Module
4

Hydraulic Structures for Flow Diversion and Storage

Lesson
8

Spillways and Energy Dissipators

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

4.8.0 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.

4.8.1 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.
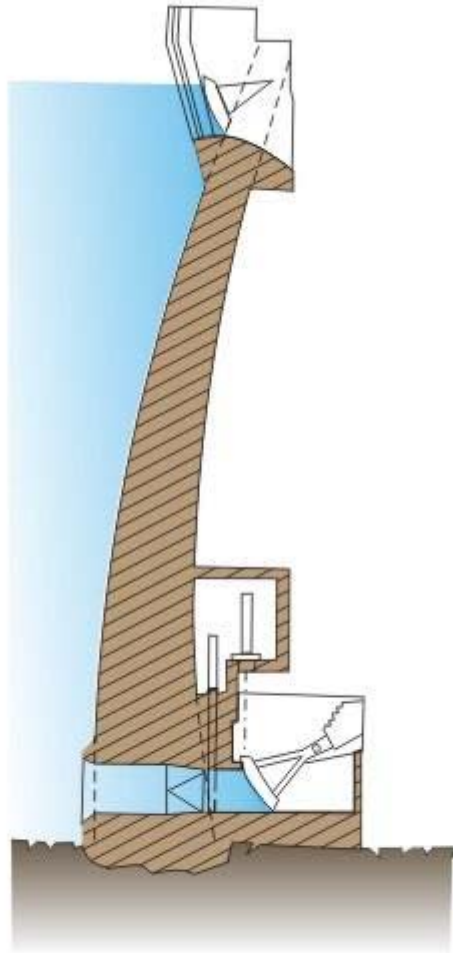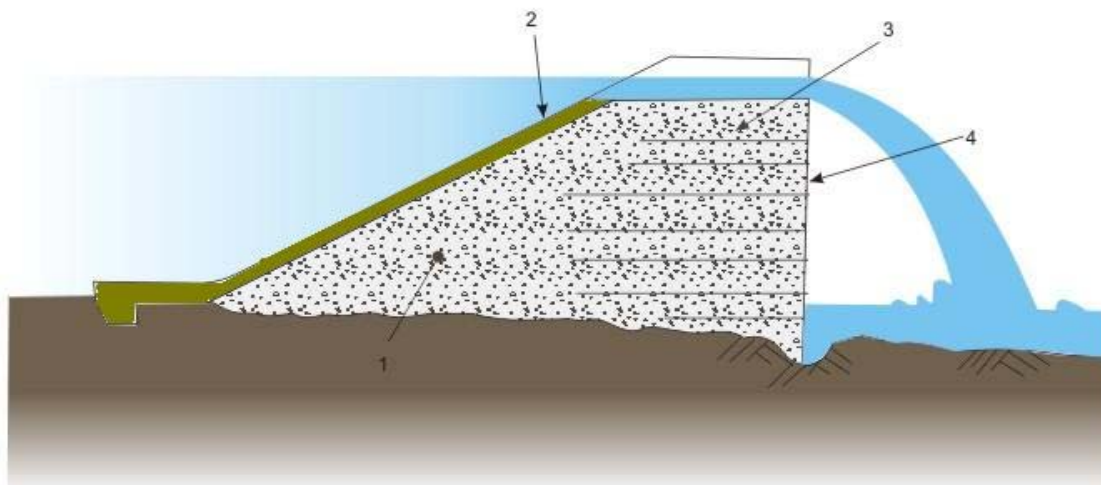
FIGURE 1. Free over fall spillway for an arch dam

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
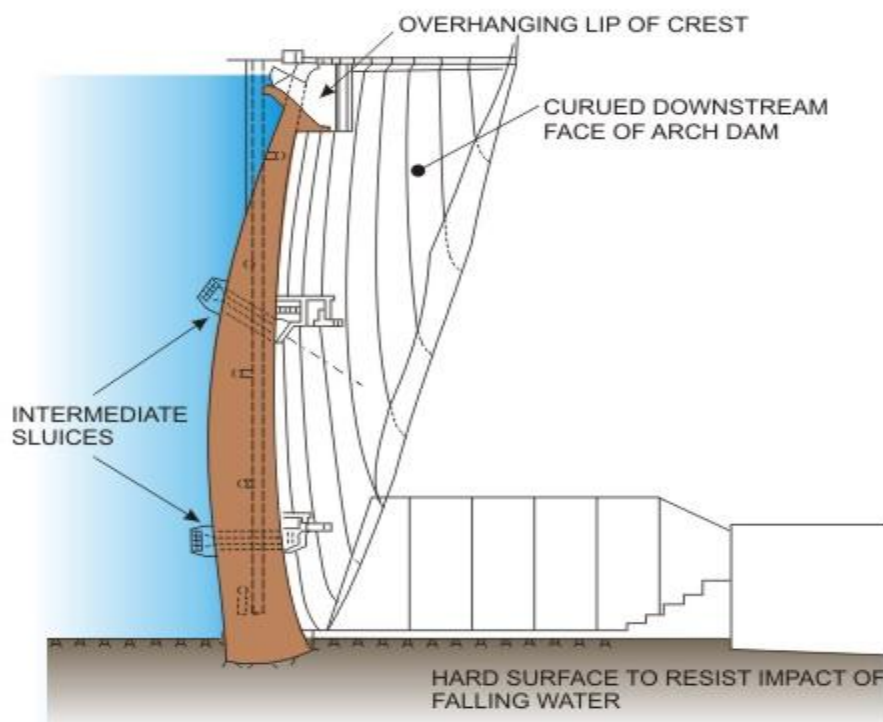

FIGURE 3. Short lip provided for overfall spilling of an arch dam

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.
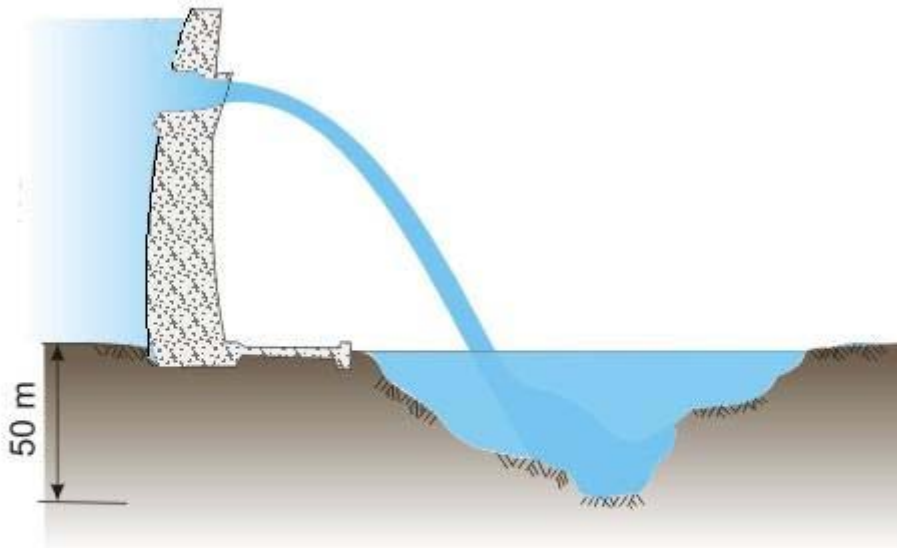
FIGURE 4. Scour below Kariba Dam Spillway, Zimbabwe

4.8.2 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.
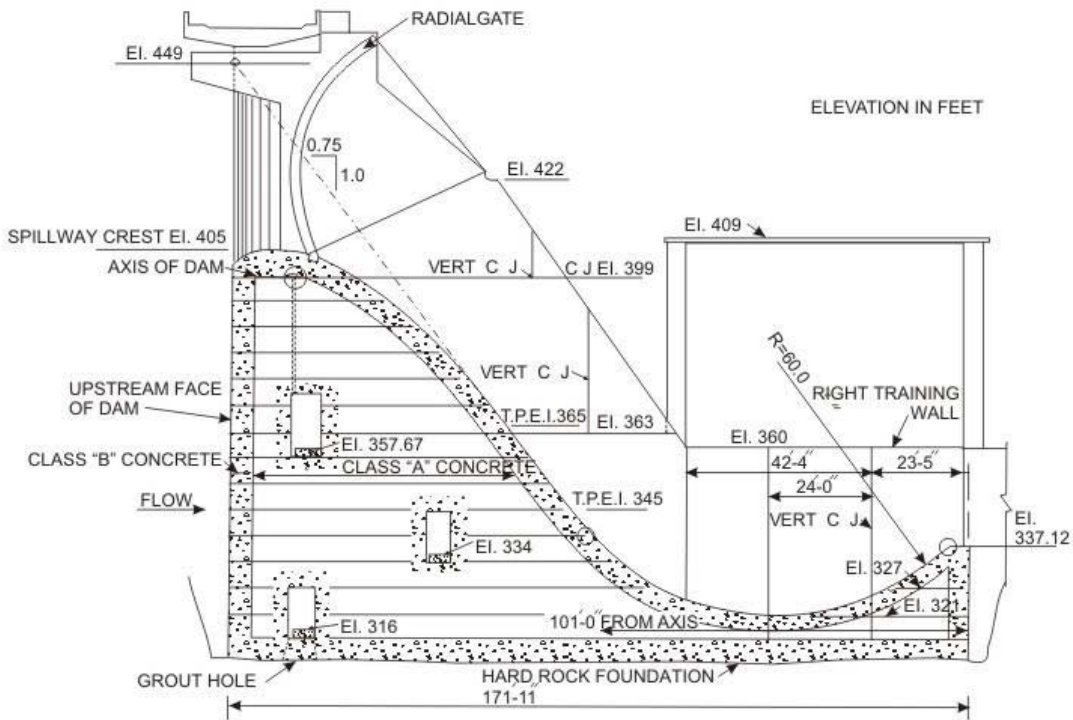

FIGURE 5. Typical overflow (ogee) spillway .Example of Panchet Dam on River Damodar
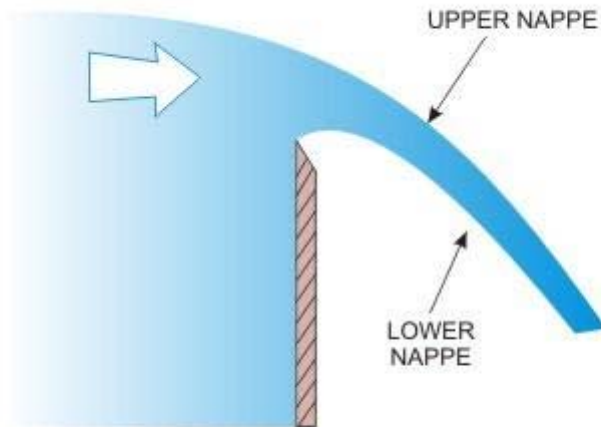
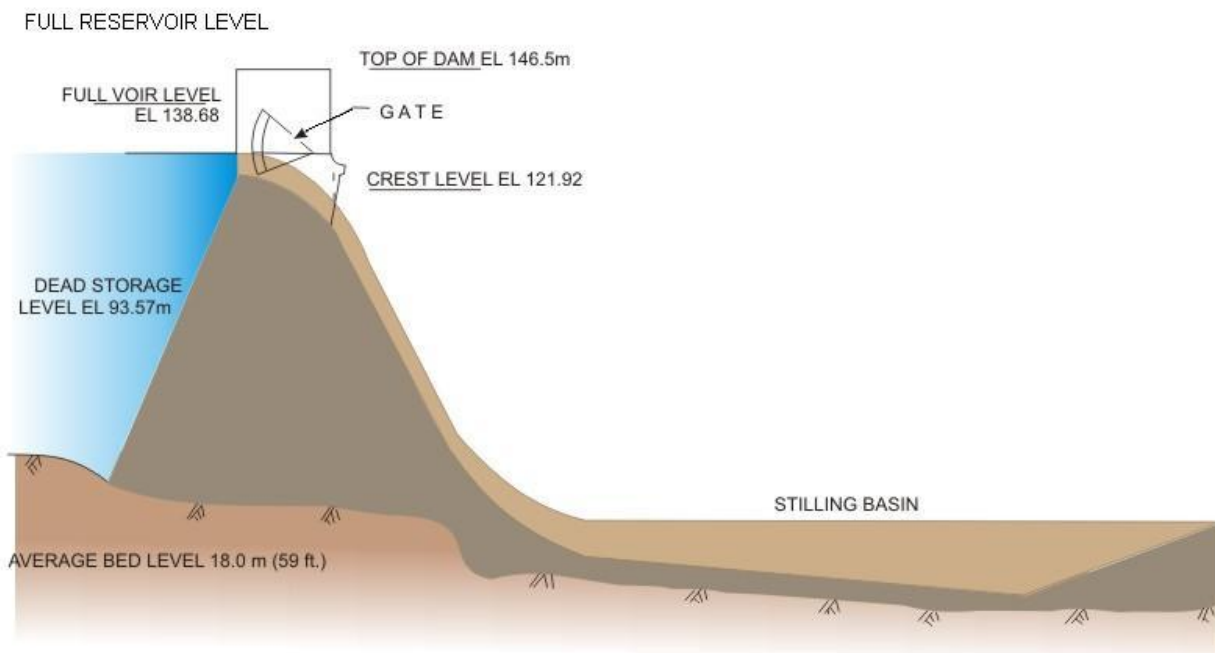FIGURE 6. Outflow from a free-falling weir , properly ventilated from below

