide Wall

- parallel to canal head regulator
- separates main weir bays (and floor) from undersluice bays (and floor)
- extends on both sides of weir upto the end of the floor or loose apron
- on d/s it avoids cross flow causing scour near the structure
- isolates canal head regulator from main river
  - flow creates still pond in front of the regulator
  - helps in deposition of sediment and relatively clear water in canal
  - improves scouring of undersluices by ensuring straight approach
- additional divide walls if possibility of cross currents exist
- generally of strong masonry with top width of about 1.5 to 2.25m and nose end sloping at 3(V):1(H); slight divergence of 1 in 10 advisable; extending upto about the u/s end of the canal or half to 2/3rd the length of the regulator

Fish ladder

- fish of various kinds
- migrate to d/s in the beginning of winter in search of warmth
- return u/s before the monsoon for clear water
- a narrow opening (Fish ladder) between the divide wall and the undersluices where water is always present
- baffles to reduce the velocity to less than 3.0 m/s
- these openings are called as fish ladder or fishways or fish pass
- should take into account the requirements of the fish of the river

DESIGN OF WEIR

Data required:

1. L-section & x-section of river at the weir site
2. stage - Q relationship
3. sediment characteristics
4. data concerning the offtaking canal (FSL, Q, L- & X-sections)

Determination of weir crest level

1. Determine HFL for the design flood (50 to 100 yr. Frequency) from stage-Q curve.
2. d/s TEL = HFL + regime velocity head
3. u/s TEL = d/s TEL + permissible afflux + retrogression, if any
4. q = design flood discharge / width of clear waterway
5. determine k (height of TEL over the weir crest) from

   C is based on model studies, otherwise

   = 1.71 for broad-crested weir

   = 1.84 for small-crested weir
6. weir crest level = u/s TEL - k

Length & number of weir bays & height of shutters/gates:

Calculate total discharge capacity of weir and undersluices taking into account the end contractions

K = f(pier shape) may vary from 0.01 to 0.1

L = overall waterway length

n = number of end

contractions H = total head

over the weir

Height of shutters/gates = pond level - weir crest level (max. value = 2m for falling shutters)

Vertical cutoffs

- Vertical cutoffs at both u/s & d/s ends of the weir and also at intermediate location (ends of slopes) guards against scouring at u/s & d/s and piping at d/s
- Intermediate cutoffs hold the main structure in case of failure of u/s & d/s cutoffs bottom of cutoff is lower than the level of possible scour
- d/s cutoff should also reduce exit gradient to safe value the depth of scour below HFL

—

q calculated taking into account the concentration factor by which q is to be multiplied to take into account the non-uniformity of flow along the waterway during the operation of weir bays

The scour depth R (for regime conditions) increased further as follows: u/s of

impervious floor 1.50 R

d/s of impervious floor 2.00 R

Nose of guide banks/divide wall 2.25 R Transition

from nose to straight part 1.50 R Straight reaches of

guide banks 1.25 R

Weir crest, glacis and impervious floor:

- Weir crest at the computed level with a width of about 2m
- For broad-crested weir, width should be greater than 2.5 times H
- u/s slope of weir : 2(H):1(V) to 3(H):1(V)
- d/s slope and horizontal floor (i.e. stilling basin) should cause max. dissipation of energy through hydraulic jump and also be economic
- Slope of d/s glacis : 3(H):1(V)
- Level of the d/s floor so that jump starts at the end of the glacis (or u/s) for all Q’s
- Location of the jump is calculated for HFL and pond level discharges

- Level of floor is at lower of the required levels for these two conditions
- length of floor such that the entire jump is on the floor (5 to 6 (h -h))
- u/s floor at the river bed level and its length so that the resulting exit gradient at the d/s end is less than the safe value for the soil under consideration (1/5 to1/6 for coarse and 1/6 to 1/7 for fine sand, 1/4 to 1/5 for shingles)
- Thickness of the floor from uplift considerations

  Note: For the estimation of uplift pressures on the Weir floor for deciding the floor thicknesses, as per the Khosla's theory, please consultant any standard book on Irrigation and Hydraulic Structures, like Arora KR Book on Irrigation, Water Power and Water Resources Engineering.

Upstream and downstream loose protection

- in the form of concrete blocks and loose stones for protection against scour
- u/s: concrete blocks of size 1500x1500x900 mm laid over loose stones for a distance equal to depth of scour below the floor level
- d/s: concrete blocks on inverted filter; space between blocks filled with gravel; length is 1.5 times the scour depth below the floor level.
- for the boulder reach the size of blocks will be increased
- Inverted filter in two or more layers
- toe wall of masonry/concrete at the end of the filter to a depth of about 500mm
- launching aprons beyond block protection on u/s & d/s ; stones larger than 300mm

Canal Head Regulator

- regulates the discharge into the canal
- controls the entry of sediment into the canal
- usually aligned at an angle of 90 - 110 degrees to the barrage axis minimises entry of sediment into the canal, prevents backflow and stagnation zones in the undersluice pocket
- discharge controlled by steel gates of 6 - 8 m spans usually; larger spans also used - electric winches required for operation
- pond level = FSL + 1.0 to 1.2 m of working head
- waterway width - so that the canal can be fed its full supply with 50% of working head if more than the canal width, provide a converging transition d/s of regulator
- The required head over the crest H for passing a discharge Q with an overall waterway L is worked out using

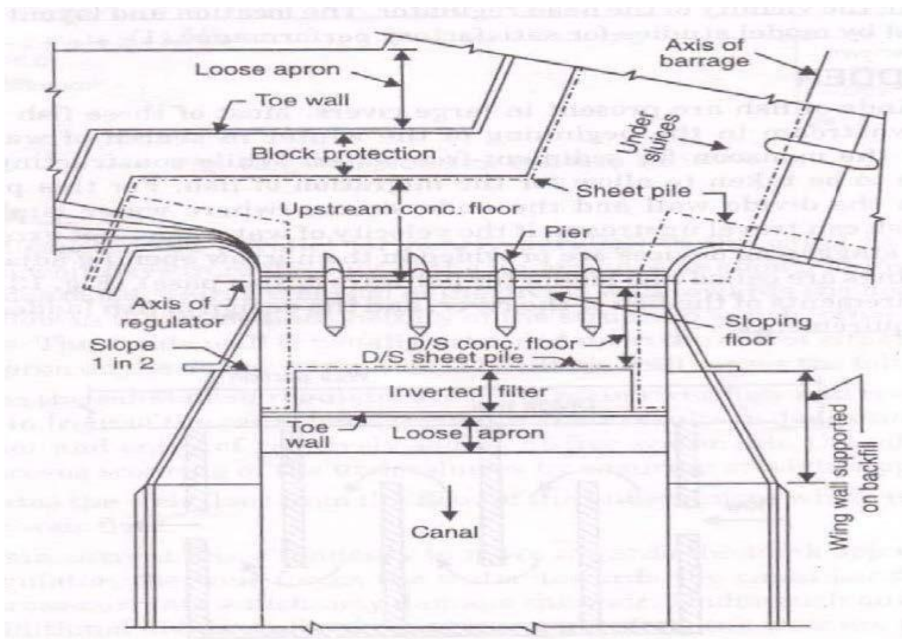
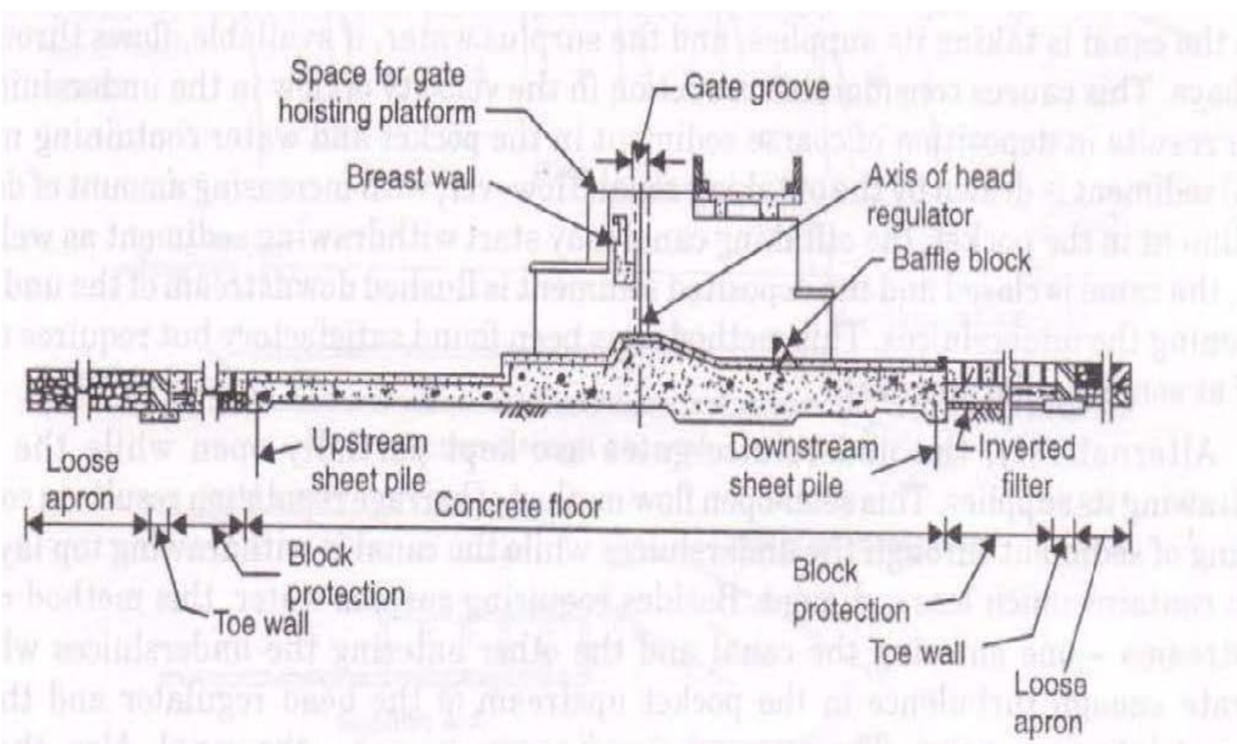


Figure 2. Typical Plan of a Head Regulator

- crest level = pond level - head over the crest required to pass the full supply Q kept higher than the cill of the undersluices to prevent sediment, entry should also take into account sediment excluder, working head waterway width
- Height of gates = pond level - crest level
- RCC breast wall between HFL and pond level



- Canal head regulator is designed as weir
- Canal kept closed during highest flood passing through the river, this is the worst static condition and the floor should resist this uplift
- jump trough region - worst condition may occur when some discharge passes safety of this part checked for different discharges including the maximum one
- a bridge and a working platform (for operation of gates) across the regulator

Sediment control in canals

Eexcess sediment entering the canal reduces its capacity, therefore, adequate preventive or curative measures for sediment control entry of sediment can be partly controlled by barrage regulation methods.

1. Still pond method:

- undersluices kept closed when canal takes its supplies
- excess water flows through weir bays
- causes still pond condition
- sediment deposited at the bed; canal draws clear water
- when considerable sediment deposited, canal is closed and sediment is flushed d/s of the undersluices though satisfactory, requires frequent closing of the canal

2. Semi-open flow method:

- undersluices gates are kept partially open while canal is taking its supplies
- results in continuous flushing of sediment
- requires surplus water
- the two streams - to river & to canal generates turbulence, bring sediment into suspension and may enter canal; not suitable except during floods when water is surplus

3. Wedge-flow method:

- undersluices near the divide wall are opened more
- the undersluices near the regulator are opened less
- results in wedge-like flow resulting in favourable flow curvature in the undersluice pocket and, thus, reduces sediment entry into canal

Some important points:

- When the stream carries high sediment load, close the canal itself
- Barrage regulation methods have their limitations requiring either closure of canal or surplus water. Therefore, sediment excluder/ejector and stilling basins are constructed.
- Sediment excluders/ejectors take advantage of the fact that the bed load part of the

transported sediment in a stream moves near bed and the suspended load part is distributed non-uniformly in the vertical with heavier concentrations near the bed.

- Settling basins reduce the sediment transport capacity of the canal flow by enlarging the flow cross-section over the length of the basin. The deposited sediment is suitably removed.

Sediment excluder (or silt excluder)

- most commonly used preventive measure
- constructed in river bed in front of the canal regulator
- flow u/s of regulator divided by a platform below which excluder tunnels
- only upper layer water enters the canal; carries much less sediment

- lower layer water passes d/s of weir/barrage through the excluder tunnels
- tunnels are parallel to the axis of the regulator and are of different lengths
- all tunnels terminate at the end of the undersluice bays
- tunnels accommodated in the space between regulator crest and undersluice floor
- design procedure based on thumb rules evolved from past experiences

1. min. discharge through excluder tunnels: about 20% of canal discharge
2. self-flushing velocity in tunnels depend on sediment size: 1.8 to 4.0 m/s
3. usually, 2 to 6 tunnels: usually rectangular x-section and bell-mouthed
4. tunnels accommodated in the space between regulator crest floor and undersluice floor - determines height of the tunnels keeping in mind the self-flushing velocity and convenience for inspection & repair
5. estimate the waterway width required for tunnels
6. divide the width into a suitable number (whole number for one sluice bay) of one sluice bay) of tunnels
7. length of the tunnel nearest the regulator equals the length of the regulator
8. other tunnels are successively shorter - mouth of longer is in the suction zone of the shorter one - so that no dead zone between adjacent tunnels
9. water and sediment discharge of all the tunnels should be the same this requires width of shorter tunnel to be smaller to satisfy resistance condition
10. designed excluder is model-tested

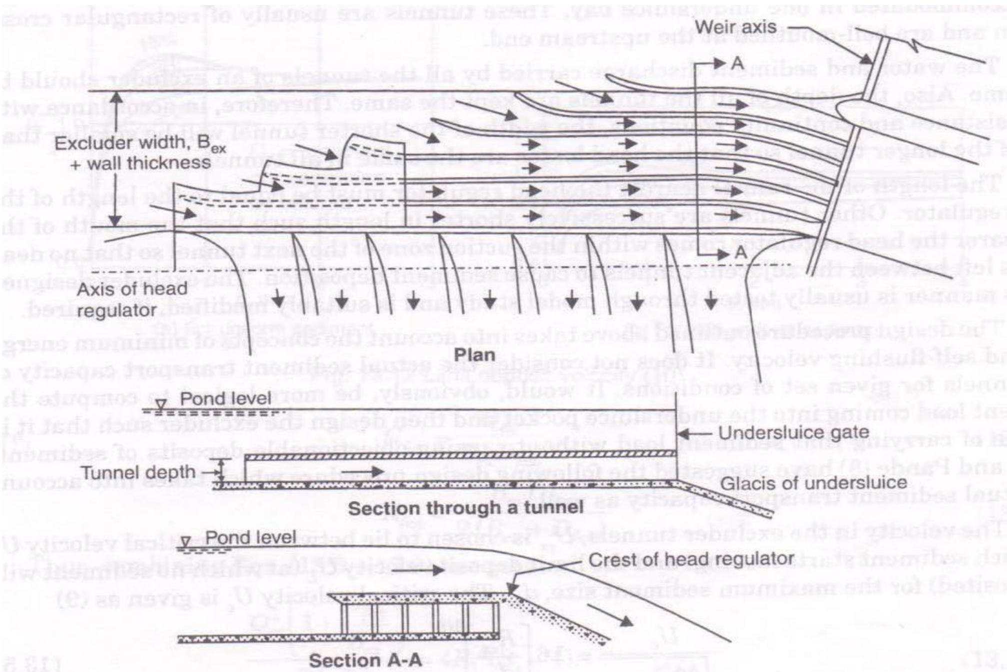


Figure 4. Typical Layout a Sediment Excluder

Above design procedure is based on self-flushing concept and does not take into account the sediment transport capacity of the tunnels. Garde and Pande have suggested an alternative method for this purpose.