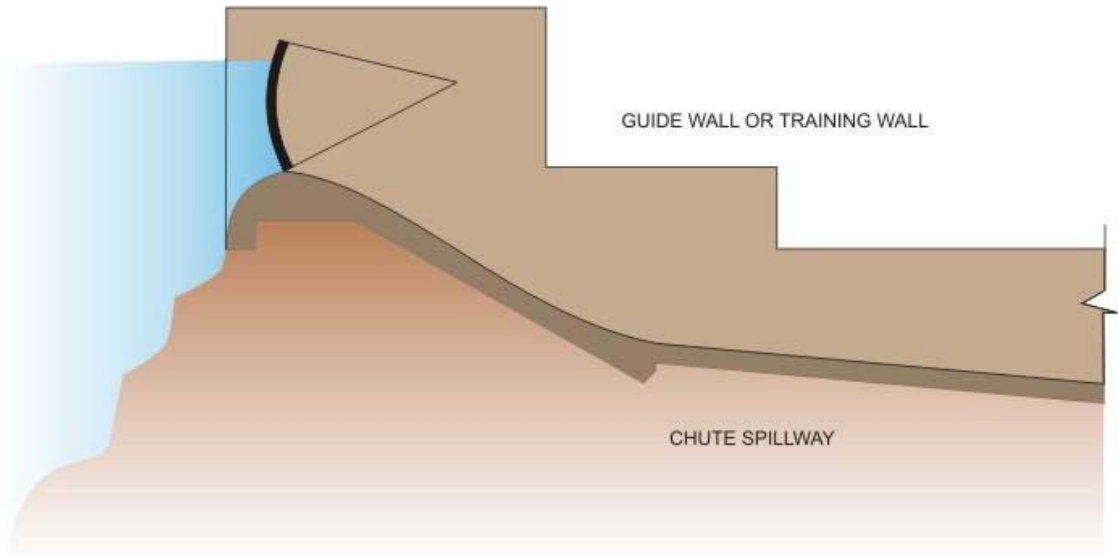FIGURE 7. Ogee spillway & apron of Sardar Sarovar Dam spillway

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

4.8.3 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".
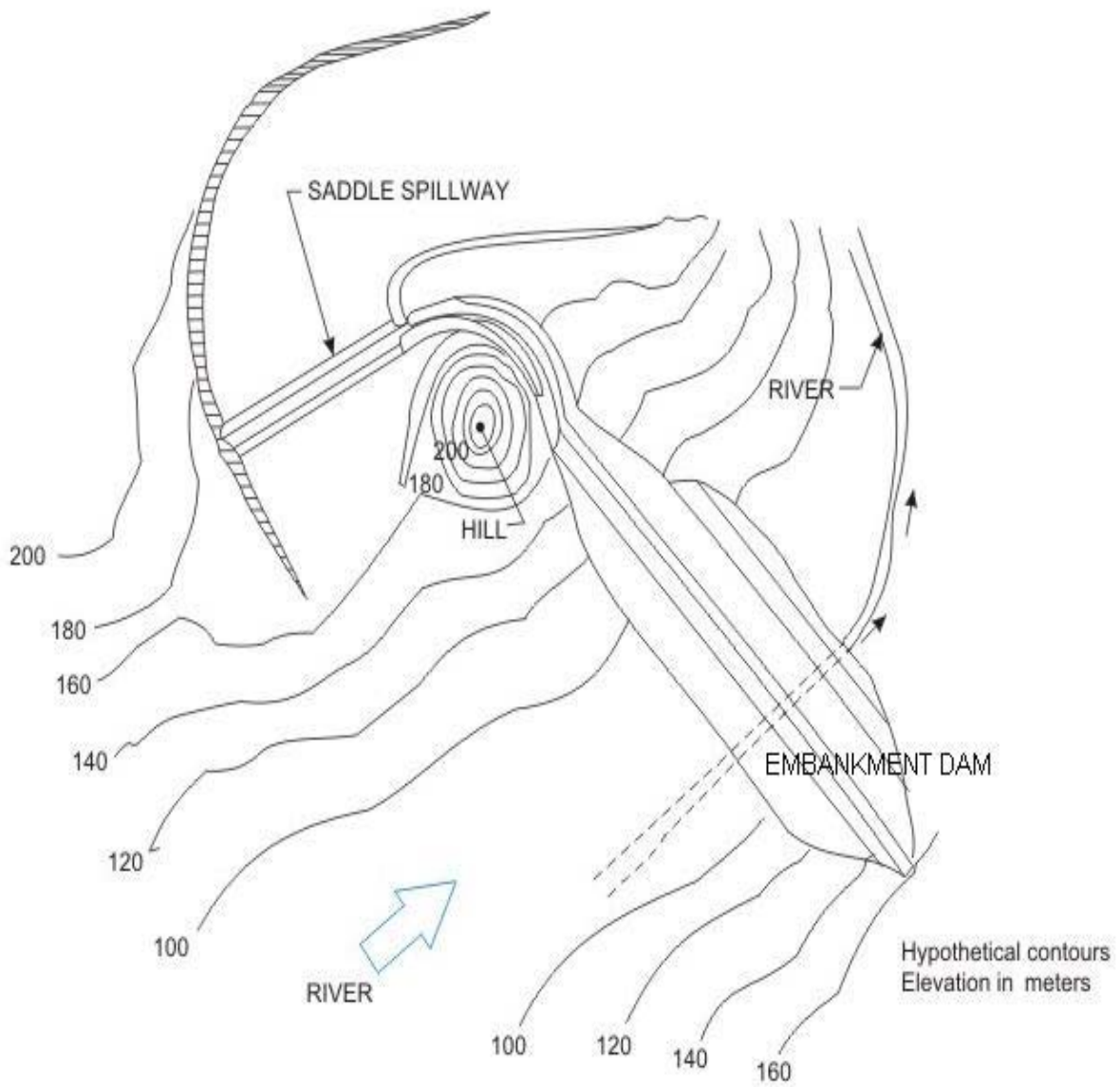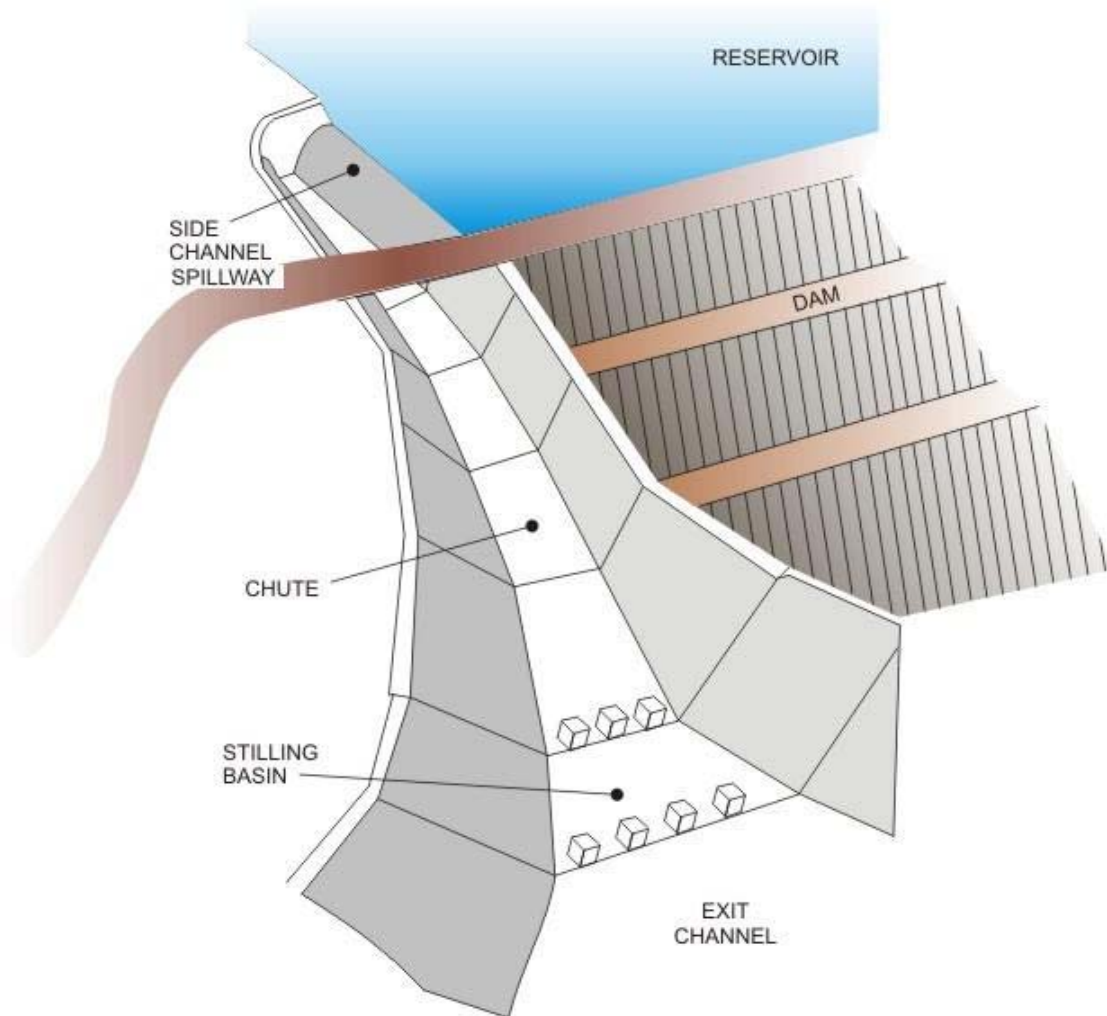
FIGURE 9. Saddle spillway

FIGURE 10. Side channel entry to a chute spillway

4.8.4 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 and precipitous.
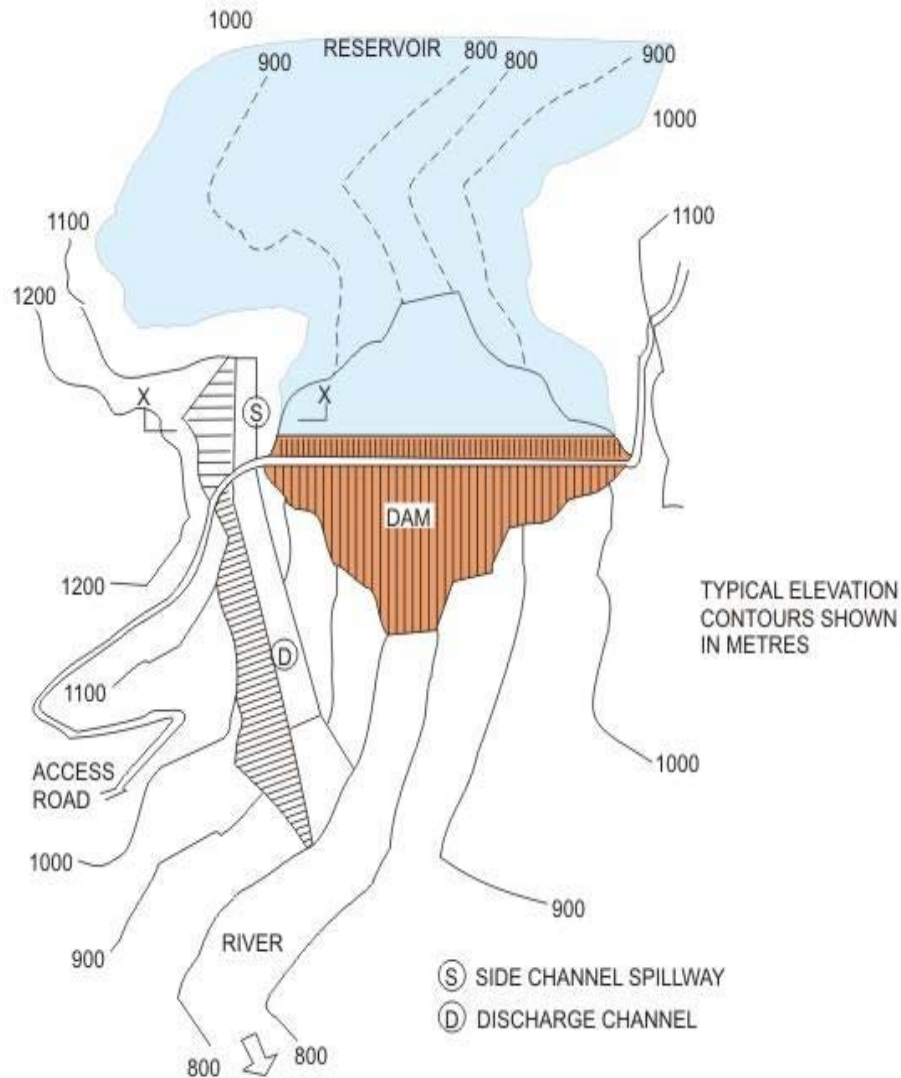

FIGURE 11. Plan of an embankment dam showing side channel spillway and chute channel
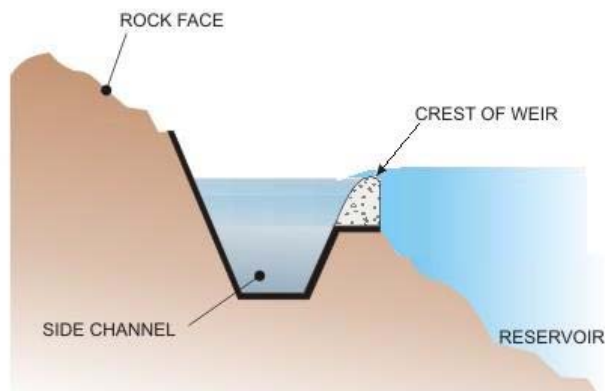
FIGURE 12. Magnified sectional view X-X through the side channel spillway shown in Figure 11

4.8.5 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 free flow.

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.

FREE BOARD

HFL

FRL

DROP
SHAFT

DAM

SPILLWAY
OUTLET

SPILLWAY CONDUIT

HFL : HIGH FLOOD LEVEL
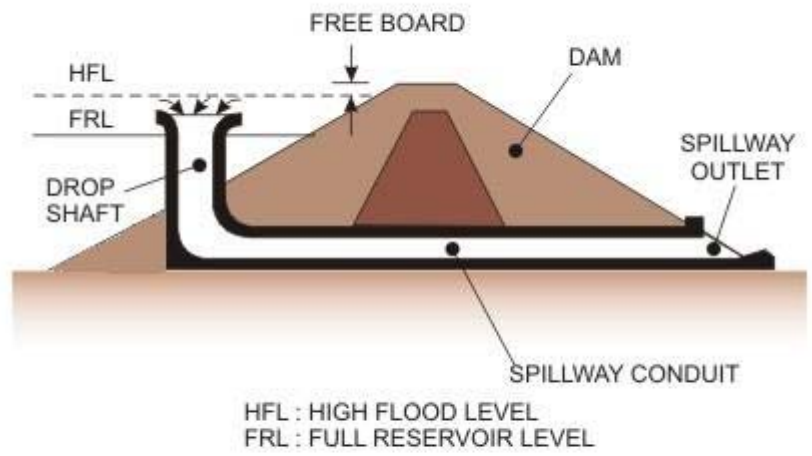FRL : FULL RESERVOIR LEVEL
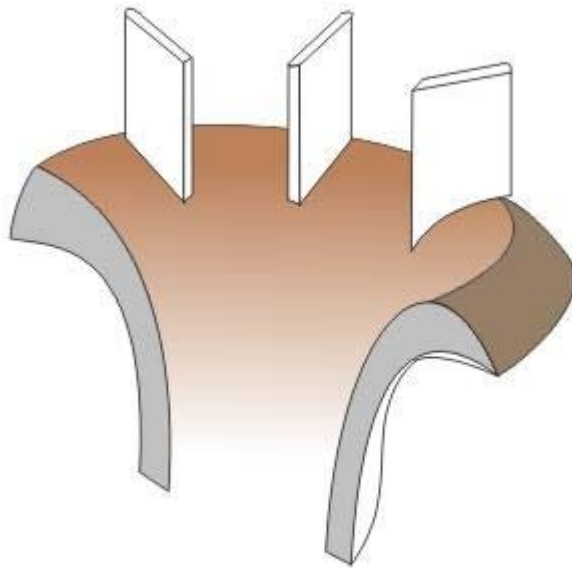
FIGURE 13. Section through a shaft spillway

FIGURE 14. Morning glory spillway with anti-vortex piers

4.8.6 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.
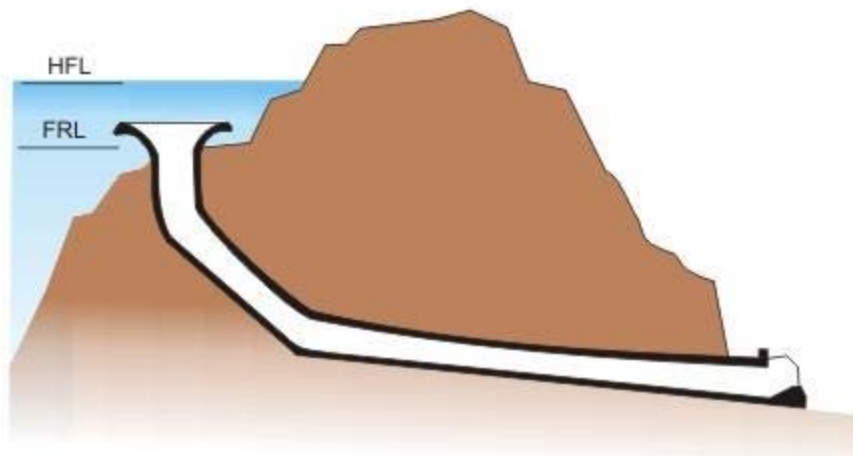

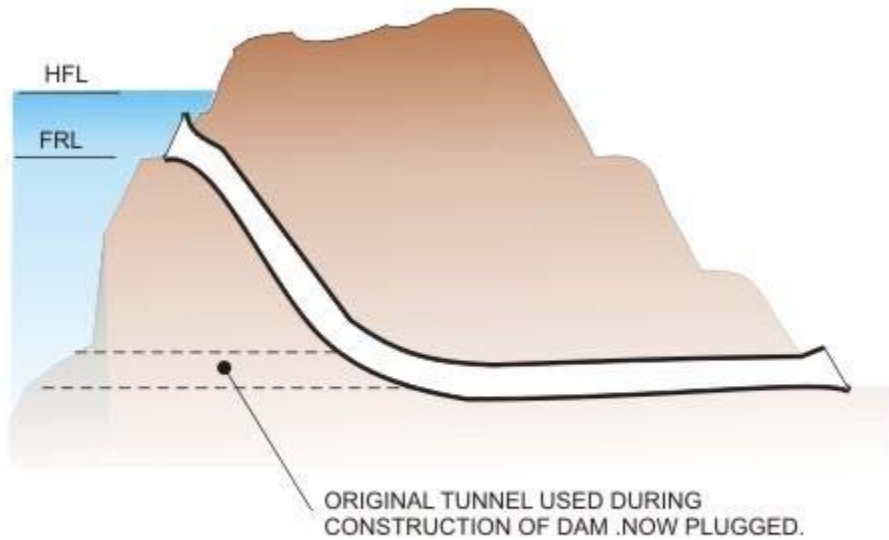Figure 15. Tunnel spillway with a morning glory entrance.

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

4.8.7 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.
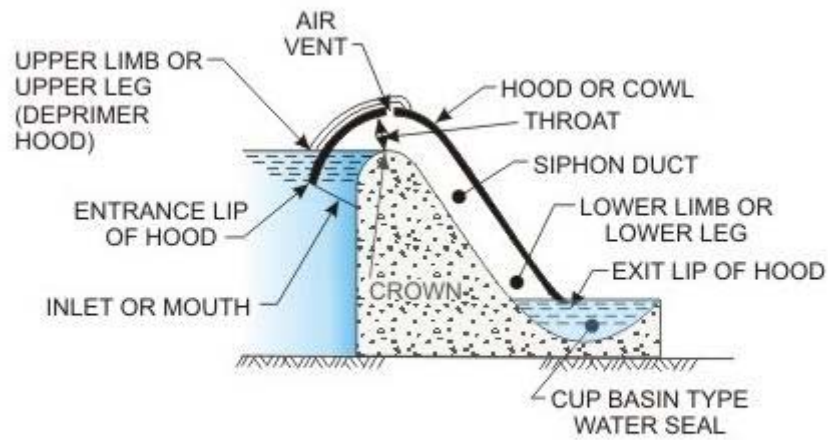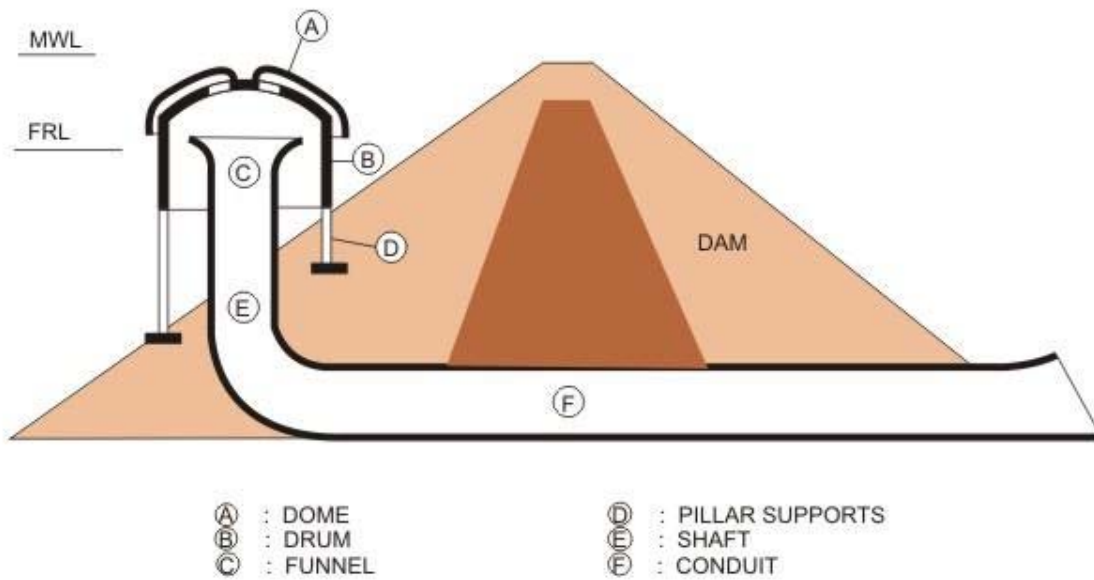
FIGURE 17. Saddle Siphon

Ⓐ	: DOME	Ⓓ	: PILLAR SUPPORTS
Ⓑ	: DRUM	Ⓔ	: SHAFT
Ⓒ	: FUNNEL	Ⓕ	: CONDUIT

FIGURE 18. Volute of siphon spillway components

4.8.8 Special types of spillways

Apart from the commonly used spillways, a few other types of spillways are used sometimes for a project, 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 sluice spillway.
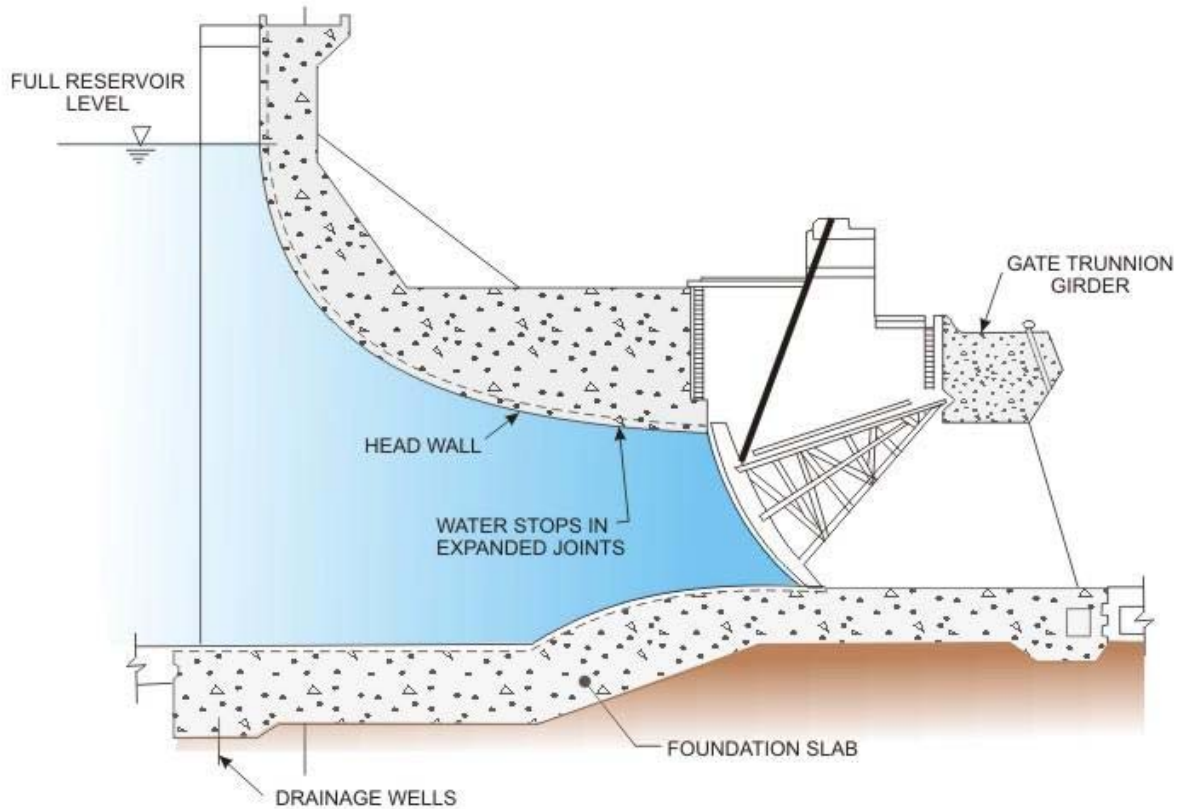
FIGURE 19. Sluice spillway

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.
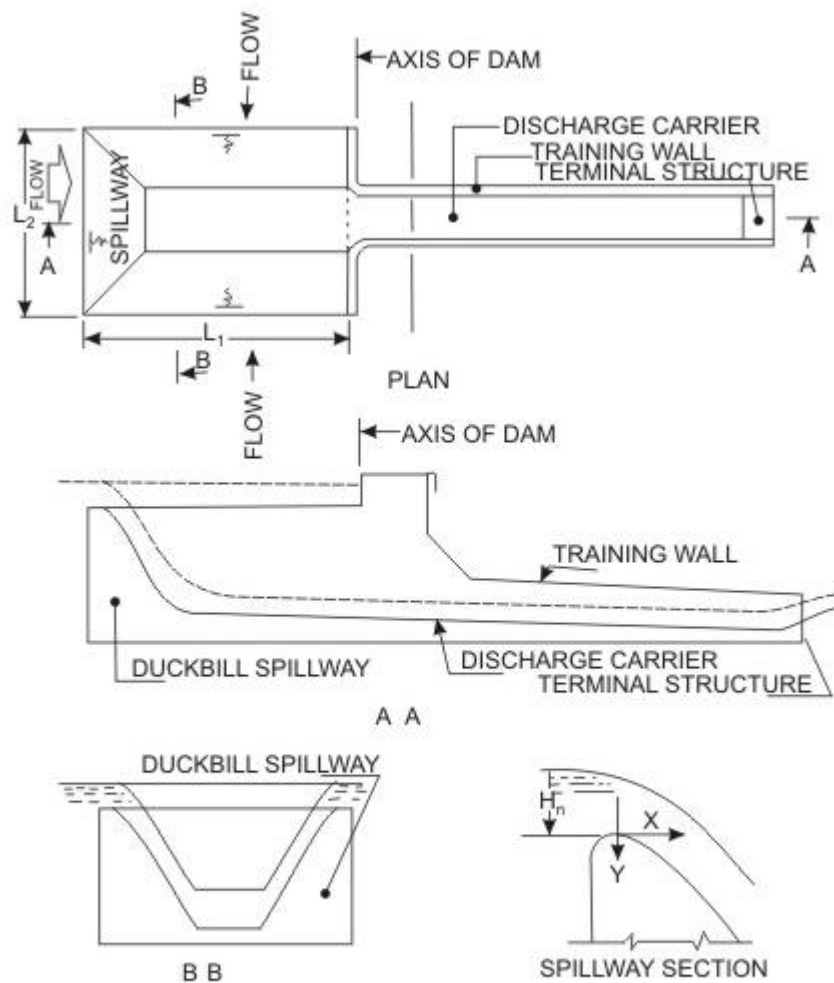
FIGURE 20. Duckbill Spillway