Sediment Ejector /Extractor (Or Silt Ejector/Extractor)

- curative measure; removes excess sediment load that has entered the canal
- constructed in canal d/s of the canal head regulator
- taking advantage of the concentration distribution, the near-bed water layers ejected
- not too near the regulator - residual turbulence keeps the sediment in suspension
- not too far from regulator - sediment deposits d/s of regulator and reduces canal capacity reach up to the ejector to be wide to carry extra discharge for ejector
- ejector spans the entire width of the canal ; divided into tunnels which, in turn, are subdivided with gradually converging turning vanes to accelerate the escaping flow
- main components - diaphragm, tunnels, control structure and an outfall channel
- diaphragm so shaped as to cause least disturbance to sediment concentration u/s of it
- diaphragm level = f(sediment size to be ejected, size of tunnels and u/s & d/s canal levels)
- lower side of u/s end of diaphragm is bell-mouthed/or made elliptical
- escaping discharge - generally 10 to 20 per cent of full supply discharge
- tunnel dimensions - resulting velocity is adequate to carry the sediment of desired size about 20-25% depth of flow in the canal
- tunnels further converged to increase the velocity further by 10-15%; range 2.5 - 6m/s
- depth of tunnels; 1.8 - 2.2 m to facilitate inspection and repair
- ejector discharge is controlled by regulator gates(near the outfall)
- outflow led to natural drainage through outfall channel design to have self-cleansing velocity
- sufficient drop between FSL of the outfall channel and HFL of the drainage
- proposed design model-tested

The efficiency E of the sediment ejector (or excluder) is given as:

IU and ID are sediment concentration in the canal u/s and d/s of the ejector.

Settling Basin:

- removal of sediment from flowing canal by reducing flow velocity through a long expansion
- reduces velocity, shear, turbulence which stops sediment movement and also deposition of suspended sediment
- material from the bed of the basin suitably removed and disposed of

Design of settling basin:

- size of sediment particle = d
- fall velocity of the particle = w
- depth of flow in the basin = D
- velocity of flow in the basin = U
- Time required by a particle on the w/s to reach the bed of the basin = D/w
- Horizontal distance travelled by the particle during this time = UD/w (i.e length of basin, L)
- Fall velocity is affected by turbulence, concentration etc.
- Therefore, length of basin L = UD/w is increased by about 20%.

Garde, Ranga Raju and Sujudi method for design of settling basin:

- Efficiency of removal of sediment (n) by settling basin:

q_{si} & q_{se} are amounts of sediment of a given size entering and leaving the basin

- Based on dimensional analysis and experimental investigation, they obtained

n_o & K are related to w/u* (u* is the shear velocity).

River training works for canal headworks

Purposes:

- to prevent outflanking of the structure
- to minimise cross flow (through weir) which may endanger the structure
- to prevent flooding of the riverine lands u/s of the weir
- to provide favourable curvature of flow u/s of the head regulator

Types (usually provided):

- Guide banks to narrow down & restrict the course of the river so that it flows centrally
- Approach embankments aligned with the weir axis and extend up to a point beyond the range of the anticipated meander loop
- Afflux embankments - earthen embankments extending from both approach embankments connected to the u/s ground above the affluxed highest flood level
- Spurs
- Launching apron, Stone pitching etc.

Module – 4

Canal Falls, Cross Drainage Works

CONCEPT:-

The two major categories of structures that are built on canals are the regulation works and cross-drainage works. A brief description of some of the important ones is given below.

Regulation Works

Canal Falls

While canals are designed with a slope which is close to the regime slope, the ground slope may differ from it considerably. Many a times, the ground slope is more than the canal slope and this may result in a canal in heavy filling. To overcome this situation, the canal has to be provided with falls which require a masonry or concrete work. The drop in canal bed results in the potential energy of water being converted to kinetic energy and this excess energy has to be dissipated before allowing the flow over the unprotected canal bed. Also, the water surface upstream of the fall also needs to be maintained at its normal level. The fall thus has to be provided with a crest and some means of energy dissipation. The fall can be flumed or unflumed. In a flumed fall, the trapezoidal canal section is contracted to a rectangular section having a width less than the bed width of the canal and expanded back after the works. In unflumed falls, while there is no reduction of the bed width, the section is however converted into a rectangular one. Only two types of falls are discussed here.

Vertical drop fall

This type of fall depends on a vertical impact for energy dissipation. The crest height is determined using the formula

$$Q = C\,L\,H^{3/2}$$

Where C can be taken as 1.71, L is the length of the crest and H the head over the crest.

The energy dissipation in this case is by vertical impact of the water on the downstream bed and a pool of water. A cistern of certain depth and length- known as the cistern element is provided. There are many empirical formulae available to compute the length and depth of the cistern such as the one given by UP Irrigation Research Institute (UPIRI) as below is SI units.

$$L_c = 5\sqrt{E\,H_L}$$

$$x_c = 0.25\,(E\,H_L)^{2/3}$$

in which L_c and x_c are the length and the depth of the cistern respectively neglecting velocity head, E can be taken equal to head H over the crest and H_L equal to drop in canal bed levels.

Glacis Fall

This type of fall is preferred for larger drops and utilizes the hydraulic jump for energy dissipation .The crest is joined to the upstream floor at a slope of 1:1, while the downstream glacis is generally at at slope of 2H:1V. The downstream floor may be carried to a level lower than the canal bed for certain length to provide a cistern, with the length and depth of the cistern being 1.25 E_2 and 0.25 E_2 respectively, where E_2 is the downstream specific energy.

Distributary Head Regulator

This is the work provided at the head of a branch canal or a distributary and serves the purpose of controlling and regulating the flow into the offtake as well as metering of the flow. The arrangement is more or less similar to that of a canal head regulator, with a raised crest, upstream and downstream floors and cutoffs. The width of the regulator and height of crest are fixed such that the offtake may be able to draw its full supply discharge even if the water level in the parent channel is lower than the full supply level. On smaller works, the control is in the form of wooden planks which can be placed in grooves provided in piers for this purpose, while on larger works manually operated gates are provided. Curved vanes or cantilever platform as discussed separately are usually provided to control entry of excess sediment into the offtake

Cross Regulator

Cross regulators are structures constructed across a canal and spanning its entire width. The width is divided into suitable number of spans and provided with gates so as to regulate the flow in the canal downstream of the regulator. Cross regulators serve many purposes such as

(i) If the canal downstream of the cross regulator has to be closed in an emergency, the cross regulator gates can be closed and the discharge diverted to any drain. This requires an escape to be constructed just upstream of the cross regulator.

(ii) The canal water level upstream of the cross regulator can be regulated depending upon the gate openings. This may be required if the canal is carrying less than the full supply discharge and some offtake upstream has to be supplied with its full supply discharge.

Cross Drainage Works

These are works provided at the crossing of a canal and a stream. Depending on whether the canal crosses the stream at top, bottom or at the same level, these are divided into three categories.

Aqueducts and Siphon Aqueducts

Aqueducts are works where the canal crosses over the stream and the high flood level of the stream is lower than the canal bed level so that the flow in the stream remains an open channel flow. The canal section may cross over the stream without any modification i.e. with the banks as they are or with slight modification wherein the outer edges of the banks are replaced by retaining walls. Such works are however suitable only when the stream to be crossed is small. For any major work, the canal is flumed to a rectangular trough – masonary or concrete – and after the crossing restored to its normal section Siphon aqueducts are provided when the high flood level of the stream is higher than the canal bed level. In such a case the flow in the stream becomes a pressure flow through the siphon barrels The design of aqueducts and siphon aqueducts requires consideration of the following factors:

Waterway of stream- The waterway provided in an aqueduct is generally close to the Lacey's regime perimeter. This helps in developing a stable channel upstream of the works without much silting or scouring. The width is divided into suitable spans with the help of piers. In a siphon aqueduct, the velocity in the barrels becomes one of the considerations in deciding the waterway.

Headway i.e. the clearance between the downstream bed of the stream and the bottom of the canal trough should be sufficient so as to prevent the blockage of the barrels. While this may not present much of a problem in aqueducts, lowering of the stream bed upstream of the siphon barrels may have to be resorted to at times

Afflux will be caused by the flow of stream under the canal trough. This results because of the head loss due to constriction, piers or siphon. Afflux can be computed using appropriate formulae for the head loss and is used in determining the hydraulic grade line specially in case of siphon aqueducts.

Fluming of the canal requires contraction as well as expansion transitions. While the splay in contraction can be kept about 1:2, the expansion is generally provided with a splay of 1:3 or more. Suitable design of transitions for contraction and expansion is required and procedures for this design are available.The considerations of uplift and exit gradient have also to be taken care of. The worst condition for the stream bed being when the stream is dry and the canal is carrying its full supply discharge. In case of siphon aqueducts, the canal trough is also subjected to uplift when the canal is dry and the stream is in high flood.

Superpassages and Siphons

These are works where the stream crosses over the canal. In a superpassage the canal full supply level is lower than the river bed level and the flow in canal is an open channel flow. In a siphon the canal full supply level is higher than the stream bed level and therefore the canal water flows under pressure through barrels under the stream trough.

The design considerations for these works are similar to those for aqueducts and siphon aqueducts. The stream however is not flumed and mostly carried with the original section.

Level Crossing

In this work the canal and stream cross at nearly the same level. There is intermixing of the canal and river water and the flow is controlled by regulator gates on the canal as well as the stream. A sill with its top at the canal full supply level is provided on the upstream side of the stream to prevent stagnant water pool in the stream during dry season.

Level crossings have a problem with sediment getting deposited in the pool formed at the crossing. This could lead to degradation in the river downstream. Also there is need for constant watch and warning mechanism so that the stream gates could be opened well in time in case a flood has to be passed. The canal may also have to be closed during floods to prevent the river sediment from entering the canal.

Selection of Type

The selection of the type of cross drainage work depends on the relative bed levels of the canal and the stream at the crossing and their discharge. Thus in case the stream is carrying a large discharge, it may not be feasible to siphon it under the canal even though the levels may dictate a siphon aqueduct. The type of crossing can be altered by a suitable realignment of the canal if required, resulting in change in bed levels of both the canal and the stream.

A cross drainage work is a structure carrying the discharge from a natural stream across a canal intercepting the stream.

Canal comes across obstructions like rivers, natural drains and other canals.

The various types of structures that are built to carry the canal water across the above mentioned obstructions or vice versa are called cross drainage works.

It is generally a very costly item and should be avoided by

- Diverting one stream into another.
- Changing the alignment of the canal so that it crosses below the junction of two streams.

Types of cross drainage works

Depending upon levels and discharge, it may be of the following types:

Cross drainage works carrying canal across the drainage:

Aqueduct:

When the HFL of the drain is sufficiently below the bottom of the canal such that the drainage waterflows freely under gravity, the structure is known as Aqueduct.In this, canal water is carried across the drainage in a trough supported on piers.

Bridge carrying water

Provided when sufficient level difference is available between the canal and natural and canal bed is sufficiently higher than HFL.

Crossing works: (aqueducts)

Siphon Aqueduct:
In case of the siphon Aqueduct, the HFL of the drain is much higher above the canal bed, and water runs under siphonic action through the Aqueduct barrels.
The drain bed is generally depressed and provided with pucci floors, on the upstream side, the drainage bed may be joined to the pucca floor either by a vertical drop or by glacis of 3:1. The downstrean rising slope should not be steeper than 5:1. When the canal is passed over the drain, the canal remains open for inspection throughout and the damage caused by flood is rare. However during heavy floods, the foundations are succeptible to scour or the waterway of drain may get choked due to debris, tress etc.

Cross drainage works carrying drainage over canal:
The structures that fall under this type are:

Super passage

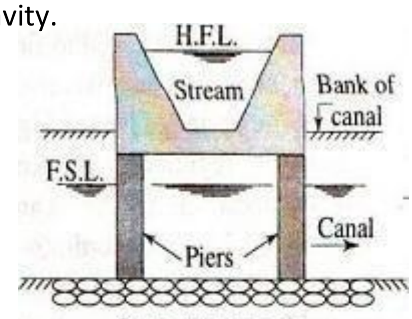
Canal siphon or called syphon only

Super passage:

The hydraulic structure in which the drainage is passing over the irrigation canal is known as super passage. This structure is suitable when the bed level of drainage is above the flood surface level of the canal. The water of the canal passes clearly below the drainage

A super passage is similar to an aqueduct, except in this case the drain is over the canal.

The FSL of the canal is lower than the underside of the trough carrying drainage water. Thus, the canal water runs under the gravity.
Reverse of an aqueduct



Super passage

Canal Syphon:

If two canals cross each other and one of the canals is siphoned under the other, then the hydraulic structure at crossing is called “canal siphon”. For example, lower Jhelum canal is siphoned under the Rasul-Qadirabad (Punjab, Pakistan) link canal and the crossing structure is called “L.J.C siphon”

In case of siphon the FSL of the canal is much above the bed level of the drainage trough, so that the canal runs under the siphonic action.

The canal bed is lowered and a ramp is provided at the exit so that the trouble of silting is minimized.Reverse of an aqueduct siphon

In the above two types, the inspection road cannot be provided along the canal and a separate bridge is required for roadway.
For economy, the canal may be flumed but the drainage trough is never flumed.

Selection of suitable site for cross drainage works

1. The factors which affect the selection of suitable type of cross drainage works are:
2. Relative bed levels and water levels of canal and drainage
3. Size of the canal and drainage.

4. The following considerations are important

5. When the bed level of the canal is much above the HFL of the drainage, an aqueduct is the obvious choice.

6. When the bed level of the drain is well above FSL of canal, super passage is provided.

7. The necessary headway between the canal bed level and the drainage HFL can be increased by shifting the crossing to the downstream of drainage. If, however, it is not possible to change the canal alignment, a siphon aqueduct may be provided.

8. When canal bed level is much lower, but the FSL of canal is higher than the bed level of drainage, a canal siphon is preferred.

9. When the drainage and canal cross each other practically at same level, a level crossing may be preferred. This type of work is avoided as far as possible.

Factors which influence the choice / Selection of Cross Drainage Works

The considerations which govern the choice between aqueduct and siphon aqueduct are:

1. Suitable canal alignment
2. Suitable soil available for bank connections
3. Nature of available foundations
4. Permissible head loss in canal
5. Availibility of funds

Compared to an aqueduct a super passage is inferior and should be avoided whenever possible. Siphon aqueduct is preferred over siphon unless large drop in drainage bed is required.