4.8.9 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

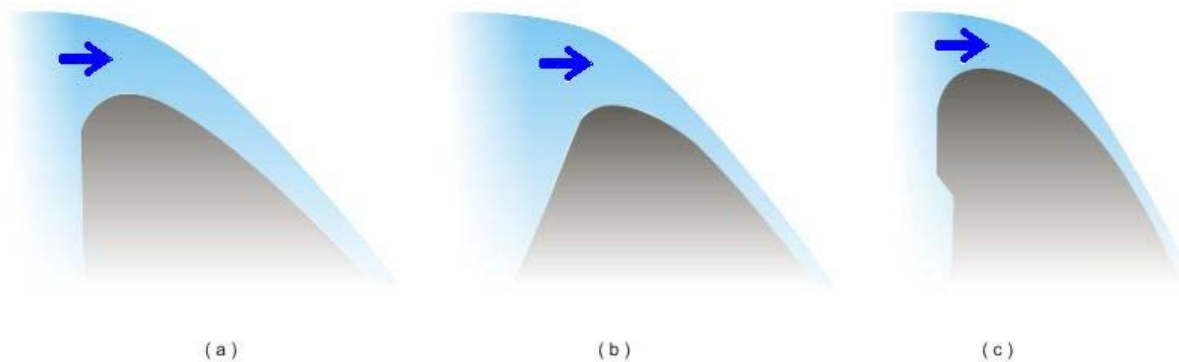(a)   (b)   (c)

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.
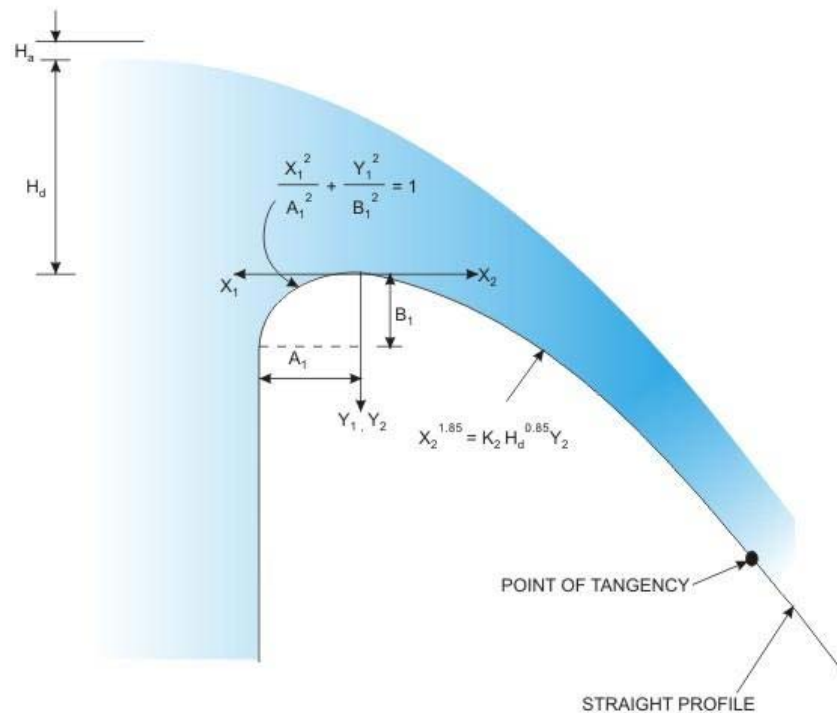
1. Ogee crests with vertical upstream face

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.
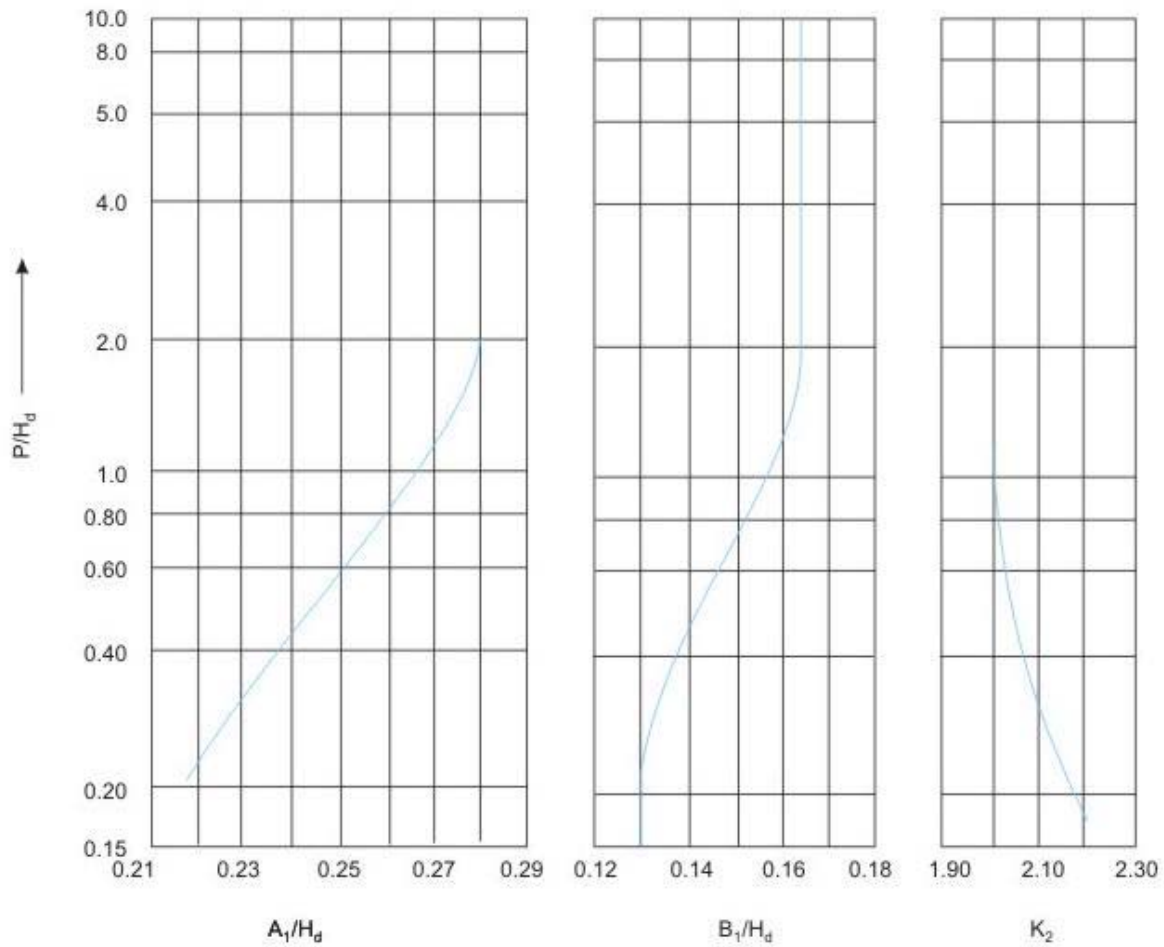
FIGURE 23. Coefficients for Figure 22

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

$$Q = C_d L_e H_e^{3/2} \tag{3}$$

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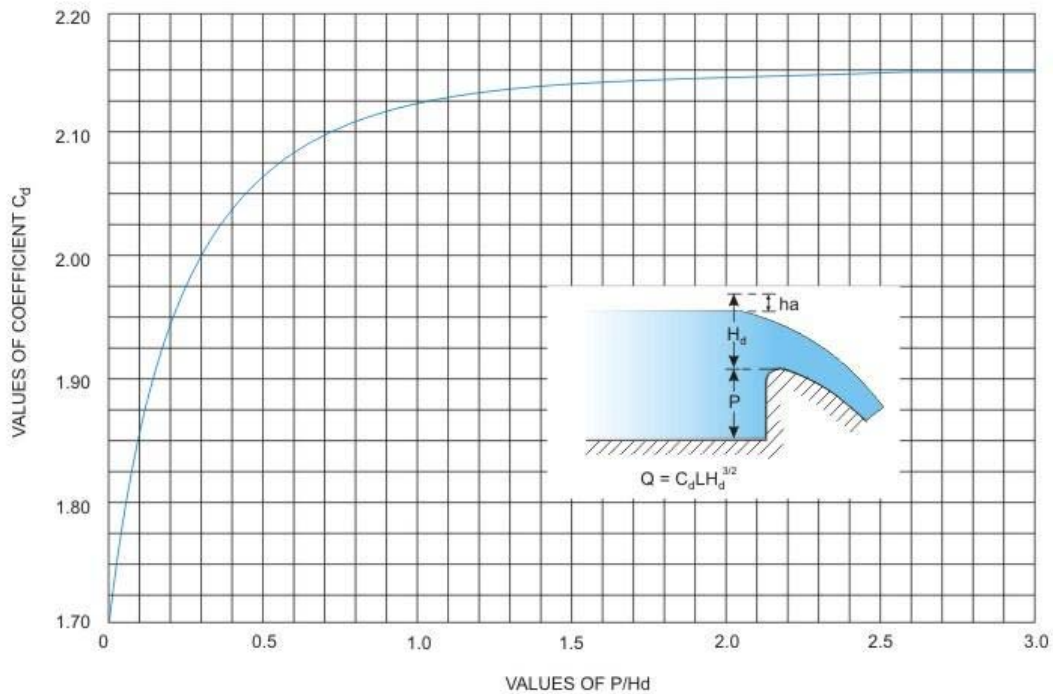
FIGURE 25. Coefficient of discharge (C$_d$) variation due to height of a vertical faced ogee crest

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.

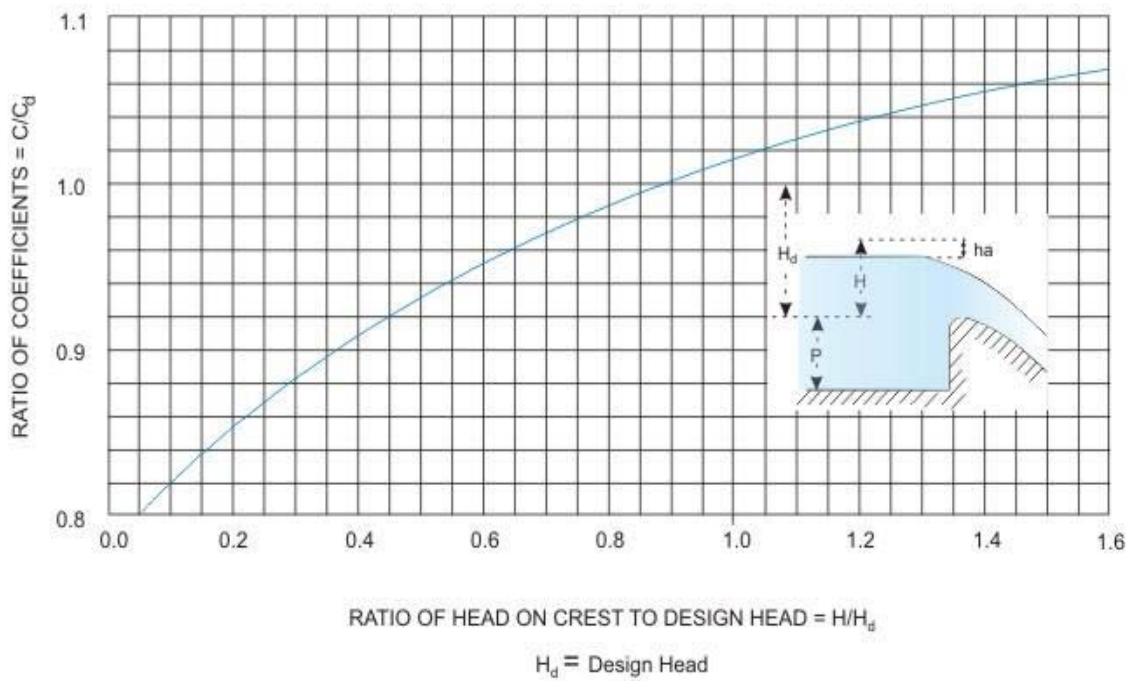RATIO OF HEAD ON CREST TO DESIGN HEAD = H/H_d

H_d = Design Head

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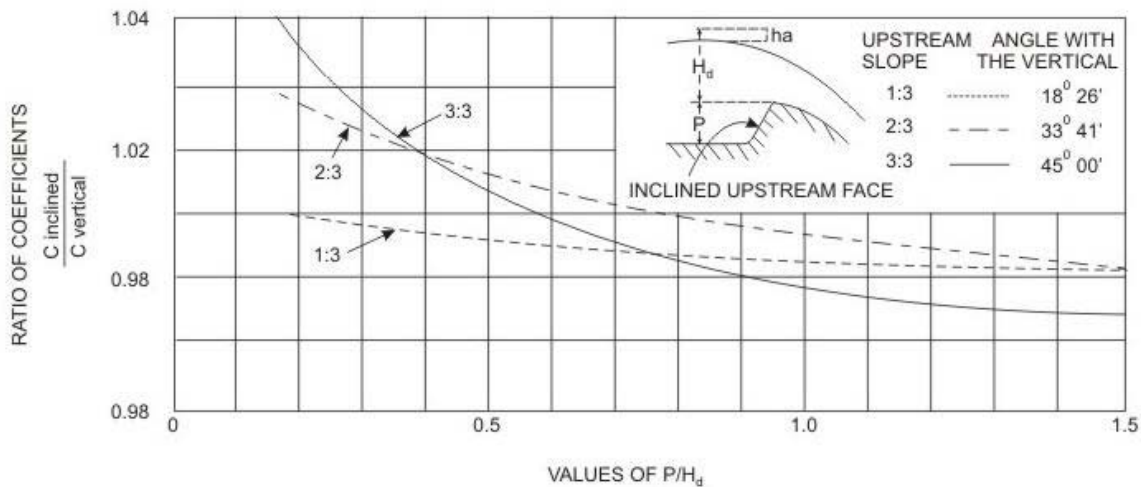
Figure 27. Coefficient of discharge variation with upstream face inclination.

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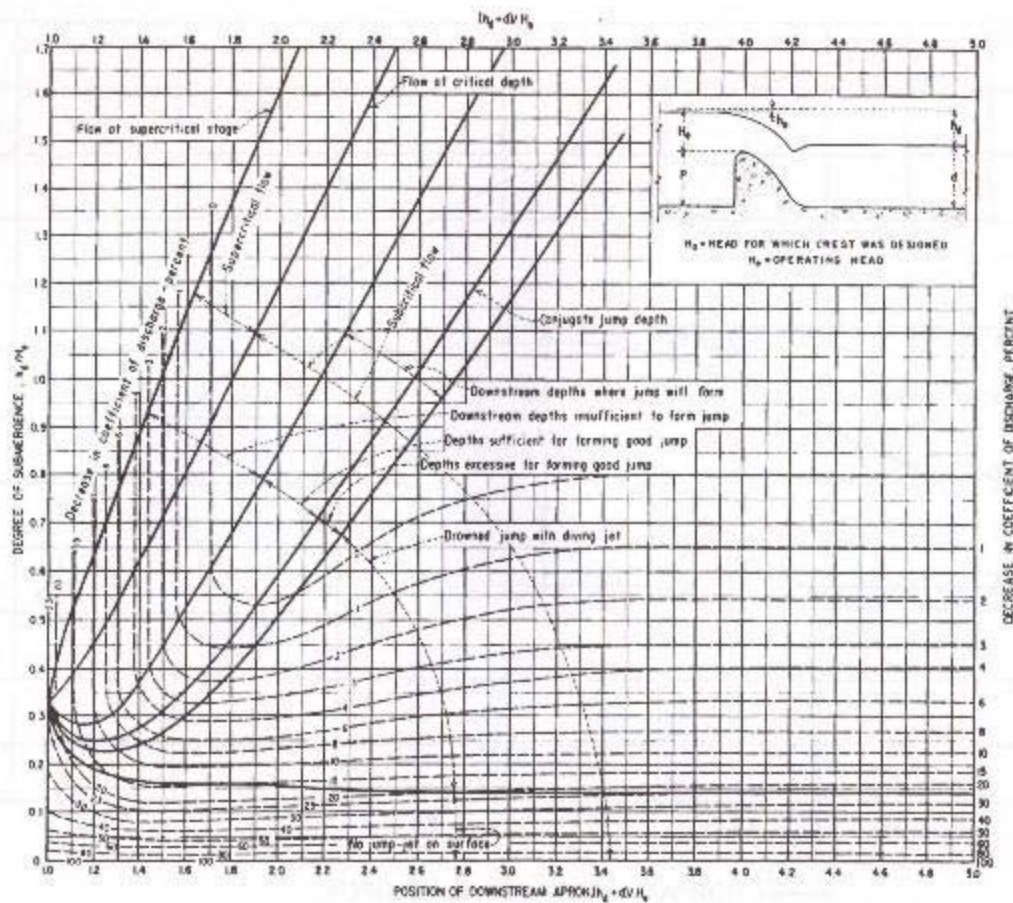
FIGURE 28. Effects of downstream influences on flow over weir crests

(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 $\dfrac{h_d + d}{H_e}$ 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.
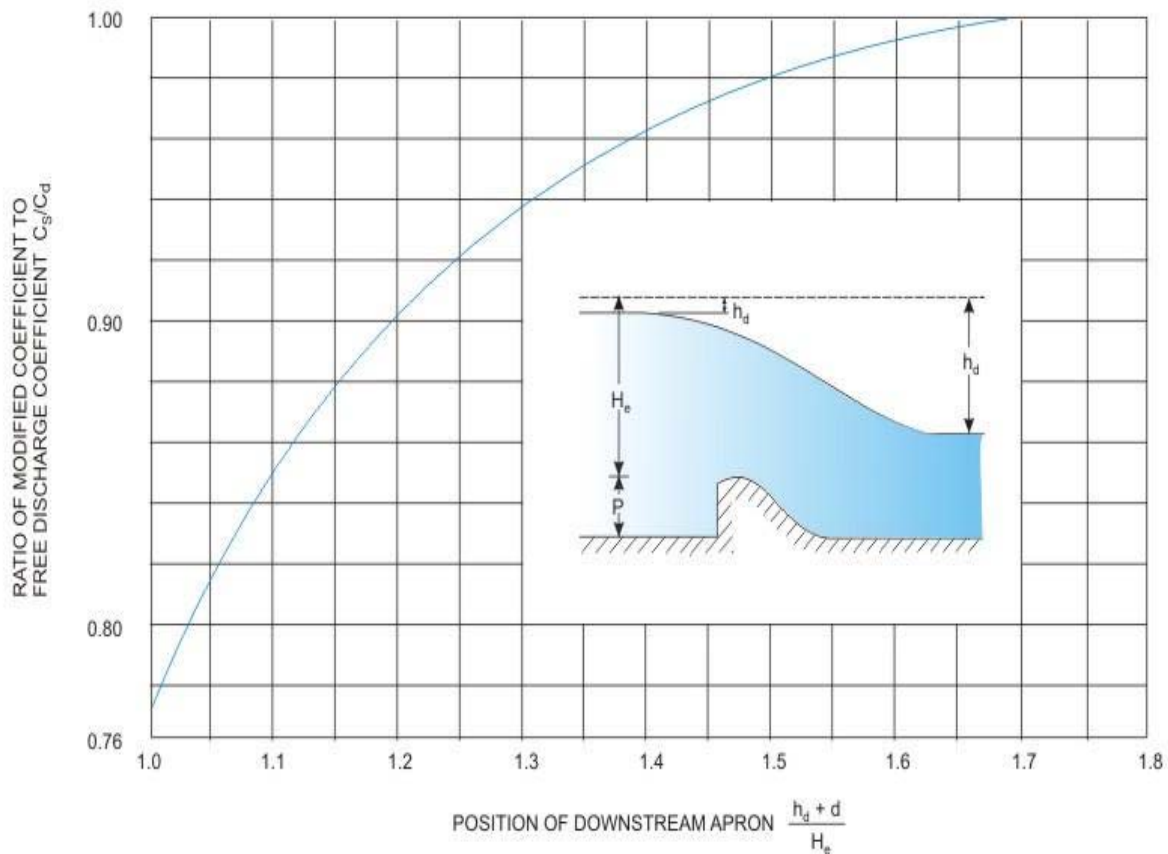
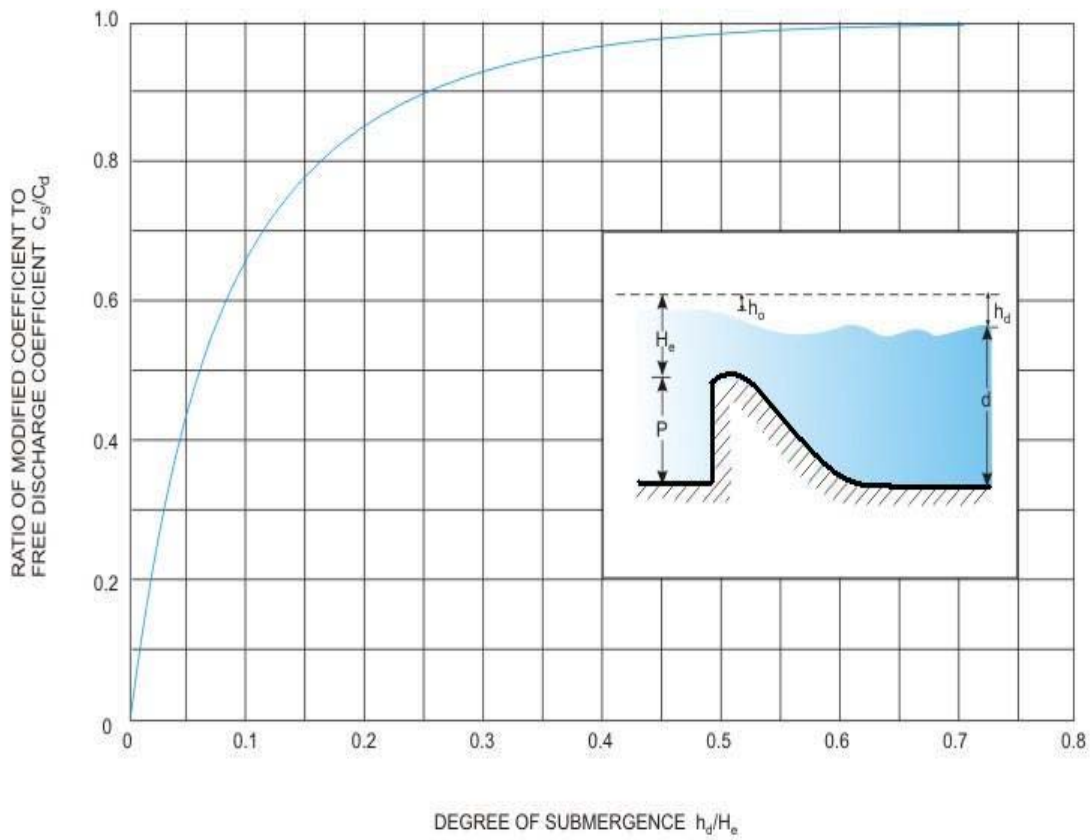FIGURE 29. Ratio of discharge coefficients resulting from the effect of the apron on flow

FIGURE 30. Ratio of discharge coefficients caused by tailwater effects

If the ordinate of Figure 30 is changed from $\dfrac{h_d}{H_e}$ to $1 - \dfrac{h_d}{H_e}$, that is, equal to $\dfrac{H_e - h_d}{H_e} = \dfrac{h}{H_e}$ where h is the downstream water depth measured above crest, then the curve of Figure 30 may be transposed as in Figure 31.
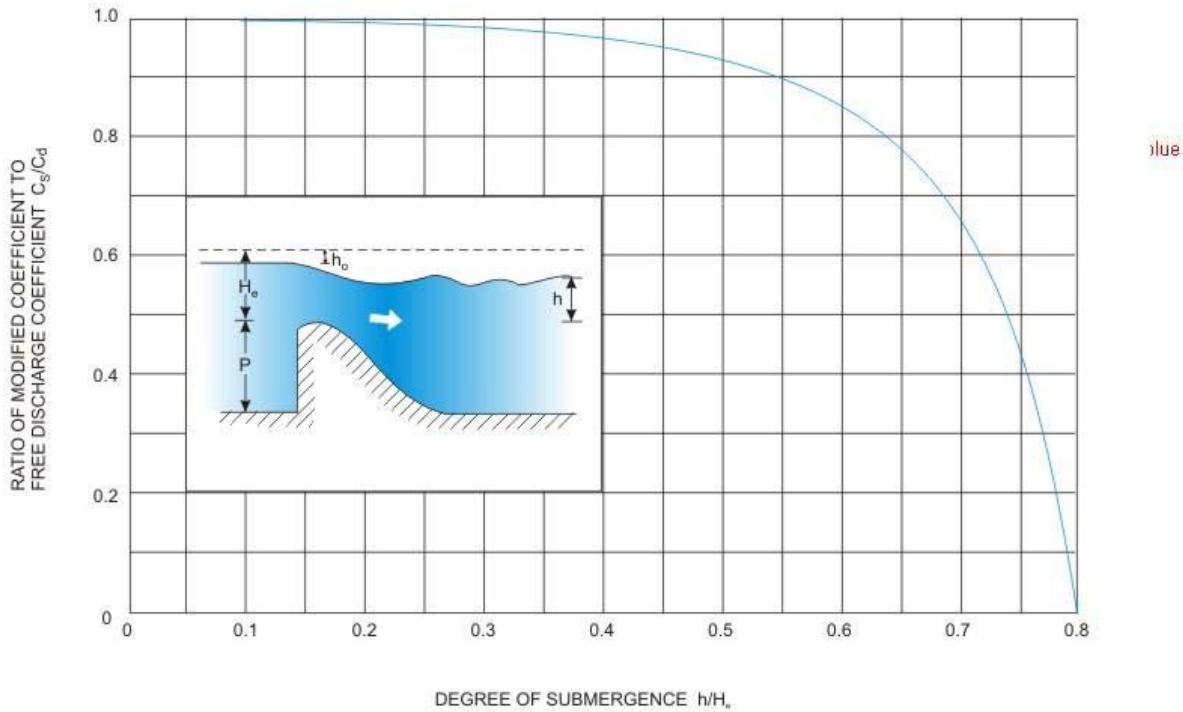
FIGURE 31. Ratio of submerged discharge coefficient to that without submergence effect
This figure is similar to that given in FIGURE 30, but with different ordinate value