Classification of aqueduct and siphon aqueduct

Depending upon the nature of the sides of the aqueduct or siphon aqueduct it may be classified under three headings:

Type I:

Sides of the aqueduct in earthen banks with complete earthen slopes. The length of culvert should be sufficient to accomodate both, water section of canal, as well as earthen banks of canal with aqueduct slope.

Sides of the aqueduct in earthen banks, with other slopes supported by masonry wall. In this case, canal continues in its earthen section over the drainage but the outer slopes of the canal banks are replaced by retaining wall, reducing the length of drainage culvert.

Type II:

Sides of the aqueduct made of concrete or masonry. Its earthen section of the canal is discontinued and canal water is carried in masonry or concrete trough, canal is general flumed in this section.

UNIT-V

WATER POWER ENGINEERING

Introduction

The water of the oceans and water bodies on land are evaporated by the energy of the sun‟s heat and gets transported as clouds to different parts of the earth. The clouds travelling over land and falling as rain on earth produces flows in the rivers which returns back to the sea. The water of rivers and streams, while flowing down from places of higher elevations to those with lower elevations, loose their potential energy and gain kinetic energy. The energy is quite high in many rivers which have caused them to etch their own path on the earth‟s surface through millions of years of continuous erosion. In almost every river, the energy still continues to deepen the channels and migrate by cutting the banks, though the extent of morphological changes varies from river to river. Much of the energy of a rivers flowing water gets dissipated due to friction encountered with its banks or through loss of energy through internal turbulence. Nevertheless, the energy of water always gets replenished by the solar energy which is responsible for the eternal circulation of the Hydrologic Cycle.

Hydropower engineering tries to tap this vast amount of energy available in the flowing water on the earth‟s surface and convert that to electricity. There is another form of water energy that is used for hydropower development: the variation of the ocean water with time due to the moon‟s pull, which is termed as the tide. Hence, hydropower engineering deals with mostly two forms of energy and suggest methods for converting the energy of water into electric energy. In nature, a flowing stream of water dissipates throughout the length of the watercourse and is of little use for power generation. To make the flowing water do work usefully for some purpose like power generation (it has been used to drive water wheels to grind grains at many hilly regions for years), it is necessary to create a head at a point of the stream and to convey the water through the head to the turbines which will transform the energy of the water into mechanical energy to be further converted to electrical energy by generators. The necessary head can be created in different ways of which two have been practically accepted.

These are:

1. Building a dam across a stream to hold back water and release it through a channel, conduit or a tunnel (Figure 1)

2. Divert a part of the stream by creating a low-head diversion structure like barrage. (Figure 2)

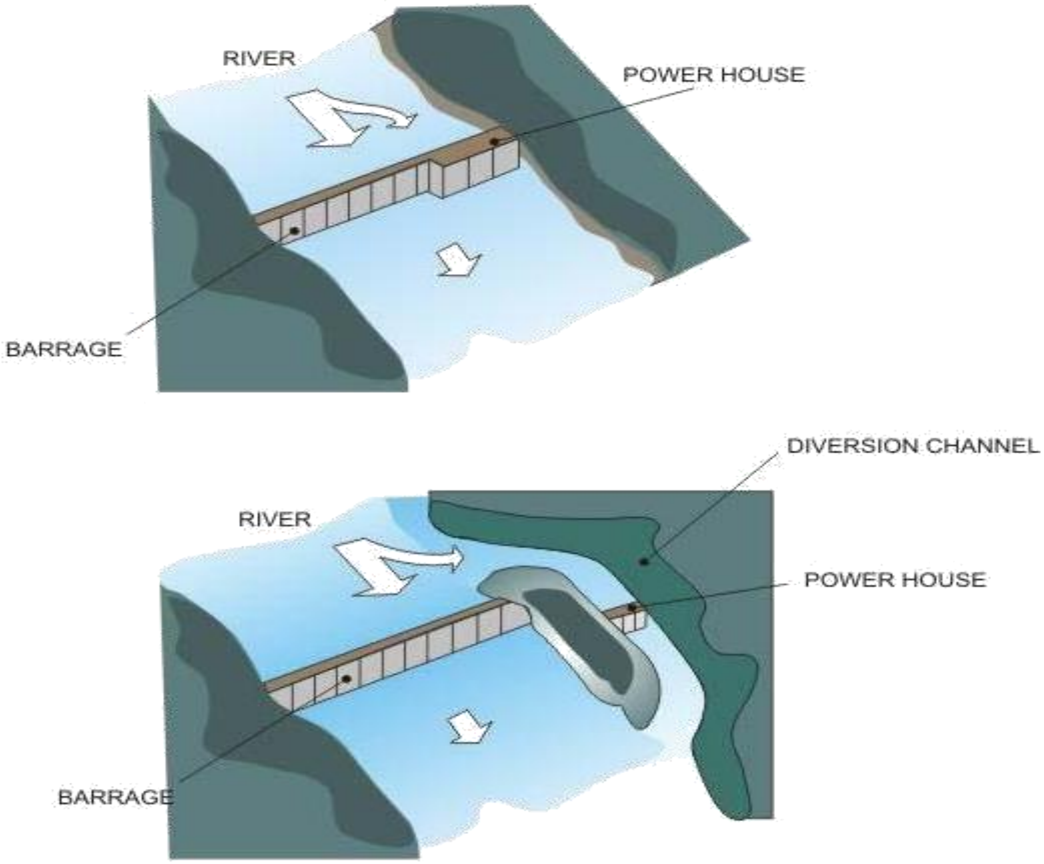


FIGURE 2. Power generation through streamflow diversion
(a) Power house as a part of diversion structure
(b) Power house on diversion channel

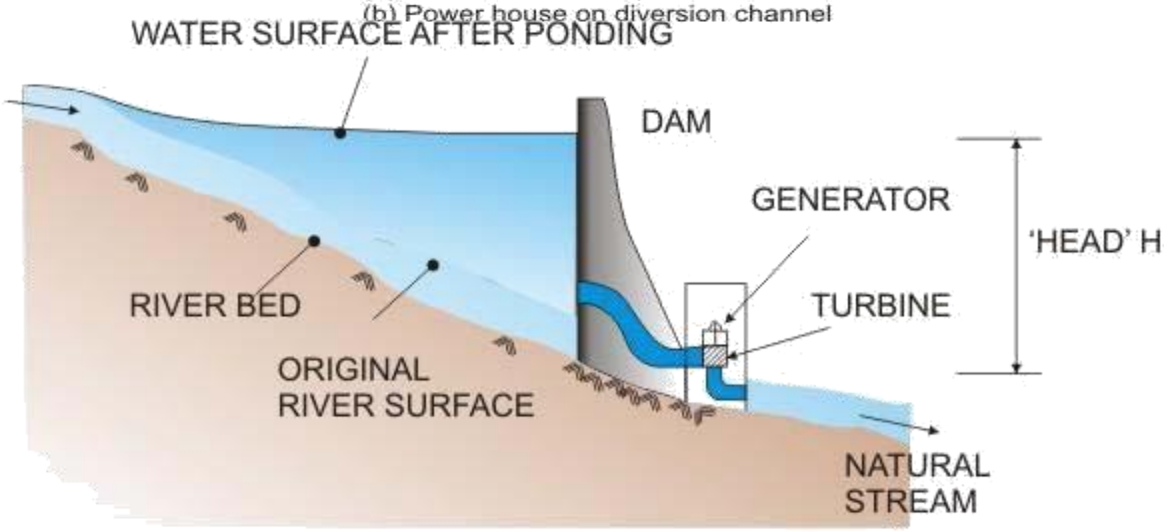


FIGURE 1. Creation of head by constructing a dam

A series of integrated power developments along the same watercourse form what may be called a multistage hydroelectric system in which each portion of the river with a power plant of its own is referred to as a stage (Figure 3). The head created by a dam put across a lowland river usually ranges from 30 to 40m. In mountainous terrain, it may run over 200m.

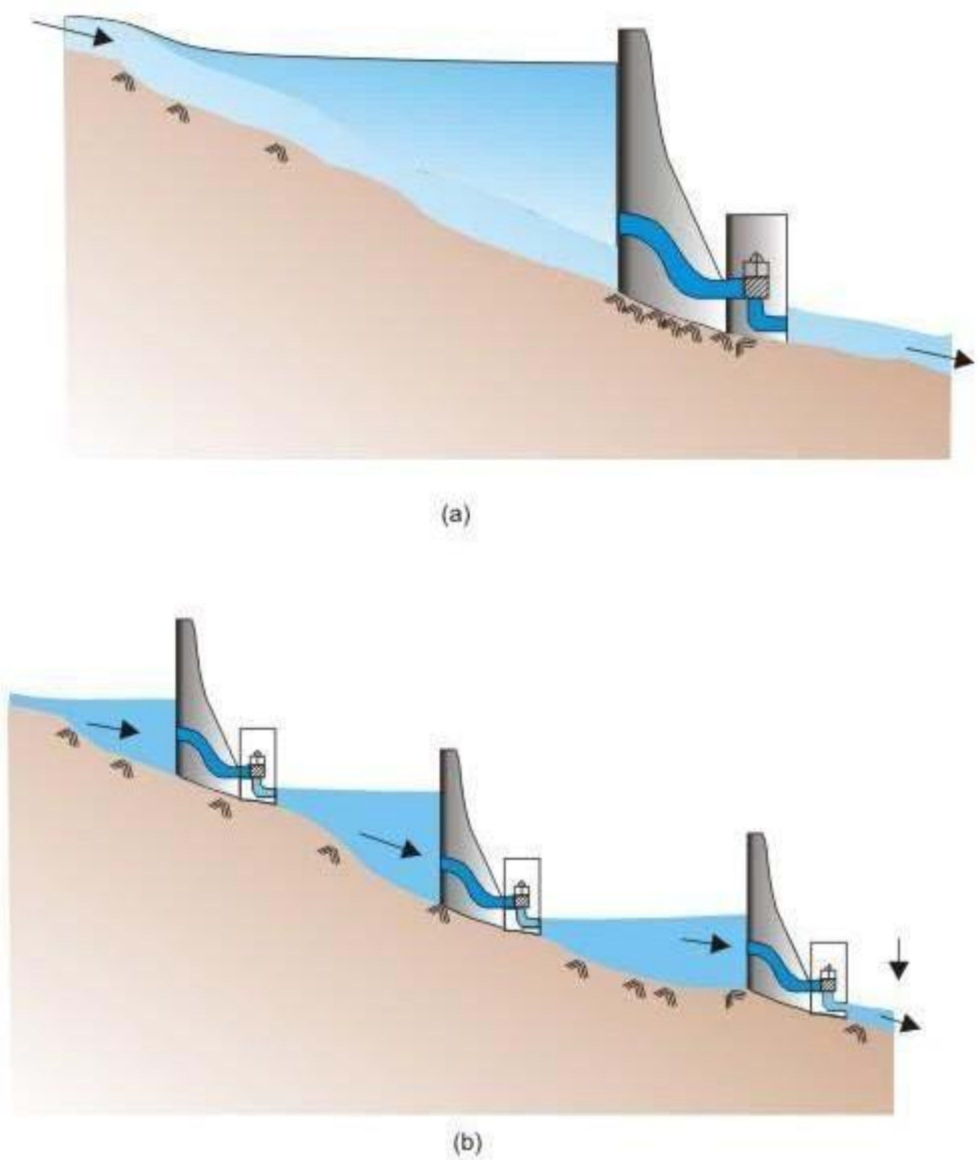


FIGURE 3 . (a) Single-stage hydro-power development scheme
(b) Cascade or Multi-stage hydro-system

The following sections briefly discuss the issues related to the fundamentals of hydropower project development.

Hydropower potential

Electricity from water is usually referred to as Hydro-Power, where the term „hydro" is the Greek word for water and hydropower is the energy contained in water. It can be converted in the form of electricity through hydroelectric power plants. All that is required is a continuous inflow of water and a difference of height between the water level of the upstream intake of the power plant and its downstream outlet.

In order to evaluate the power of flowing water, we may assume a uniform steady flow between two cross-sections of a river, with **H** (metres) of difference in water surface elevation between two sections for a flow of **Q** (m^3/s), the power (**P**) can be expressed as

$$P = \gamma\, Q \left(H + \frac{v_1^2 - v_2^2}{2g} \right) \quad \text{[Nm/s]}$$

where $\mathbf{v_1}$ and $\mathbf{v_2}$ are the mean velocities in the two sections. Neglecting the usually slight difference in the kinetic energy and assuming a value of γ as 9810N/m obtains the ^{2}, one expression of power as

$$P = 9810QH \quad \text{[Nm/s]}$$

Since an energy of 1000Nm/s can be represented as 1kW (1kilo-Watt), one may write the following:

$$P = 9.81QH \quad \text{[kW]}$$

The above expression gives the theoretical power of the selected river stretch at a specified discharge.

In order to evaluate the potential of power that may be generated by harnessing the drop in water levels in a river between two points, it is necessary to have knowledge of the hydrology or stream flow of the site, since that would be varying everyday. Even the average monthly discharges over a year would vary. Similarly, these monthly averages would not be the same for consecutive years. Hence, in order to evaluate the hydropower potential of a site, the following criteria are considered:

1. Minimum potential power is based on the smallest runoff available in the stream at all times, days, months and years having duration of 100 percent. This value is usually of small interest
2. Small potential power is calculated from the 95 percent duration discharge
3. Medium or average potential power is gained from the 50 percent duration discharge
4. Mean potential power results by evaluating the annual mean runoff.

Since it is not economically feasible to harness the entire runoff of a river during flood (as that would require a huge storage), there is no reason for including the entire magnitude of peak flows while calculating potential power or potential annual energy.