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:

$$L_e = L - 2(N \cdot K_p + K_a) \cdot H_e$$

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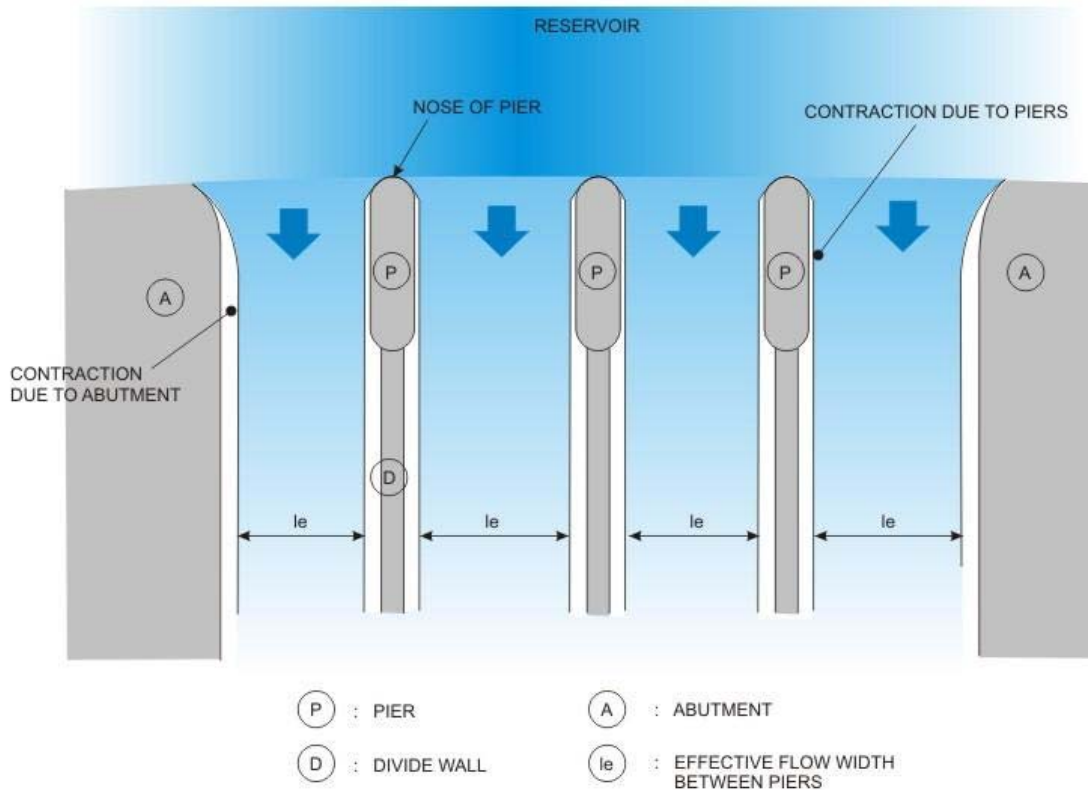
FIGURE 32. Abutment and pier contractions shown on a spillway plan

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.
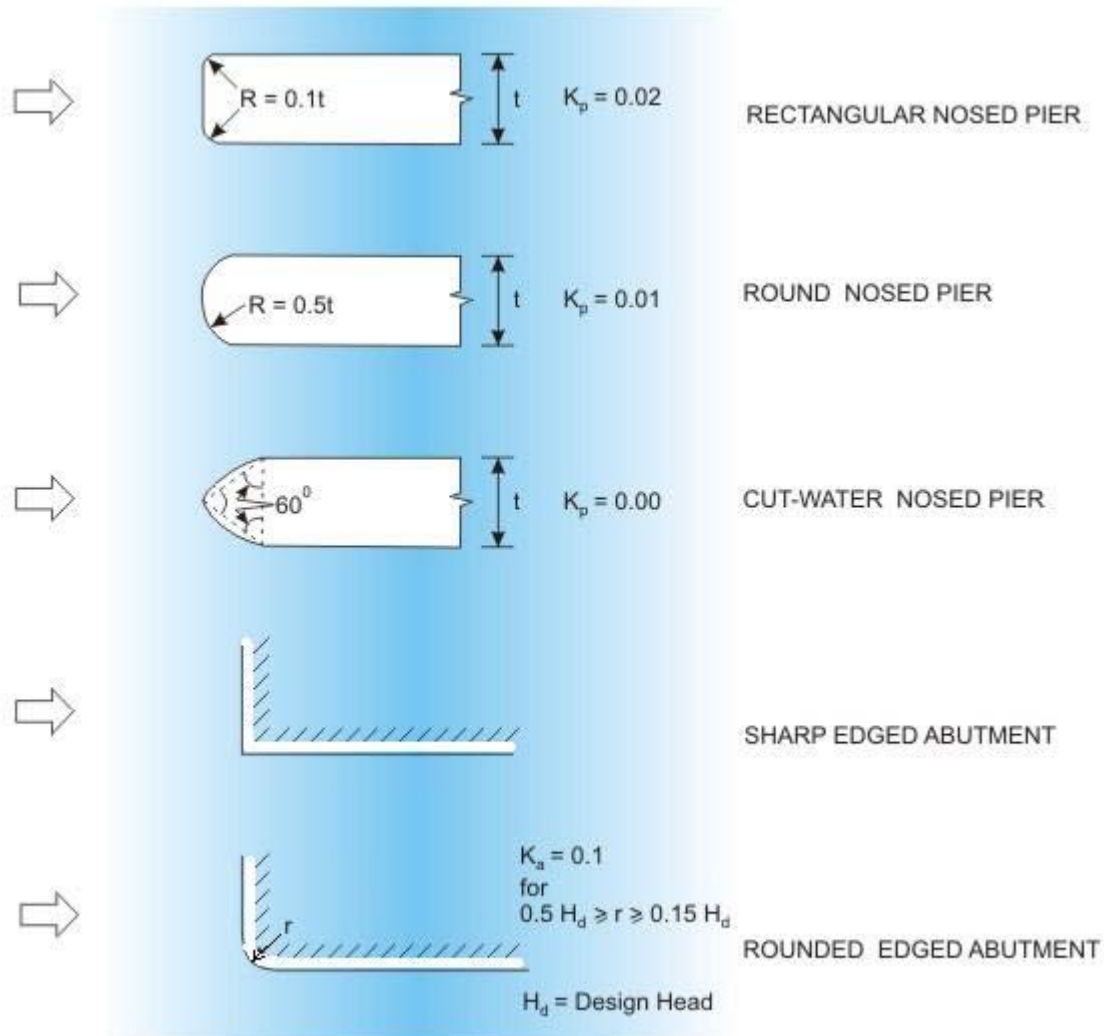
FIGURE 33. Recommended values of K_p and K_a

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".

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

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).
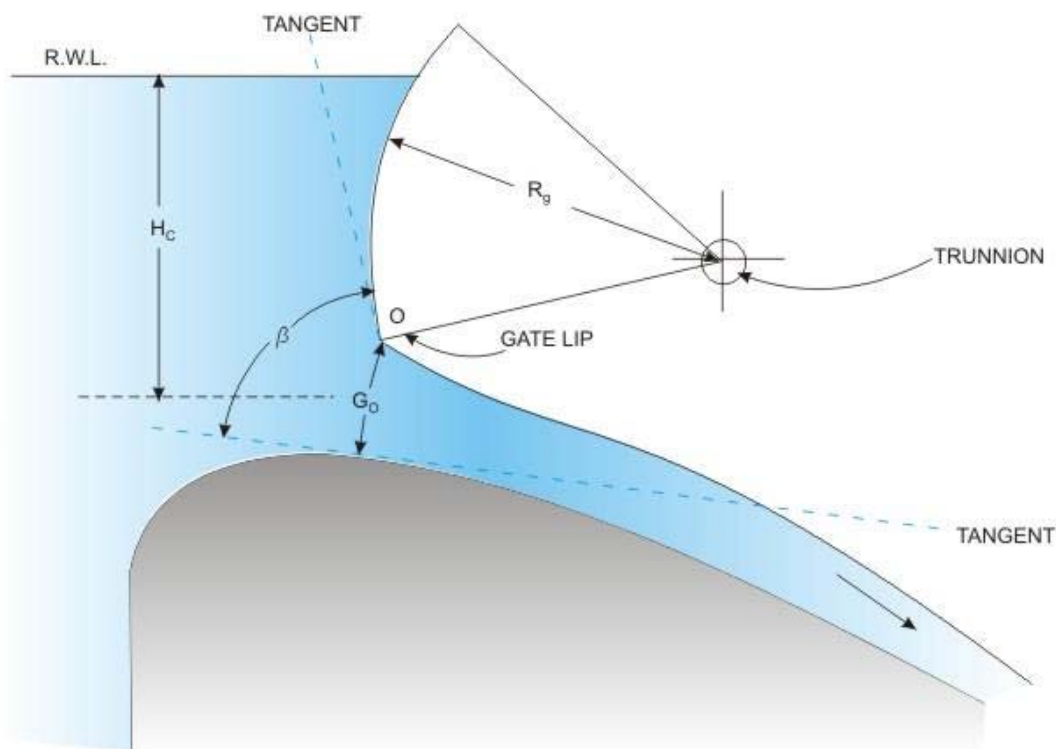

FIGURE 34. Partially opened radial gate discharging flow

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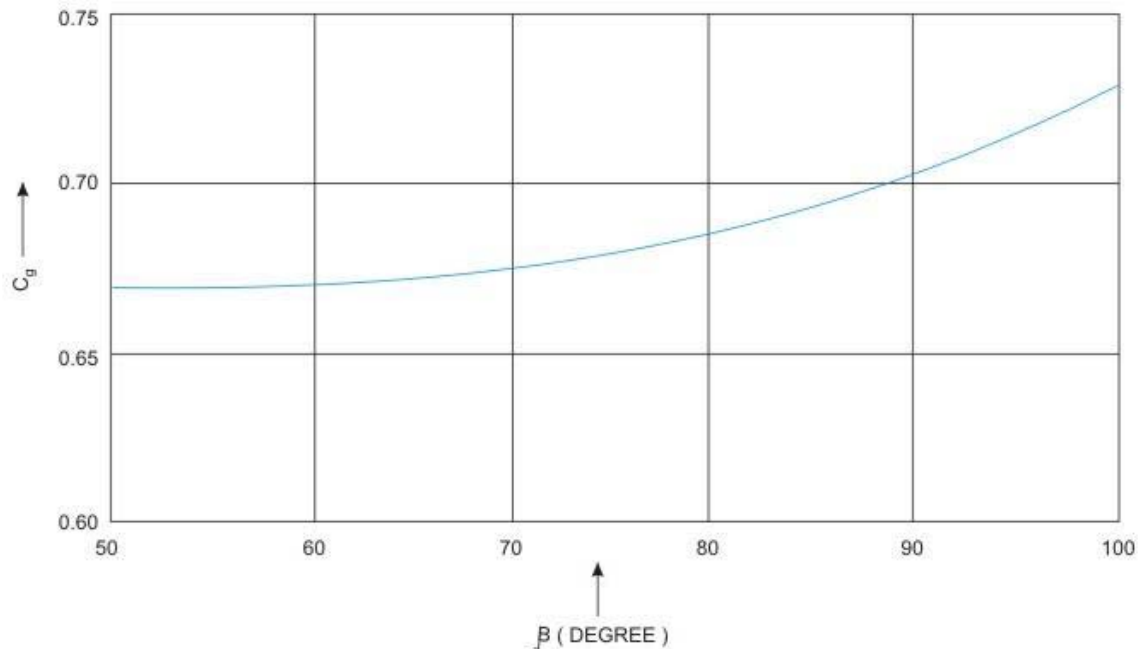
FIGURE 35. Coefficient of gated discharge Cg variation with lip angle β

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.

4.8.10   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.
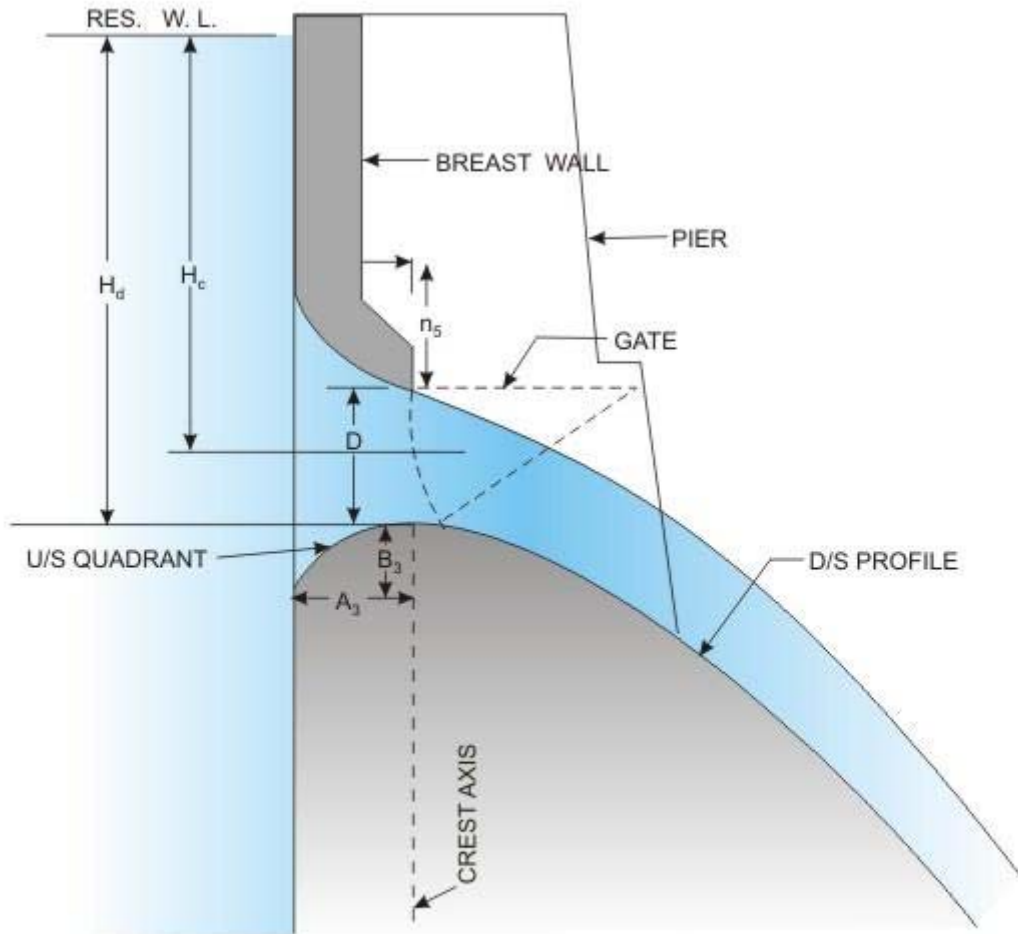

FIGURE 36. Spillway with breast wall

Ogee Profile - Upstream Quadrant

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

$$\frac{X_3^2}{A_3^2} + \frac{Y_3^2}{B_3^2} = 1 \tag{5}$$

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:

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

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:

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

where

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

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

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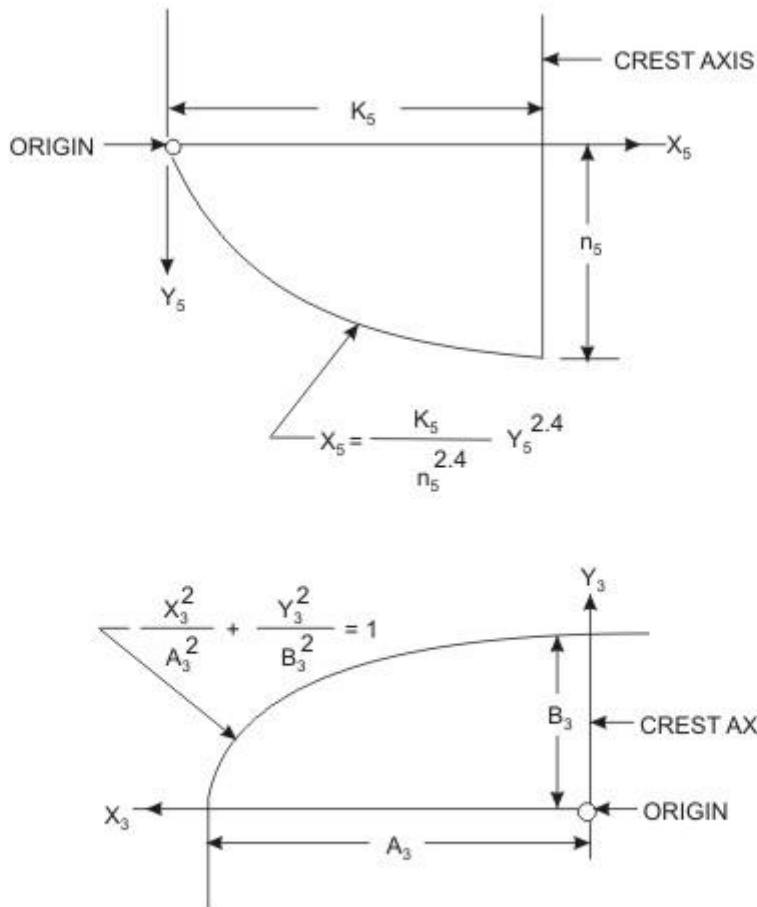
FIGURE 37. Upstream profile of ogee crest and bottom profile of breast wall details

Discharge Computation

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

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

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

4.8.11   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.

4.8.12   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
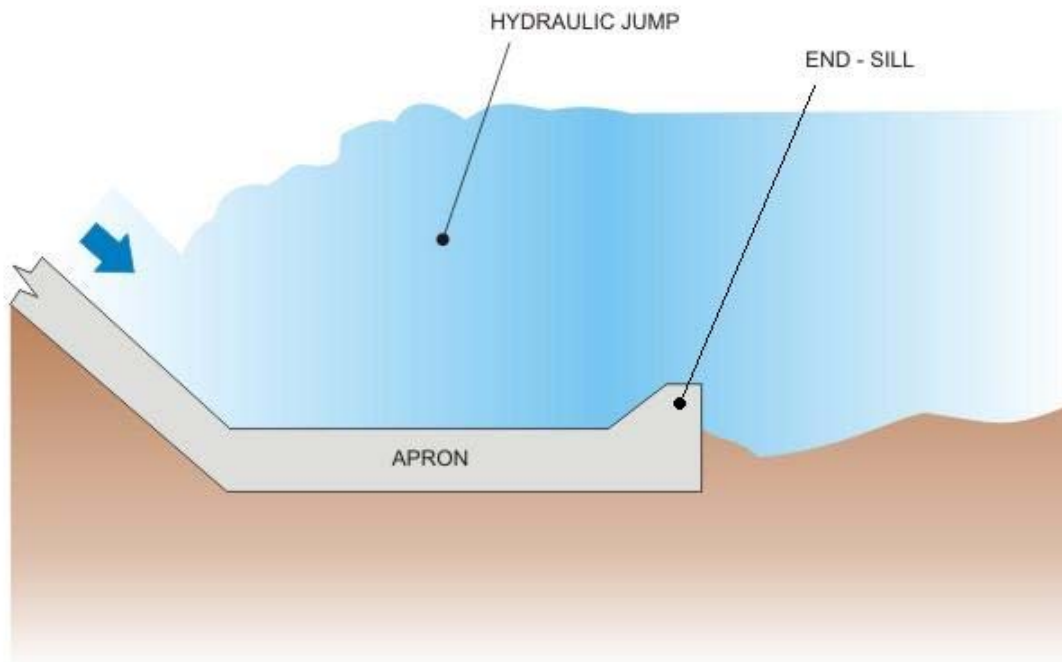 1. Horizontal apron type (Figure 38)

FIGURE 38. Horizontal apron stilling basin with end-sill
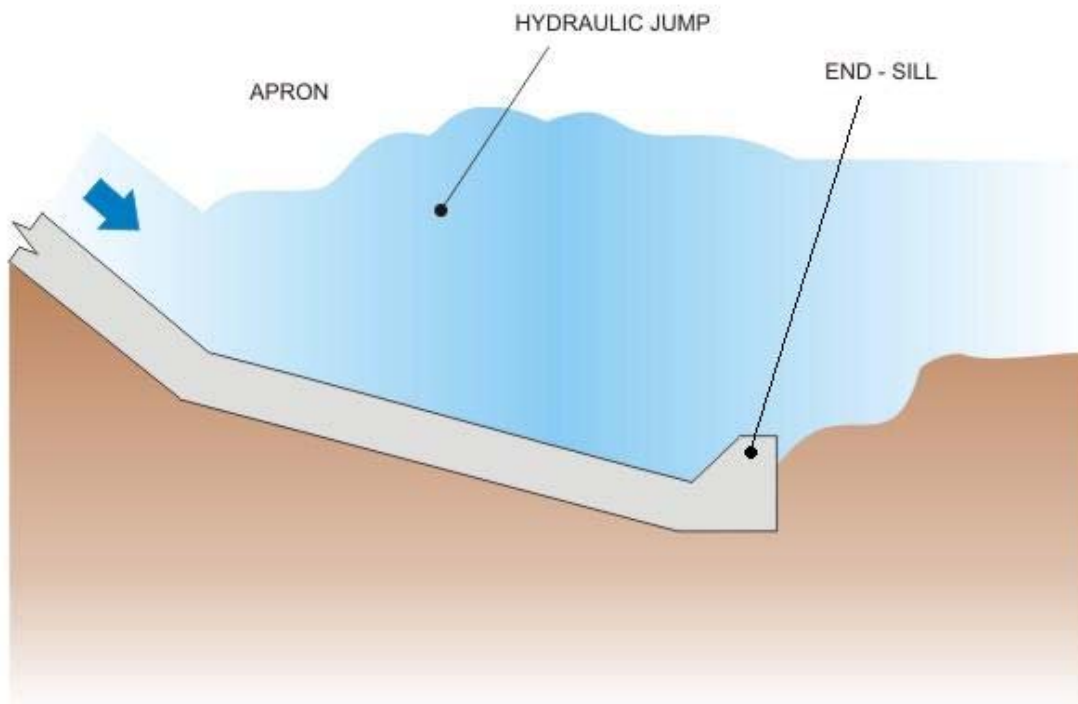
2. Sloping apron type (Figure 39)

FIGURE 39. Sloping apron stilling basin with end-sill

b) Jet diffusion type stilling basins
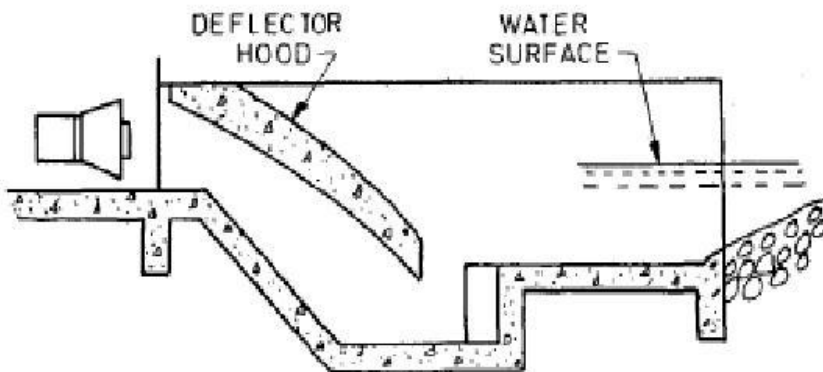   1. Jet diffusion stilling basins (Figure 40)

FIGURE 40. Jet diffusion stilling basin

(Image courtesy: IS 10137)
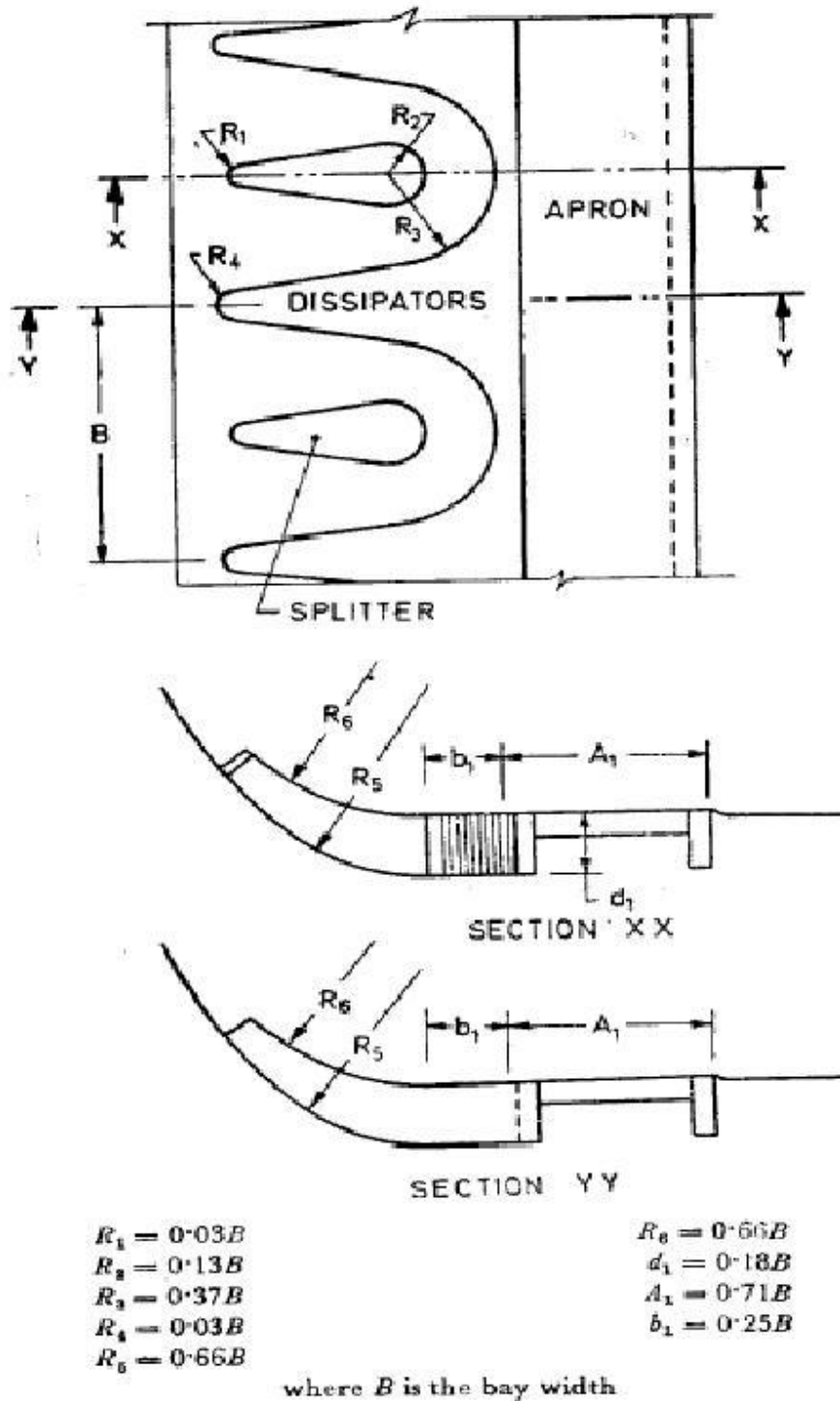
2. Interacting jet dissipators (Figure 41)

$$R_1 = 0.03B$$
$$R_2 = 0.13B$$
$$R_3 = 0.37B$$
$$R_4 = 0.03B$$
$$R_5 = 0.66B$$

$$R_6 = 0.66B$$
$$d_1 = 0.18B$$
$$A_1 = 0.71B$$
$$b_1 = 0.25B$$

where B is the bay width

FIGURE 41. Interacting jet dissipators

(Image courtesy: IS 10137)