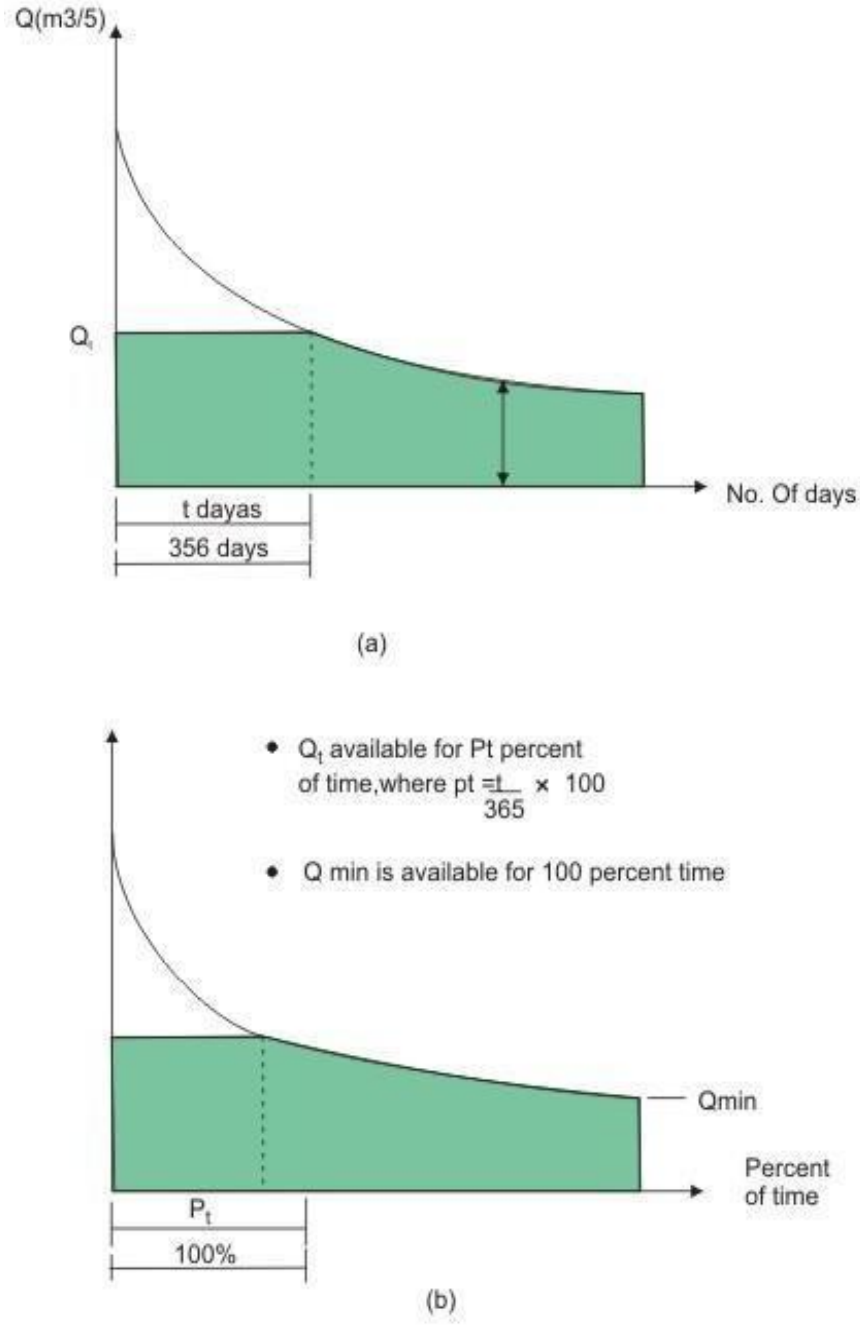


FIGURE 4. Flow curve for one year (a) expressed in time; (b) expressed in percentage of time

Hence, a discharge-duration curve may be prepared (Figure 4) which plots the daily discharges at a location in the decreasing order of magnitude starting from the largest daily discharge observed during the year and going upto the minimum daily discharge.

From this annual discharge curve, a truncation is made at a discharge Q_t which is the discharge corresponding to a time of „t" days, where t can be the median (say, 182 days or 50 percent duration, denoted by (Q_{182} or $Q_{50\%}$), or a higher Q_t (t less than 182 days) can be selected by specialists who are familiar with the local conditions and future plans for power supply. Accordingly, the annual magnitude of potential (theoretical) energy can be computed in KWh as below and referring to Figure 4:

$$E_p = 24 \times 9.81 H\, Q_t \,\square\, \sum_{i}^{365} Q_i$$

$$\approx 235 H \cdot A \qquad \text{(in kWh)}$$

Where Q_i denotes the daily mean flow during the period 365-t days and A, the hatched area cut by Q_t , where the area under the curve has a unit m^3 day/s.

The massive influx of water in the hydrologic cycle has an estimated potential for generating, on a continuous basis, 40,000 billion units (TWh) of power annually for the whole world (CBIP, 1992). Hydropower potential is commonly divided into three categories:

a) ***Theoretical*** : 40,000 TWh
b) ***Technical*** : 20,000 TWh
c) ***Economical*** : 9,800 TWh

The terms used above are explained below:

Theoretical

The gross theoretical potential is the sum of the potential of all natural flows from the largest rivers to the smallest rivulets, regardless of the inevitable losses and unfeasible sites.

Technical

From technical point of view, extremely low heads (less than around 0.5m), head losses in water ways, efficiency losses in the hydraulic and electrical machines, are considered as infeasible. Hence, the technically usable hydro potential is substantially less than the theoretical value.

Economic

Economic potential is only that part of the potential of more favourable sites which can be regarded as economic compared to alternative sources of power like oil and coal. Economically feasible potential, therefore, would change with time, being dependent upon the cost of alternate power sources. This potential is constantly updated and shows an increasing trend with the exhausting stock of fossil fuel.

The following table taken from CBIP (1992) shows a continental break-up of world‟s economical hydropower potential. Asia is seen to be endowed with the maximum hydropower potential.

Region	Available potential (Billion units)
Asia (except C.I.S and Russia)	2700
C.I.S and Russia	1100
Africa	1590
North America	1580
South America	1910
Europe(except C.I.S and Russia)	720
Oceania	200
Total	9800

Some nations have enough hydropower to become exporters of electricity. Switzerland, for example, exports electricity to neighbouring France and Italy. Nepal, Bhutan, Peru and Laos are similarly blessed with abundant hydro resources. Within India, Meghalay is probably the only state generating hydropower more than its requirements and exports power to the neighbouring state of Assam.

In India, it has been estimated by the Central Electric Authority, that the hydroelectric potential of the entire country is around 84,044MW at 60 percent load factor. The annual energy contribution of this potential would be about 600 billion units including seasonal/secondary energy which is the additional energy generation in any year above the firm annual energy.

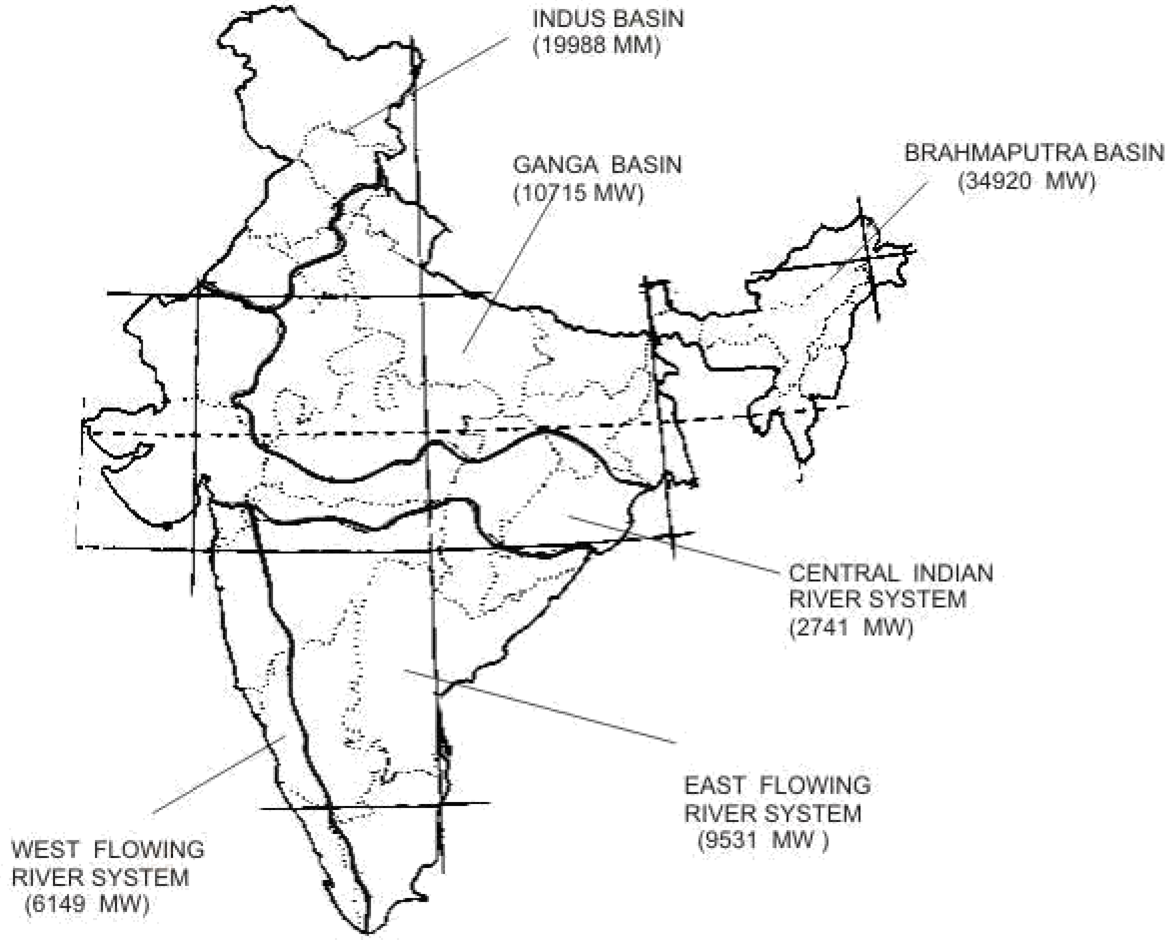


FIGURE 5. Basin-wise hydro power potential of India.

Types of hydroelectric projects

Hydroelectric plants are classified commonly by their hydraulic characteristics, that is, with respect to the water flowing through the turbines that run the generators.

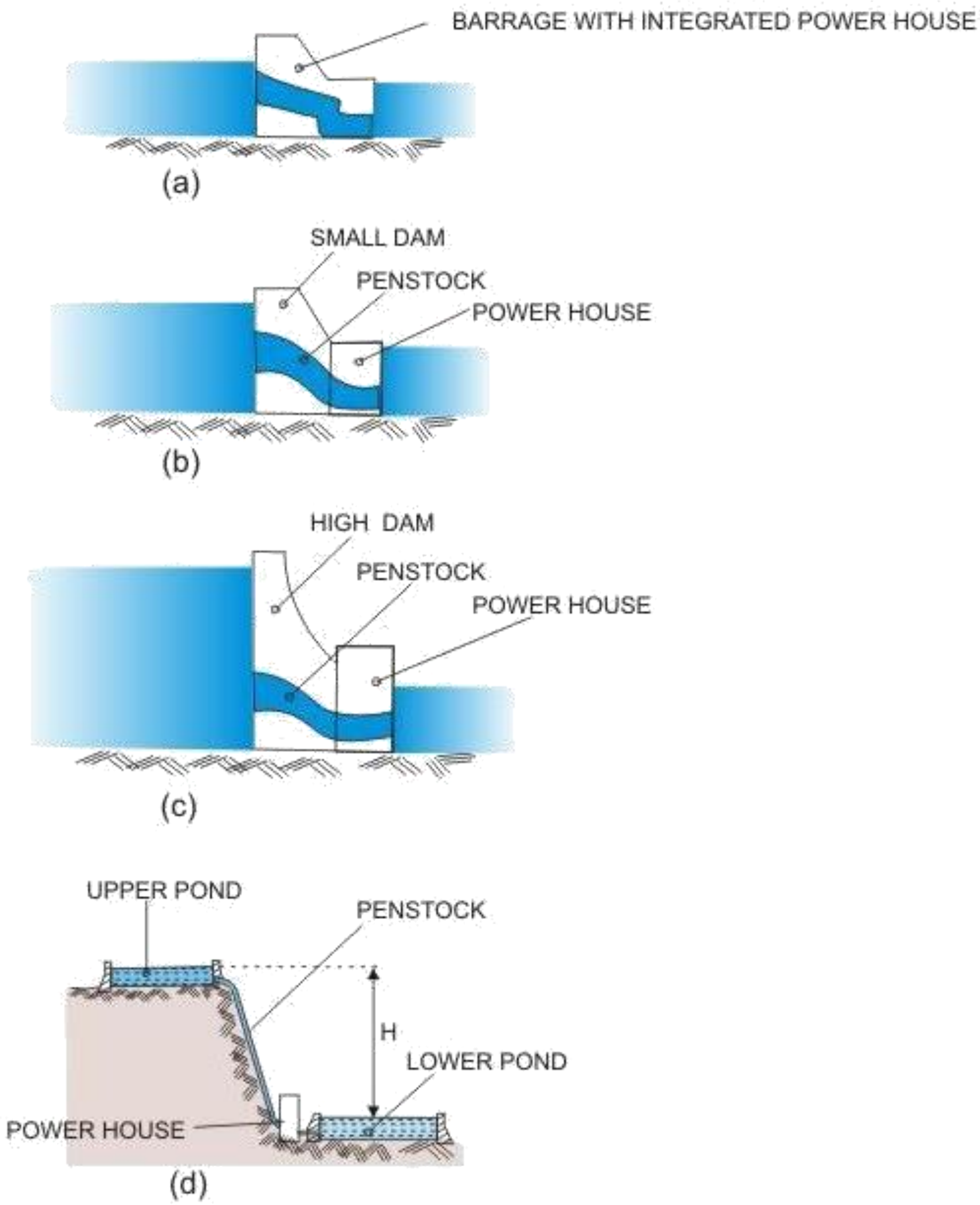


FIGURE 6. Types of hydroelectric schemes

(a) Run-of-river without pondage(little or no storage)

(b) Run-of-river with pondage (storage suitable to balance diurnal variation in power generation)

(c) Storage schemes (reservoirs to store excess water of flood flows)

(d) Pump- storage schemes

1. Run-of-river schemes

These are hydropower plants that utilize the stream flow as it comes, without any storage being provided (Figure6a). Generally, these plants would be feasible only on such streams which have a minimum dry weather flow of such magnitude which makes it possible to generate electricity throughout the year. Since the flow would vary throughout the year, they would run during the monsoon flows and would otherwise remain shut during low flows. Of course, the economic feasibility of providing the extra units apart from the regular units have to be worked out. Further, the monsoon tailwater in rivers with flat slopes becomes higher, causing the plants to become inoperative. Run-of-river plants may also be provided with some storage (Figure6b) to take care of the variation of flow in the river as for snow-melt rivers, emerging from the glaciers of Himalayas. During off-peak hours of electricity demand, as in the night, some of the units

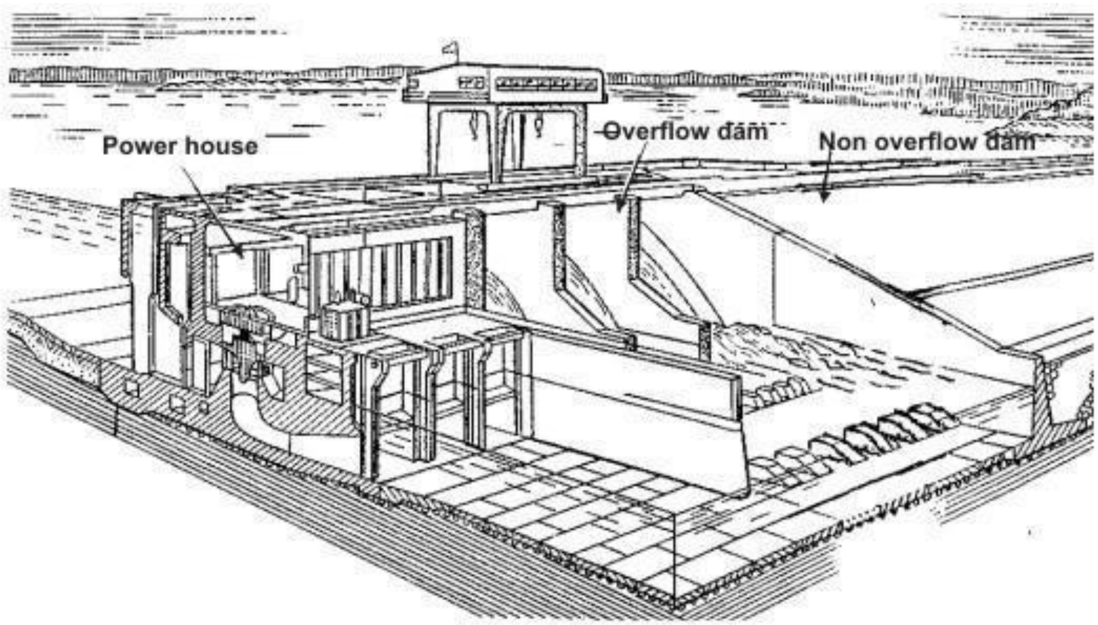


FIGURE 7. A typical run of-river hydroelectric station using a dam and an in-stream power house

may be closed and the water conserved in the storage space, which is again released during peak hours for power generation. A schematic cross sectional view of a typical run-of-river scheme is shown in Figure 7.

2. Storage schemes

Hydropower plants with storage are supplied with water from large storage reservoir (Figure 6c) that have been developed by constructing dams across rivers. Generally, the excess flow of the river during monsoon would be stored in the reservoir to be released gradually during periods of lean flow. Naturally, the assured flow for hydropower generation is more certain for the storage schemes than the run-of-river schemes. A typical schematic cross sectional view of a storage scheme power plant is shown in Figure 8.

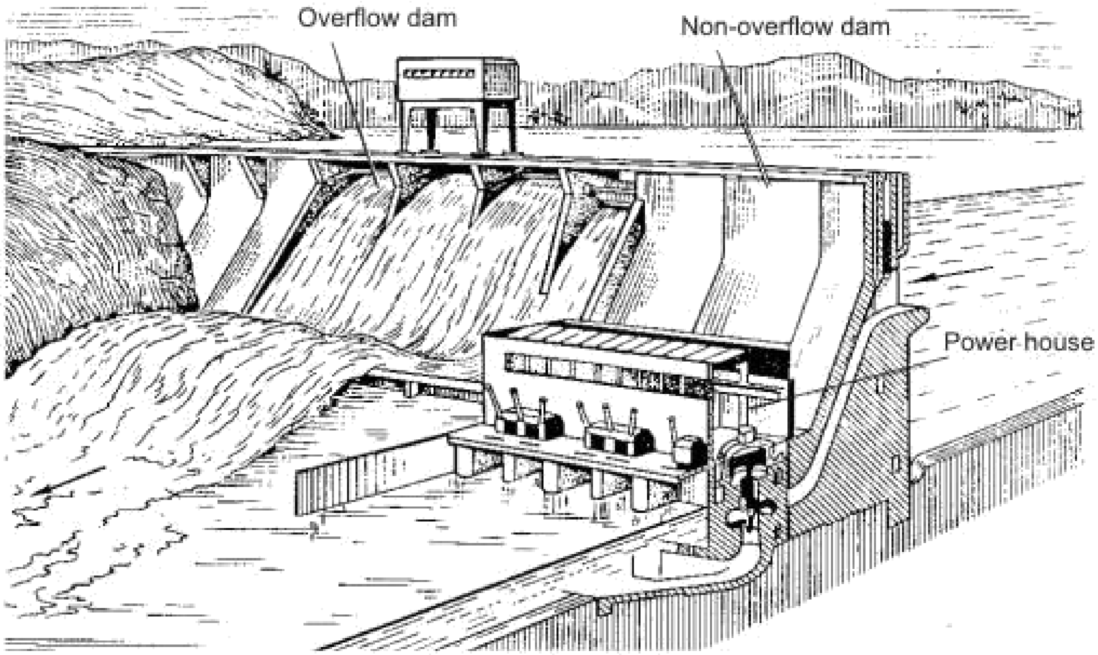


FIGURE 8. A typical storage-type hydroelectric station with a power house build at the toe of the dam

1. Pumped-Storage schemes

Hydropower schemes of the pumped-storage type are those which utilize the flow of water from a reservoir at higher potential to one at lower potential (Figure 6d). A typical schematic view of such a plant is shown in Figure 9. The upper reservoir (also called the head-water pond) and the lower reservoir (called the tail-water pond) may both be constructed by providing suitable structure across a river (Figure 10). During times of peak load, water is drawn from the head-water pond to run the reversible turbine-pump units in the turbine mode. The water released gets collected in the tail-water pond. During off-peak hours, the reversible units are supplied with the excess electricity available in the power grid which then pumps part of the water of the tail-water pond back into the head-water reservoir. The excess electricity in the grid is usually the generation of the thermal power plants which are in continuous running mode. However, during night, since the demand of electricity becomes drastically low and the thermal power plants can not switch off or start immediately, there a large amount of excess power is available at that time.

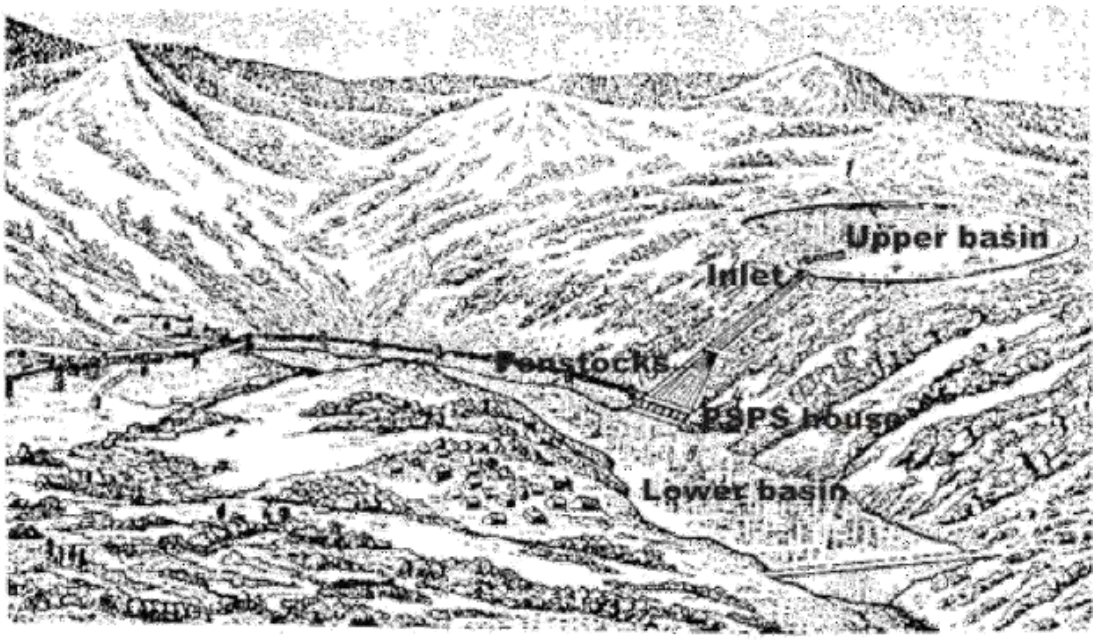


FIGURE 9. General view of pumped stroage power station

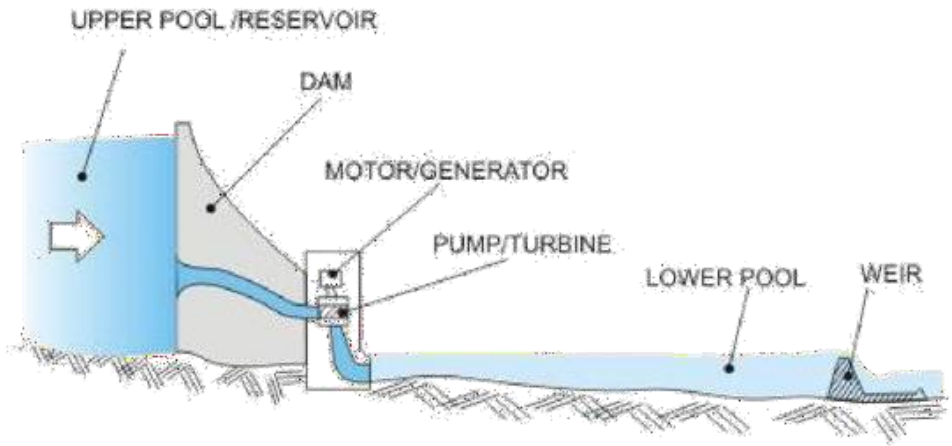


Figure 10. Pump-storage scheme development with upper and lower pools in the same river

2. Tidal power development schemes

These are hydropower plants which utilize the rise in water level of the sea due to a tide, as shown in Figure 11. During high tide, the water from the sea-side starts rising, and the turbines start generating power as the water flows into the bay. As the sea water starts falling during low tide the water from the basin flows back to the sea which can also be used to generate power provided another set of turbines in the opposite direction are installed. Turbines which generate electricity for either direction of flow may be installed to take advantage of the flows in both directions.

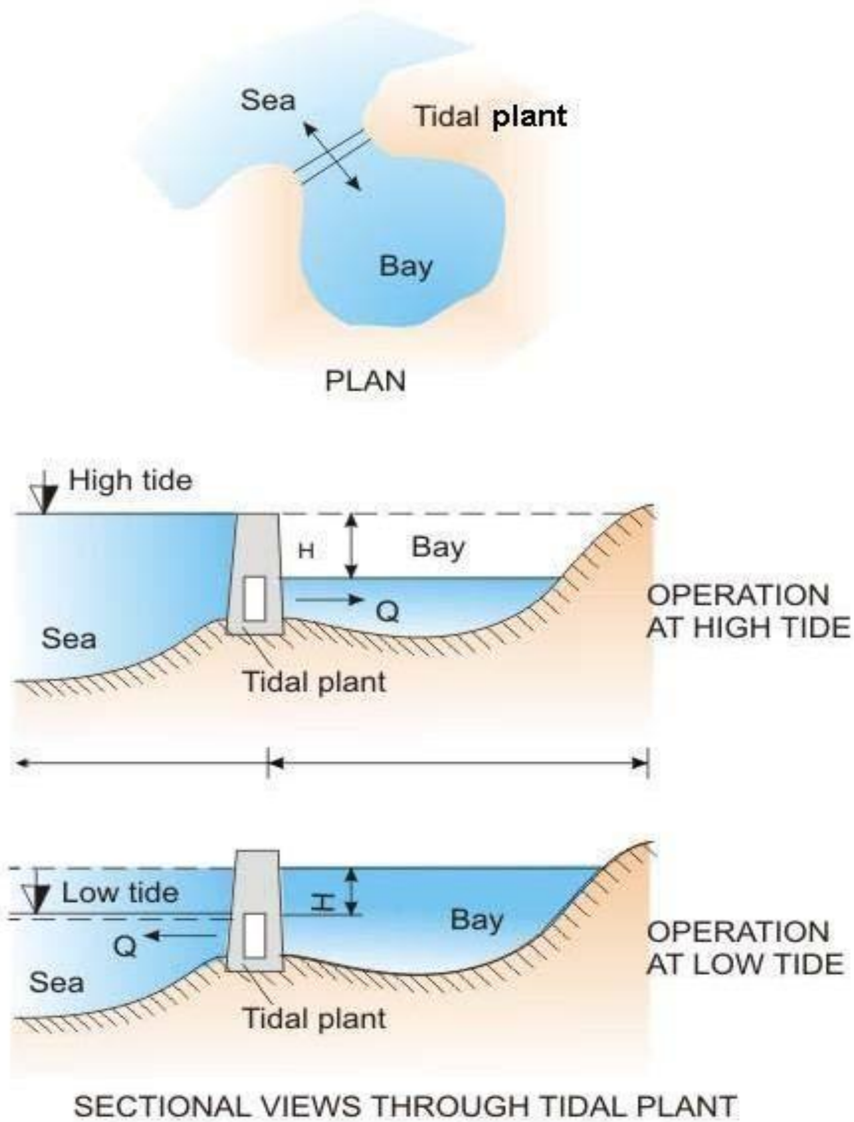


FIGURE 11. Concept of a tidal power development scheme

According to the National Oceanographic and Atmospheric Administration, USA, the potential energy of tides (often referred to as Blue Oil) is estimated at $3*10^6$ MW, of which one-third is dissipated in shallow seas. This implies that the exploitable energy available on sea coasts is of the order of 10^6 MW. Power can be generated where sufficiently large tides are available. According to experts it may be techno-economically possible to eventually develop 170,000MW at 30 sites worldwide. Globally, so far around 265 MW has been developed, although around 120,000MW are in the planning stage.

Hydroelectric power plants are also sometimes classified according to the head of water causing the turbines to rotate. The Bureau of Indian Standards code IS: 4410(Part10)- 1998 “Glossary of terms relating to river valley projects: Hydroelectric power station including water conductor system,” the following types of power plants may be defined:

1. **Low head power plant:** A power station that is operating under heads less than 30m (Figure12).

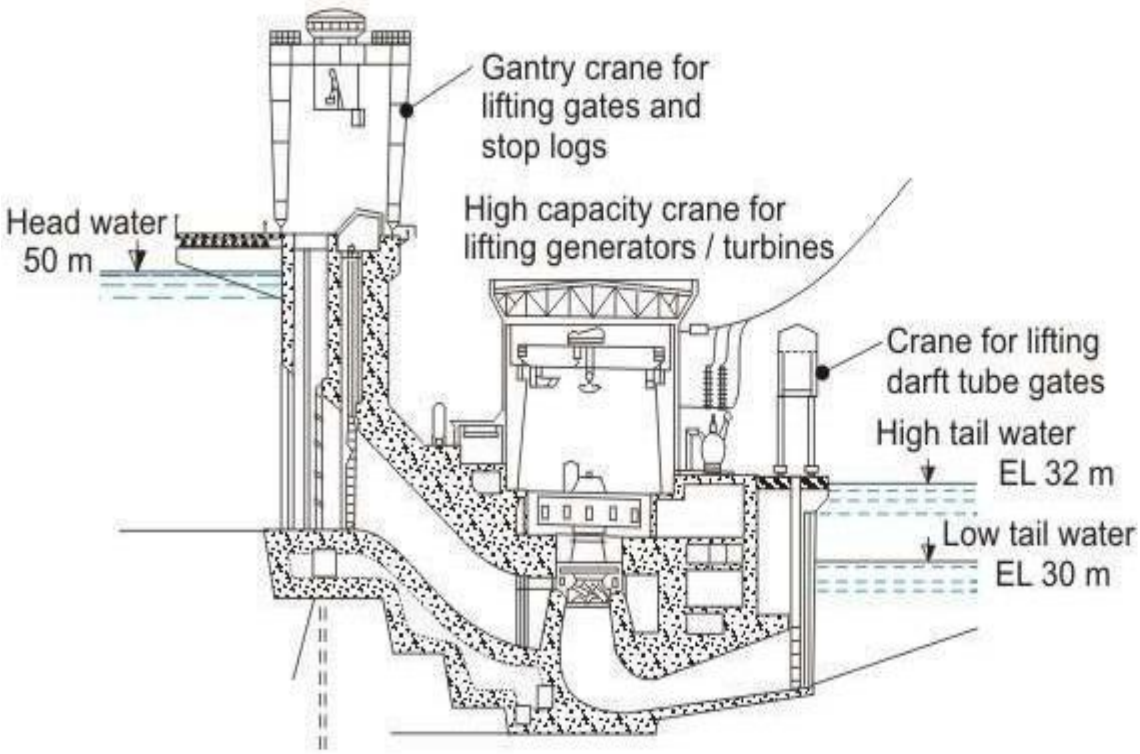


FIGURE 12. Sectional view of a typical low head hydro power station

2. **Medium head power plant:** A power station operating under heads from 30m to 300m. Of course, the limits are not exactly defined and sometimes the upper limit for medium head power station may be taken as 200 to 250m. (Figure 13)

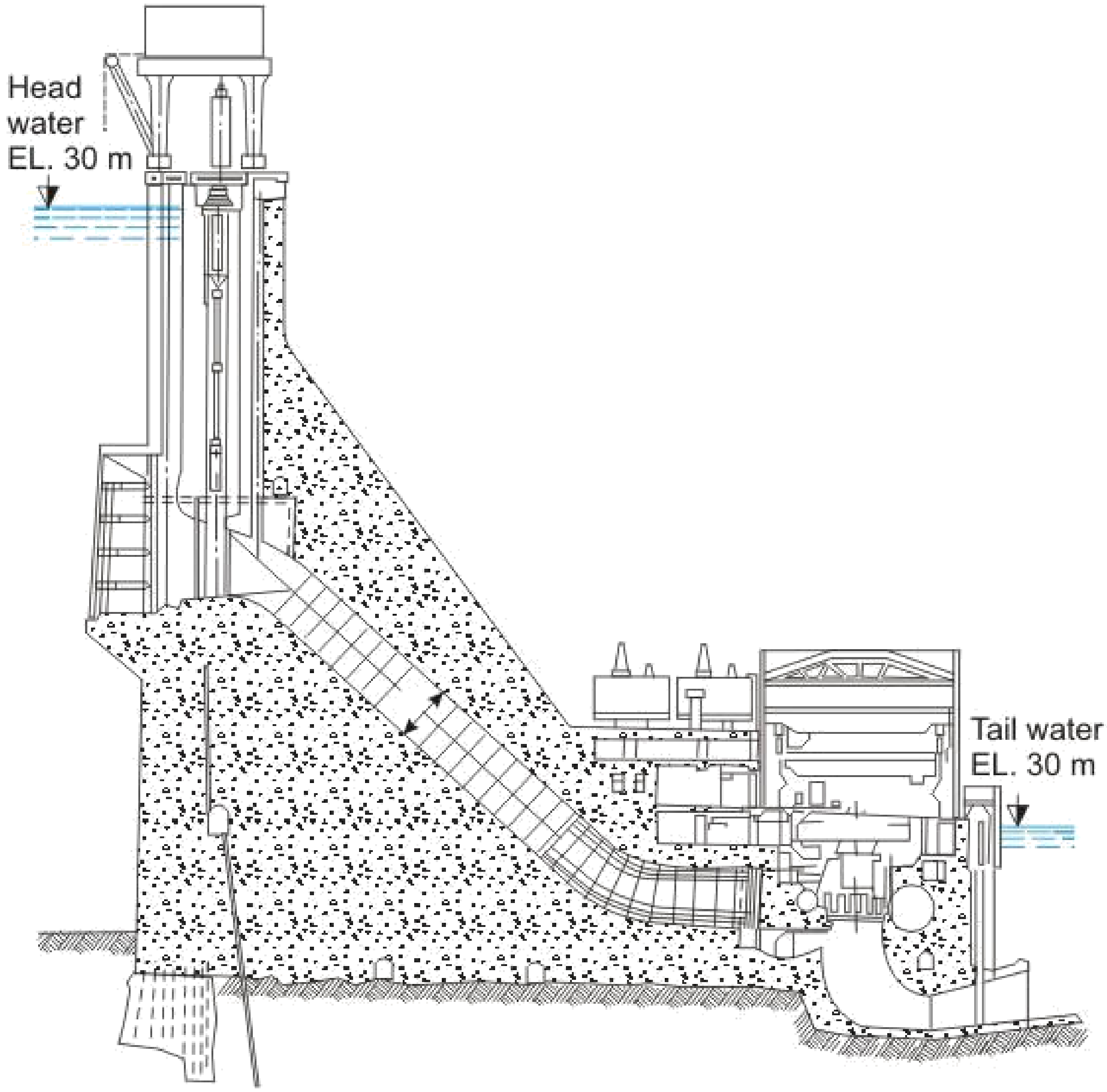


FIGURE 13. Sectional view of a typical medium head hydropower station.

1. **High head power station:** A power station operating under heads above about 300m. A head of 200m/250m is considered as the limit between medium and high head power stations. (Figure 14).

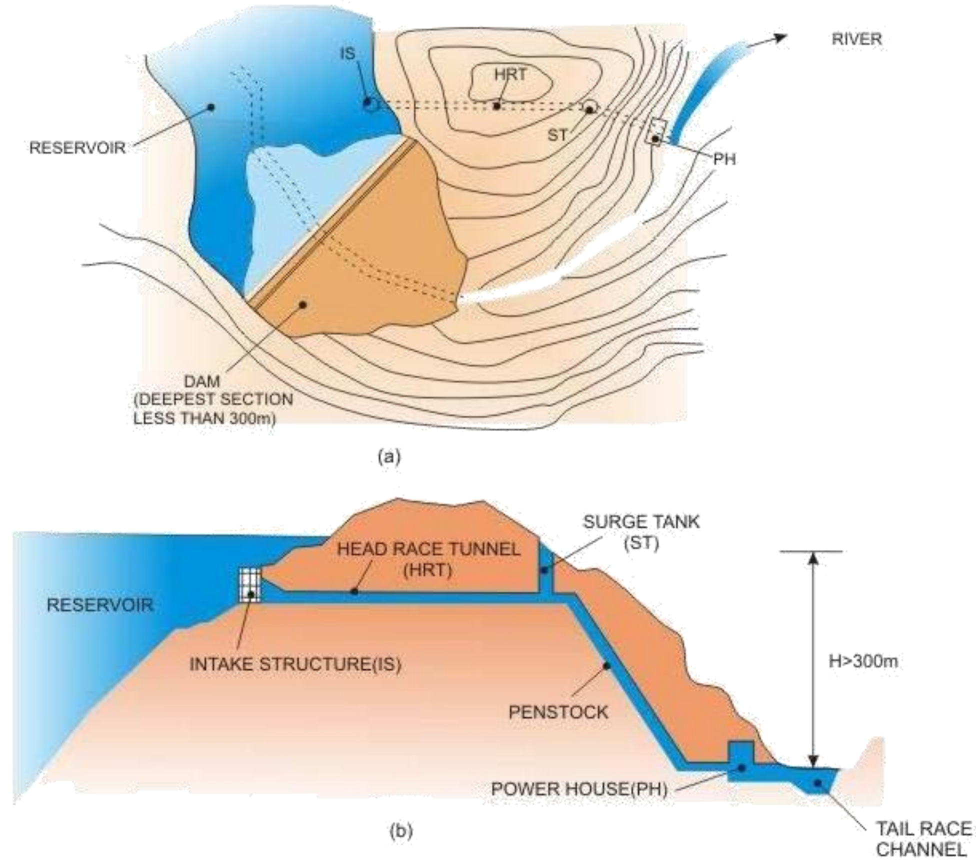


FIGURE 14. (a) Layout of the project showing an embankment dam creating a reservoir with a high head
(b) Sectional view through the water conducting system for hydropower

IS: 4410(part10)-1998 also classifies hydropower plants according to their operating functions as follows:

1. Base load power plant: A power station operating continuously at a constant or nearly constant power and which operates at relating high load factors. It caters to power demand at base of the load curve.
2. Peak load power plant: A power station that is primarily designed for the purpose of operating to supply the peak load of a power system. This type of power station is also, therefore, termed as „Peaking station".

According to Mosonyi (1991), hydropower plants can also be classified according to plant capacity as follows:

1. Midget plant: up to 100KW
2. Low-capacity plant up to 1,000KW
3. Medium capacity plant up to 10,000KW
4. High capacity plant > 10,000KW

In India, Micro-hydel plants with capacity less than 5000KW are being encouraged to tap small streams and canal falls. Of the larger hydropower stations in India, the following are at the top of the list:

Sl. No	Project	Number of units × Capacity	Total capacity (MW)
1.	Bhakra	5*108(MW)+5*132(MW)	1200
2.	Dehar	6*165(MW)	990
3.	Koyna	4*165(MW)+4*75(MW)+4*80(MW)	880
4.	Nagarjuna sagar	1*110(MW)+7*100(MW)	891
5.	Srisailam	7*110(MW)	770
6.	Sharavathy	10*89.1(MW)	891
7.	Kalinadi	6*135(MW)	810
8.	Idukki	6*130(MW)	780

A map showing the major hydroelectric power station in India as given in CBIP (1987) is shown in Figure 15.

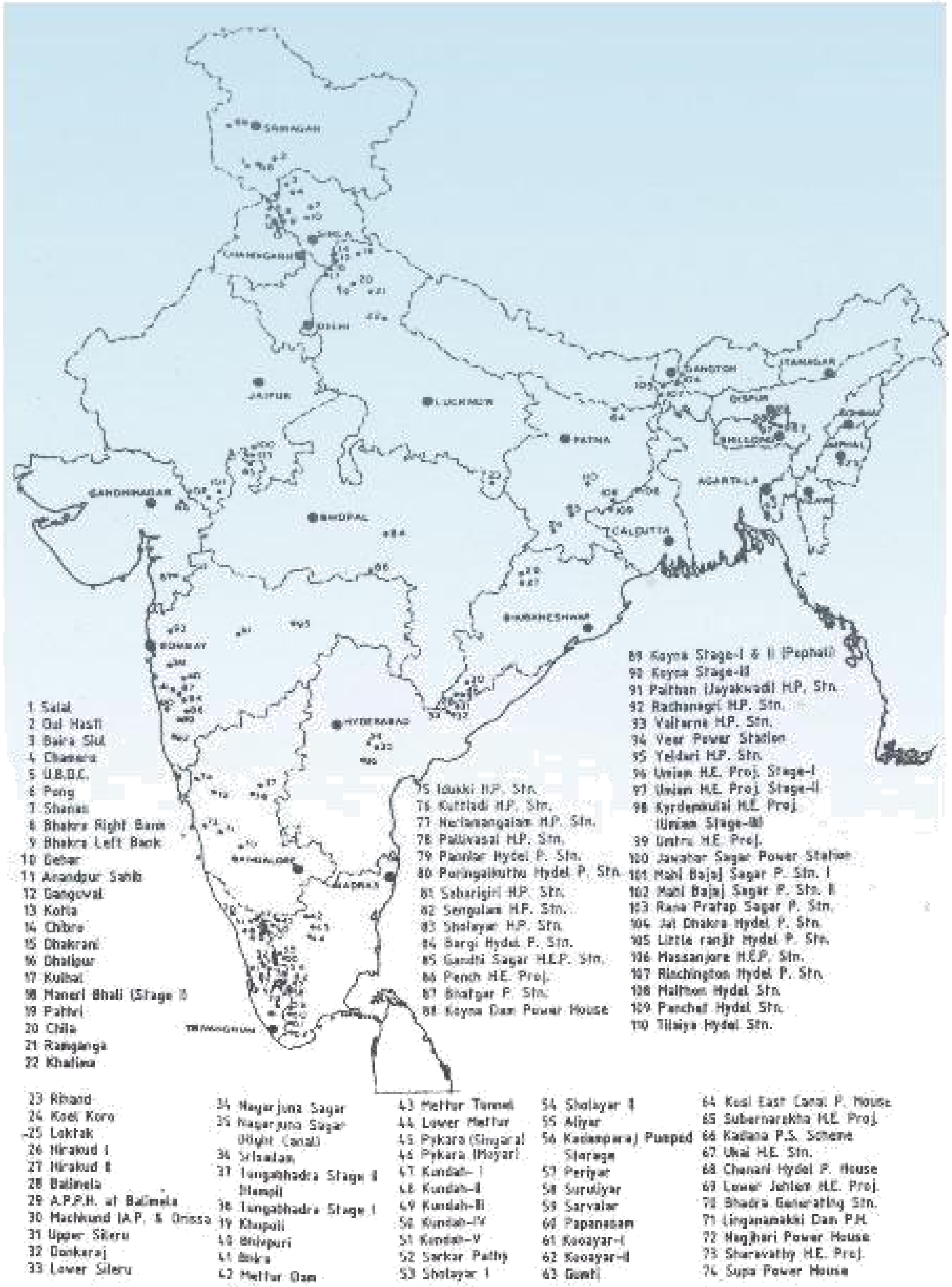


FIGURE 15. Hydro electric power stations in India

World-wide, there are a few hydropower plants with capacity greater than 10,000MW. These are given in the following list.

Sl.no	project	Country	Total Capacity
1.	Turukhnok	C.I.S.	20,000MW
2.	Three Gorges	China	13,400MW
3.	Itaipu	Brazil	12,000MW
4.	Grand Coulee	U.S.A	10,830MW
5.	Guri	Venezuela	10,300MW

On the other hand, China has over 88,000 small hydropower stations with a total installed capacity of 6929MW generating one-third of all the electricity consumed in rural areas. Hence, emphasis on micro-hydel development cannot be overlooked and similar developments can be done in the hilly regions of India where streams and small rivers may be tapped to provide power locally to the neighbouring rural community.

Though there has been a study growth of hydropower development in India over the years (Figure16), the proportional contribution of hydropower to the country‟s total energy production is rather small (Figure17).

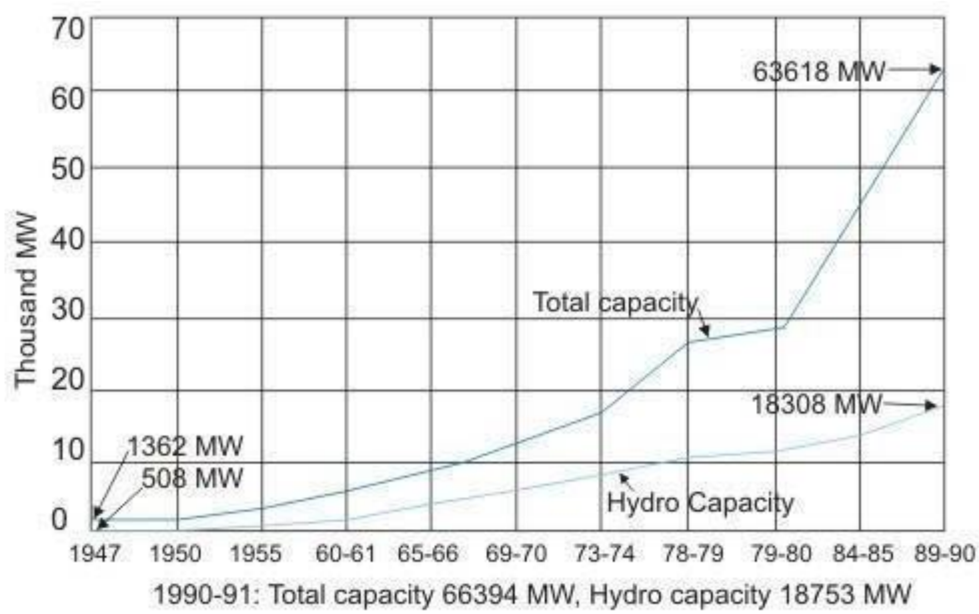


FIGURE 16. Total energy production and hydro power contribution for india

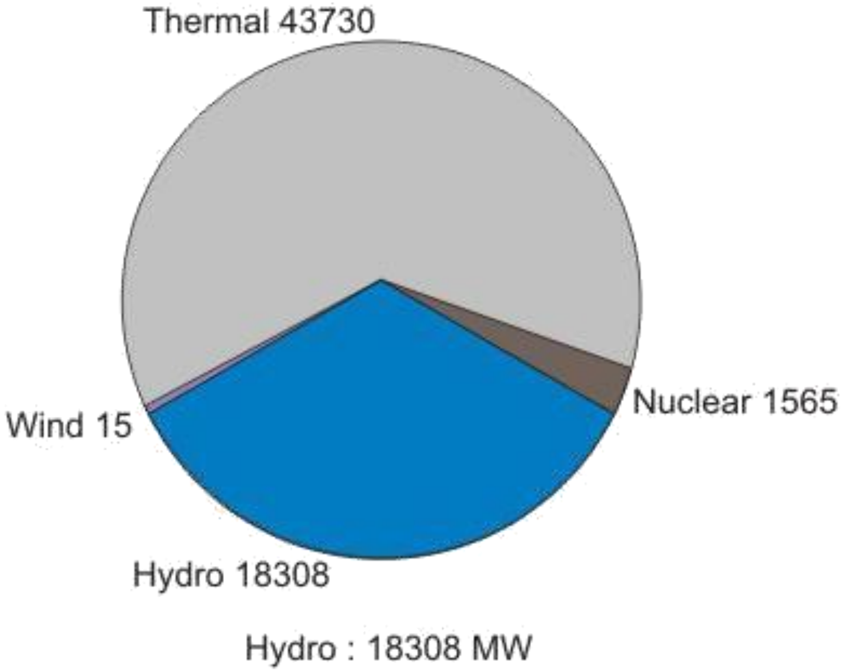


Figure 17. Installed capacity of india's energy producing units. This shows a hydro thermal mixratio of 29:71 approximately

In comparison, there are countries where hydropower production is the major source of electricity as given in the following table:

Rank	Country	Hydro production	Share of electricity
1.	Norway	83.5	99.4
2.	Zambia	8.8	98.9
3.	Zaire	4.3	98.6
4.	Ghana	4.7	98.5
5.	Mozambique	13.6	97.1
6.	Brazil	127.0	92.7
7.	Zimbabwe	4.0	88.9
8.	Sri Lanka	1.5	88.6
9.	New Zealand	16.3	74.1
10.	Nepal	0.2	73.6
11.	Switzerland	33.6	69.7
12.	Austria	29.1	69.3
13.	Canada	251.0	68.4
14.	Colombia	13.8	67.0
15.	North Korea	22.5	64.3
16.	Sweden	61.8	64.1

Ideally, for India, the Hydro: Thermal mix of around 40:60 has been considered to be the optimum.

Hydropower plant scheme layout

Typical components of a hydroelectric plant consist of the following:

1. Structure for water storage and/or diversion, like a dam or a barrage.
2. A head-race water conveying system like a conduit (penstock) or an open channel to transport water from the reservoir or head-water pool up to the turbines.
3. Turbines, coupled to generators
4. A tail race flow discharging conduit of open channel that conveys the water out of the turbine up to the river.

Although the above components are common for all hydropower development schemes the general arrangement for high and medium head power houses are more or less similar. The low head power plants, which are usually of run-of-power type schemes, have a slightly different arrangement as mentioned in the paragraphs below.

High and medium head development

Usually, there could be two types of power scheme layout:

- Concentrated fall schemes
- Diversion schemes

In the concentrated fall type projects, the powerhouse would be built at the toe of a concrete gravity dam, shown as a schematic view in Figure 7 and sectional view in Figure 12. This type of project development is suitable for medium head projects since a high head project would require an enormous concrete gravity dam, which is generally not adopted. A medium or high head project with an earthfill or rockfill dam may have an isolated or off-stream power house as shown in Figure 13. Here, the water is conveyed to the turbines via penstocks laid under, or by-passing, the dam. Spillways are provided separately to take care of floods. A distinction of such projects is that it consists of a long system of water conduits. Surge tanks are sometimes provided at the end of the conduits to relieve them of water hammer, which is the very high pressure developed by causing the stoppage of flow too suddenly at the turbine end.

In the diversion type of layout, the diversion could be using a canal and a penstock (Figure 18) or a tunnel and a penstock (Figure 19). The former is usually called the Open-Flow Diversion System and the latter Pressure Diversion System.

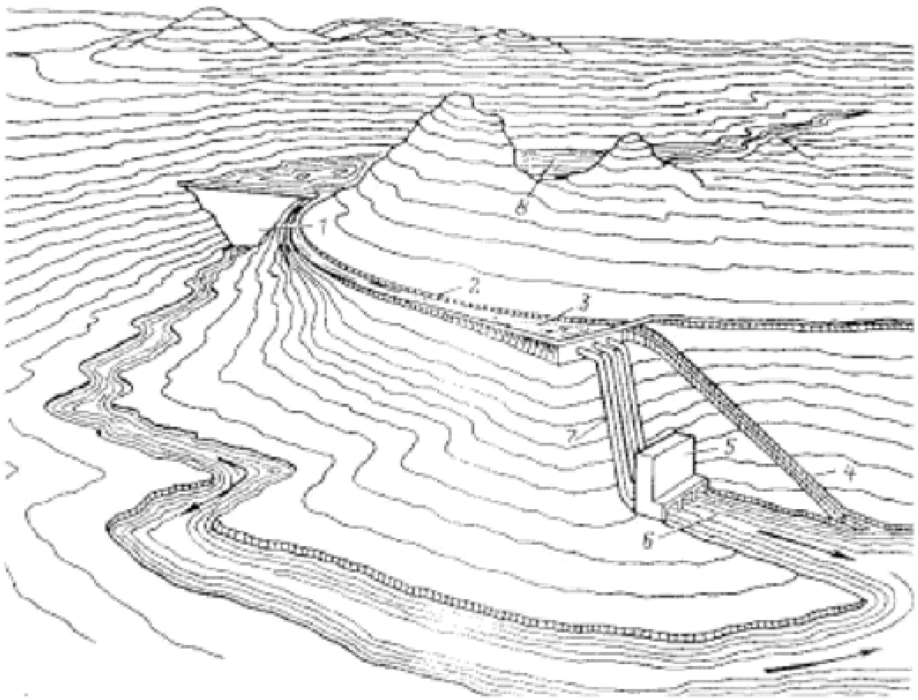


FIGURE 18 HYDROELECTRIC PROJECT BASED ON OPEN FLOW DIVERSION SCHEME
1-DAM .2-INTAKE DIVERSION CONDUIT. 3-HEAD POND. 4- SPILLWAY . 5- POWER HOUSE.
6- TAILVACE. 7-PENSTOCKS. 8-RESERWAIR

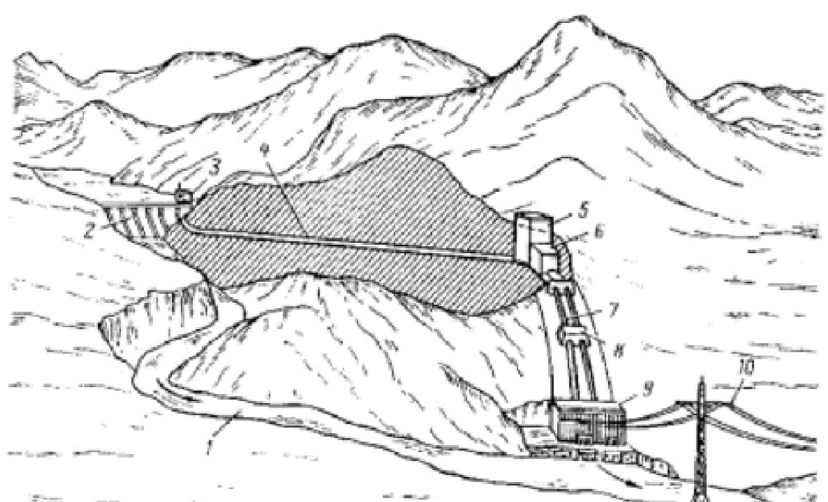


FIGURE 19. HYDROELECTRIC PROJECT USING A PRESSURE DIVERSION SYSTEM
1-WATERCOURSE. 2-DAM. 3- INTAKE STRUCTURES. 4-DIVERSION TUNNEL. 5- SURGE TANK. 6-PENSTOCK FORK HOUSE
7-PENSTOCKS.8- PENSTOCKS SUPPORT.9- POWER HOUSE. 10- POWER LINE

In fact, the combination of open channel and pressure conduit and penstock may be done in a variety of ways

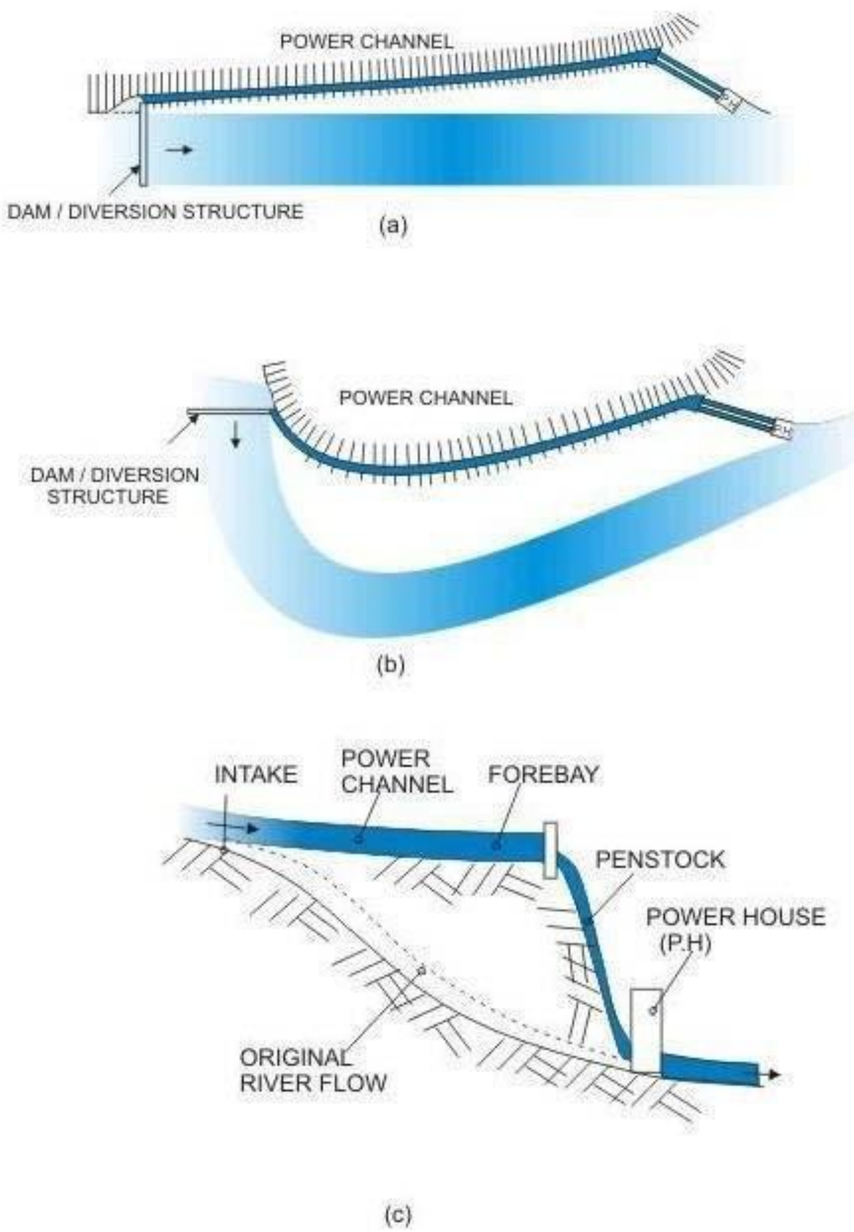
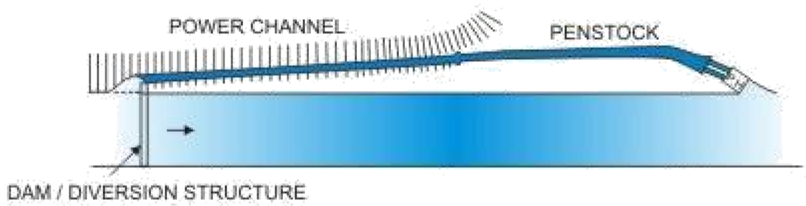
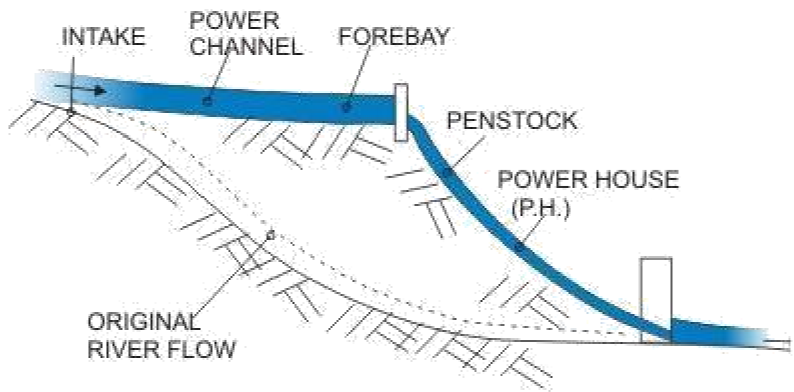


FIGURE 20. DIVERSION HYDRO POWER PROJECT BASED ON OPEN CHANNEL AND PRESSURE FLOW SYSTEMS
(a) LONG CANAL AND SHORT SURFACE PENSTOCK ALONG STRAIGHT RIVER REACH
(b) SAME AS (a) BUT IN A CURVED RIVER REACH
(c) SECTIONAL VIEW ALONG WATER CONDUCTING SYSTEM FOR (a) AND (b)

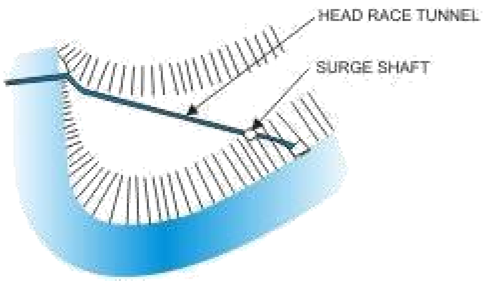


(d)

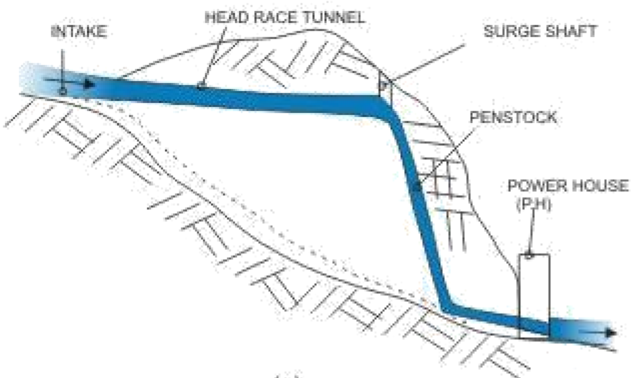


(e)

FIGURE 20. DIVERSION HYDRO POWER PROJECT BASED ON OPEN CHANNEL AND PRESSURE FLOW SYSTEMS
(d) SHORT CANAL AND LONG SURFACE PENSTOCK
(e) SECTIONAL VIEW FOR (d)



(f)



(g)

FIGURE 20. DIVERSION HYDRO POWER PROJECT BASED ON OPEN CHANNEL AND PRESSURE FLOW SYSTEMS

(f) HEAD RACE TUNNEL AND PENSTOCK IN A CURVED RIVER REACH

(g) SECTIONAL VIEW FOR (f)

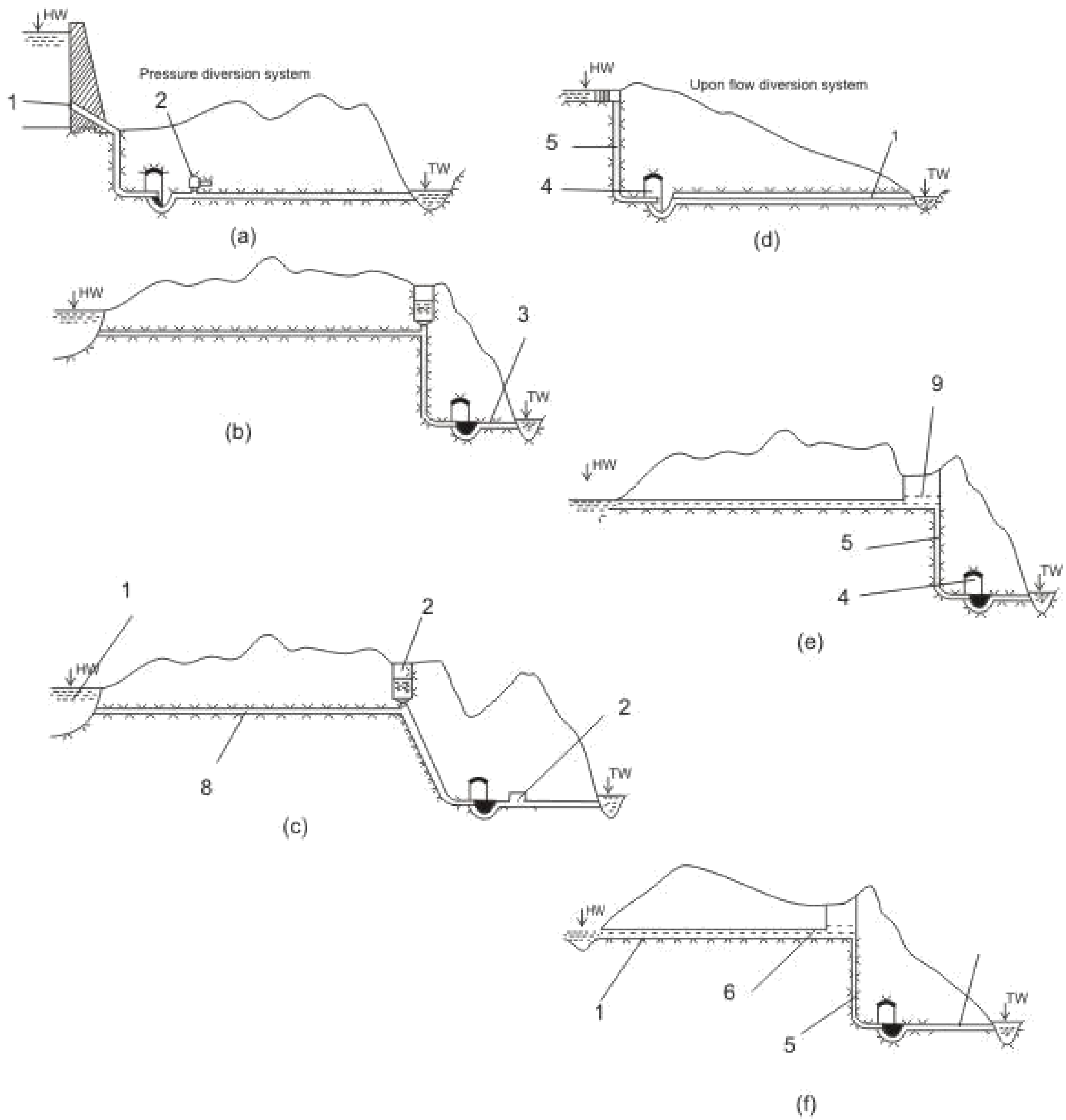


FIGURE 21. Underground project with
(a),(b) and (c) pressure diversion system, and (d),(e) and (f) open flow diversion system;
1-intake structure; 2- surge tank; 3-tailrace pressure tunnel; 4- power house; 5-penstock
6-intake open flow tunnel; 7-tailrace open flow tunnel ;8-intake pressure tunnel; 9--head pond

Low head development

Here too, two types of layouts may be possible:

- In-stream scheme
- Diversion scheme

In the in-stream type of project, the powerhouse would be built as a part of the diversion structure, as shown in Figure 2(a) or a general detailed view in Figure 6. A typical layout of such a power house and its cross section is shown in Figure 22.

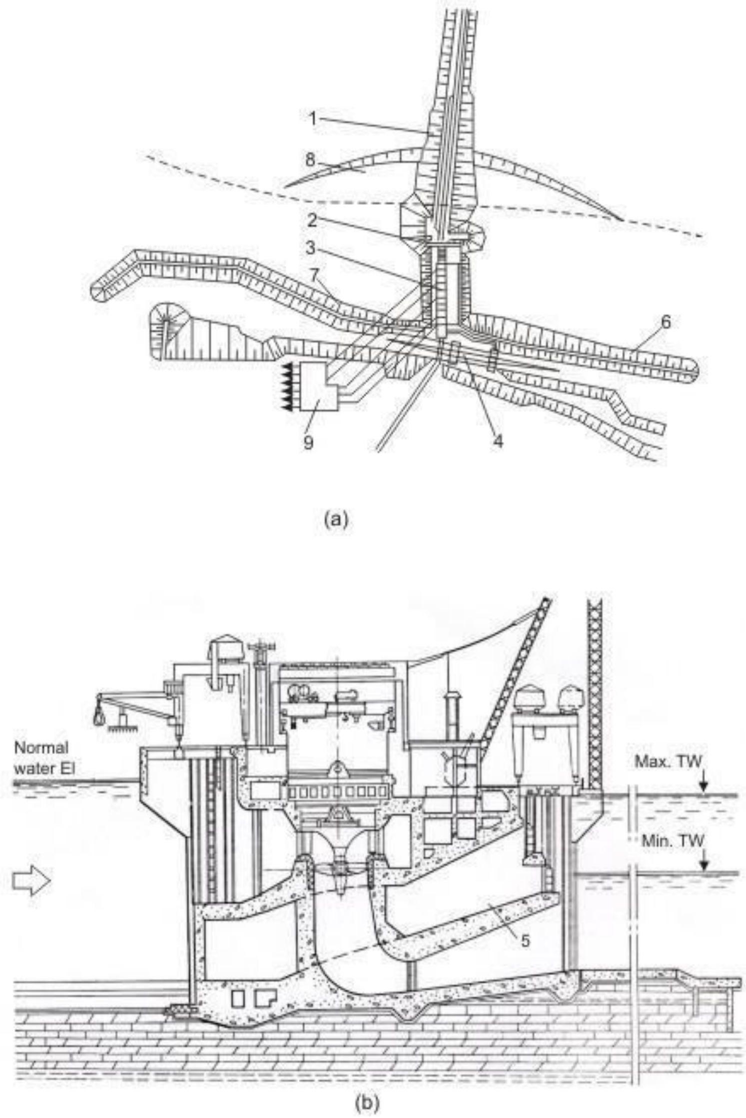


FIGURE 22. A TYPICAL LOW-HEAD IN STREAM POWER HOUSE
(a) plane ;(b) sectional elevation of the power house; 1-earth dam.2-over flow dam
3- power house; 4-lock;5-spillways in power house; 6- navigable canal dike downstream of dam;
7-output dike; 8- left bank clearing; 9- electrical switch yard

In the diversion type of scheme, there has to be a diversion structure as well as a diversion canal, as shown in Figure 2(b). The power house may be located at some convenient point of the canal, that is, at its upstream end, middle, or at the downstream end. The location of the power house depends upon conditions such as hydrological, topographical, geological, environmental economic conditions. The ground water table has to be taken into account.

Position of power houses

As might have been noticed from the layouts, there could be a variety of position for the power house with respect to natural ground level.

IS: 4410(Part10)-1988 differentiates between the following types of power stations, which may be constructed as per site conditions:

1. Surface power station or over ground power station: A power station which is constructed over the ground with necessary open excavation for foundations. Typical examples may be seen from Figs. 7, 11 or 12.
2. Underground power station: A power station located in a cavity in the ground with no part of the structure exposed to outside. A typical example of this type is shown in Figure 23.

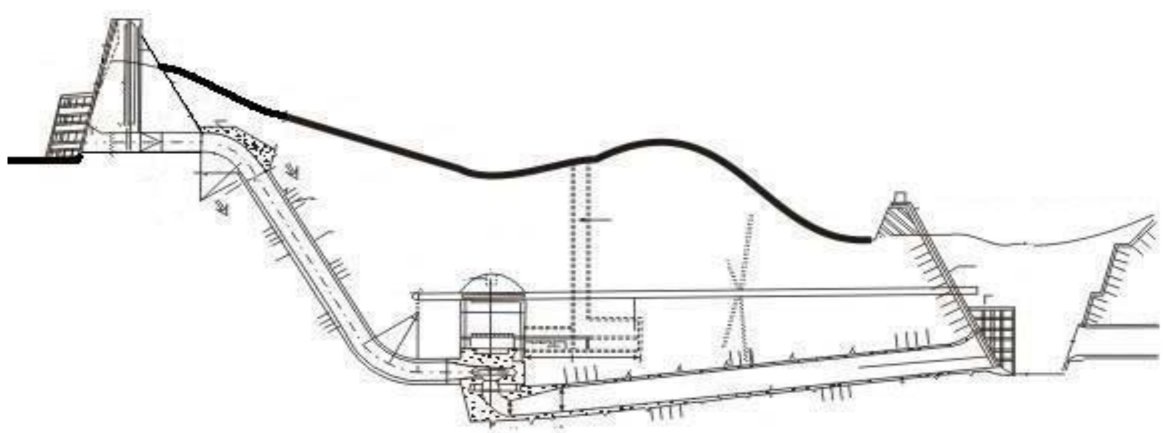


FIGURE 23 Underground power house of Sardar Sarovar Dam project

3. Semi-underground power station: A power station located partly below the ground level and followed by a tail race.

Electrical terms associated with hydropower engineering

Electrical power generated or consumed by any source is usually measured in units of Kilowatt-hour (kWh). The power generated by hydropower plants are normally connected to the national power grid from which the various withdrawals are made at different places, for different purposes. The national power grid also obtains power generated by the non- hydropower generating units like thermal, nuclear, etc. The power consumed at various points from the grid is usually termed as electrical load expressed in Kilo-Watt (KW) or Mega- Watt (MW). The load of a city varies throughout the day and at certain time reaches the highest value (usually in the evening for most Indian cities), called the Peak load or Peak demand. The load for a day at a point of the national power grid may be plotted with time to represent what is known as Daily Load Curve. Some other terms associated with hydropower engineering are as follows:

Load factor

This is the ratio of average load over a certain time period and the maximum load during that time. The period of time could be a day, a week, a month or a year. For example, the daily load factor is the ratio of the average load may be obtained by calculating the total energy consumed during 24 hours (finding the area below the load vs. time graph) and then divided by 24. Load factor is usually expressed as a percentage

Installed capacity

For a hydro electric plant, this is the total capacity of all the generating units installed in the power station. However all the units may not run together for all the time.

Capacity factor

This is the ratio of the average output of the hydroelectric plant for a given period of time to the plant installed capacity. The average output of a plant may be obtained for any time period, like a day, a week, a month or a year. The daily average output may be obtained by calculating the total energy produced during 24hours divided by 24. For a hydroelectric plant, the capacity factor normally varies between 0.25 and 0.75.

Utilization factor

Throughout the day or any given time period, a hydroelectric plant power production goes on varying, depending upon the demand in the power grid and the power necessary to be produced to balance it. The maximum production during the time divided by the installed capacity gives the utilization factor for the plant during that time. The value of utilization factor usually varies from 0.4 to 0.9 for a hydroelectric plant depending upon the plant installed capacity, load factor and storage.

Firm (primary) power

This is the amount of power that is the minimum produced by a hydro-power plant during a certain period of time. It depends upon whether storage is available or not for the plant since a plant without storage like run-of-river plants would produce power as per the minimum stream flow. For a plant with storage, the minimum power produced is likely to be more since some of the stored water would also be used for power generation when there is low flow in the river.

Secondary Power

This is the power produced by a hydropower plant over and above the firm power.

Intake or Control Gates

These are the gates built on the inside of the dam. The water from reservoir is released and controlled through these gates. These are called inlet gates because water enters the power generation unit through these gates. When the control gates are opened the water flows due to gravity through the penstock and towards the turbines. The water flowing through the gates possesses potential as well as kinetic energy

The Penstock

The penstock is the long pipe or the shaft that carries the water flowing from the reservoir towards the power generation unit, comprised of the turbines and generator. The water in the penstock possesses kinetic energy due to its motion and potential energy due to its height The total amount of power generated in the hydroelectric power plant depends on the height of the water reservoir and the amount of water flowing through the penstock. The amount of water flowing through the penstock is controlled by the control gates.

Water Turbines

Water flowing from the penstock is allowed to enter the power generation unit, which houses the turbine and the generator. When water falls on the blades of the turbine the kinetic and potential energy of water is converted into the rotational motion of the blades of the turbine. The rotating blades causes the shaft of the turbine to also rotate. The turbine shaft is enclosed inside the generator. In most hydroelectric power plants there is more than one power generation unit.There is large difference in height between the level of turbine and level of water in the reservoir. This difference in height, also known as the head of water, decides the total amount of power that can be generated in the hydroelectric power plant.

There are various types of water turbines such as Kaplan turbine, Francis turbine, Pelton wheels etc. The type of turbine used in the hydroelectric power plant depends on the height of the reservoir, quantity of water and the total power generation capacity.