3. Free jet stilling basins (Figure 42)
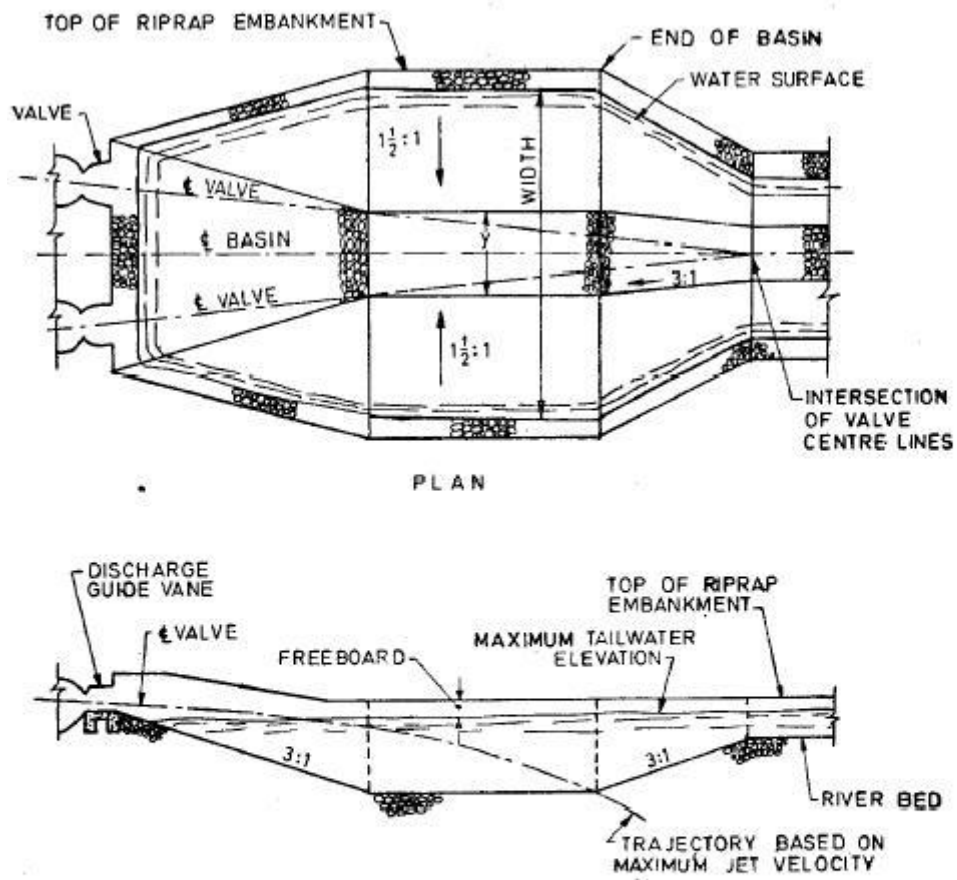
FIGURE 42. Free jet stilling basin

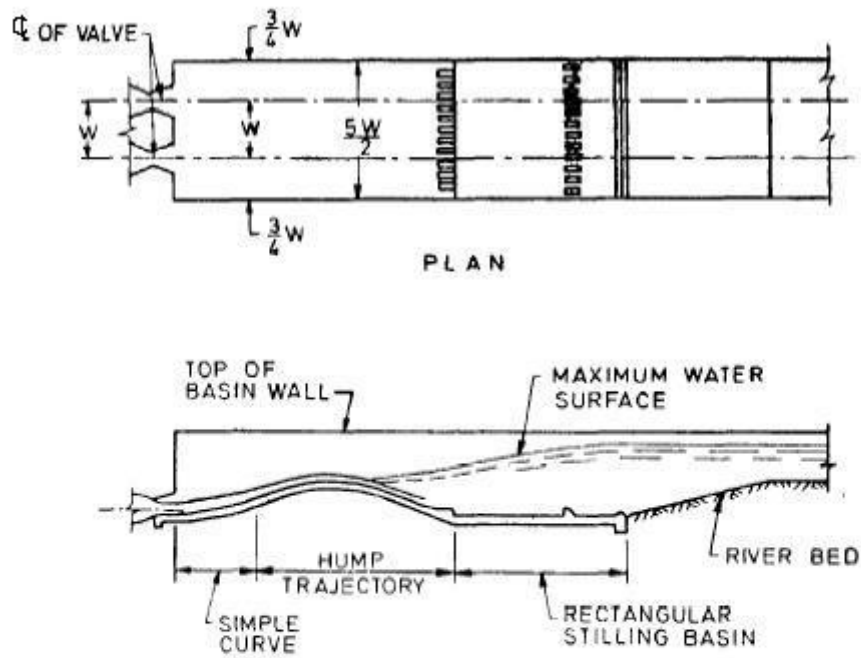(Image courtesy: IS 10137)

4. Hump stilling basins (Figure 43)

FIGURE 43. Hump stilling basin

(Image courtesy: IS 10137)

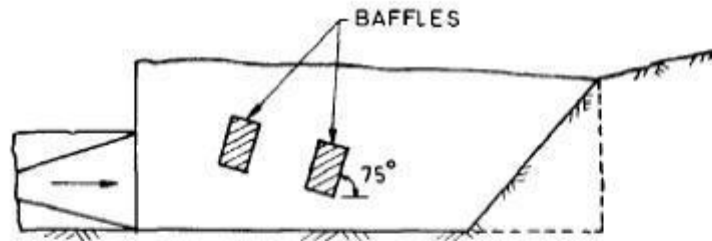5. Impact stilling basins (Figure 44)

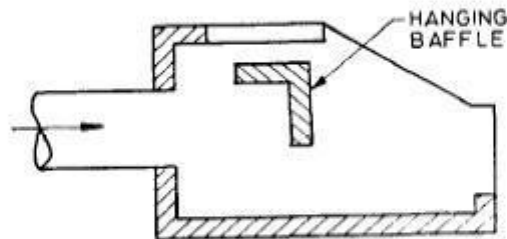FIG. 15 IMPACT STILLING BASIN WITH INCLINED BAFFLES

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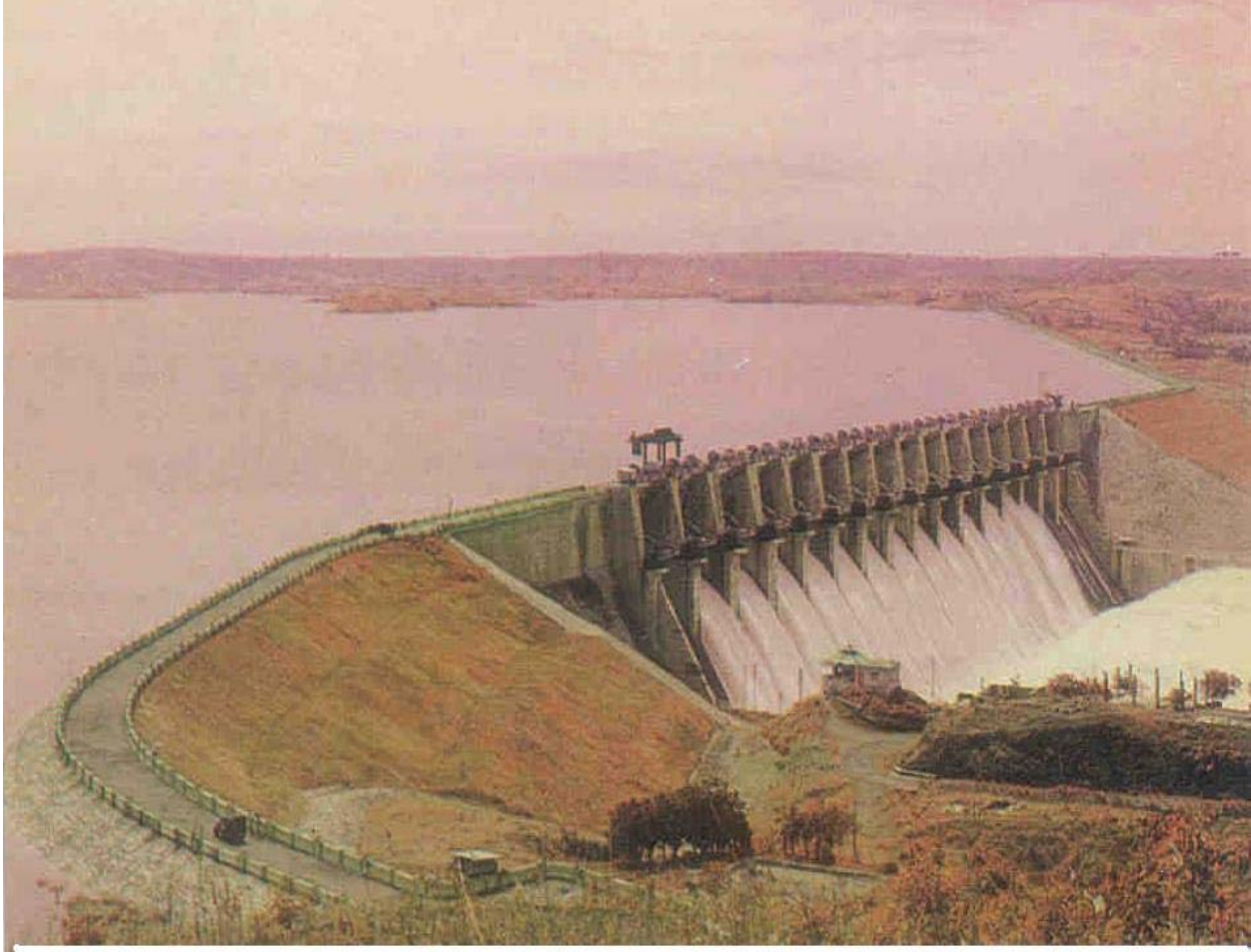
FIGURE 45. Mahi Bajaj Sagar Dam across river Mahi in Rajasthan showing ski-jump
bucket energy dissipators in action

(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)

4.8.13 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.
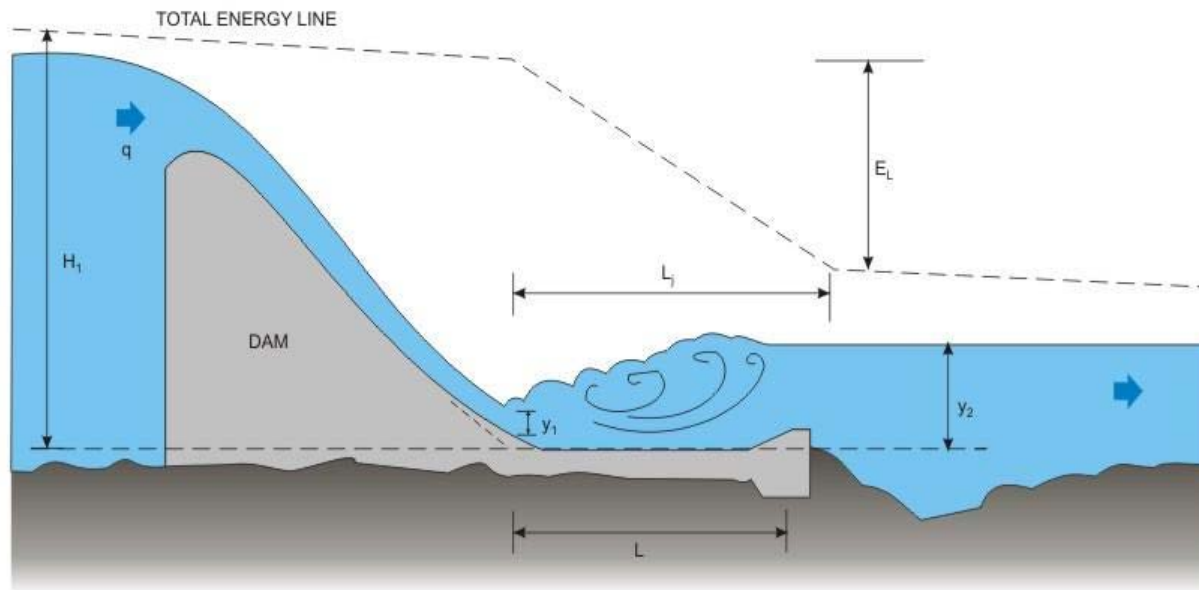
L : LENGTH OF STILLING BASIN APRON

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:

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

(14)

Where F$_1$ is the incoming Froude number = $\dfrac{V_1}{\sqrt{gy_1}}$

Alternatively, the expression may be written in terms of the outgoing Froude number F$_2$

$\left(=\dfrac{V_2}{\sqrt{gy_2}}\right)$ as

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

(15)

where **V$_1$** and **V$_2$** are the incoming and outgoing velocities and **g** is the acceleration due to gravity.

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

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

In most cases, it is possible to find out the pre-jump depth (y_1) and velocity (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. V_1 may be assumed to be equal to $\sqrt{2gH_1}$, where 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 (y_2) and velocity (V_2).

The length of the jump (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 (F_1), and post-jump depth (y_2) as shown in Figure 48. It is evident that while L_j/y_2 varies most for small values of F_1, at higher values, say above 5 or so, L_j/y_2 is practically constant at a value of about 6.1.
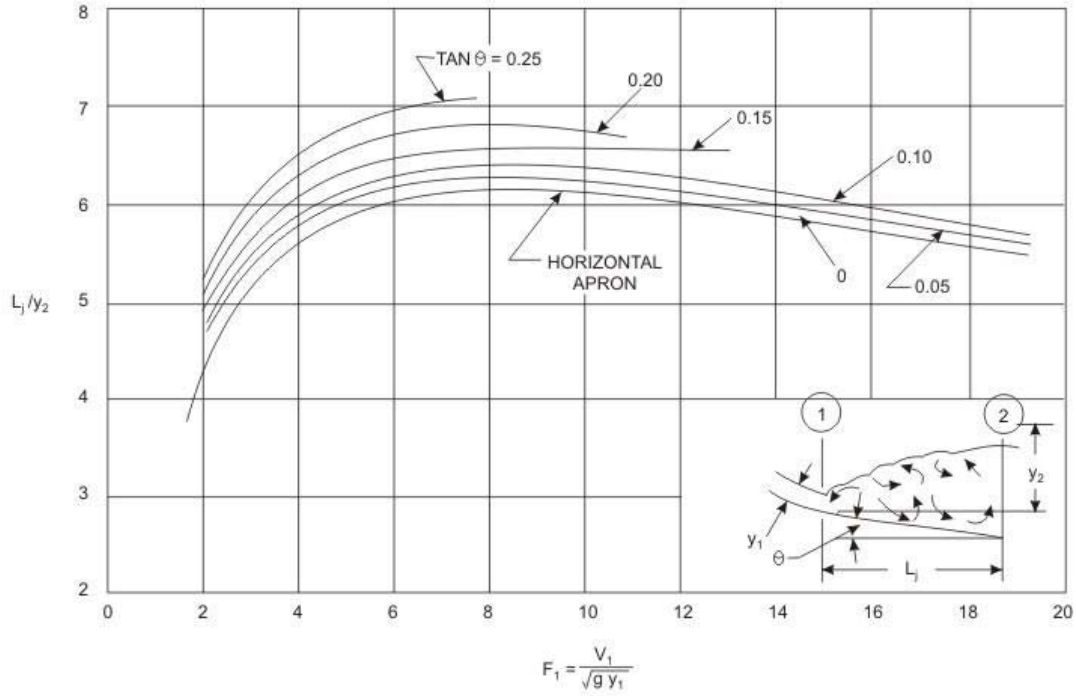
FIGURE 48. Length of hydraulic jump on a horizontal or inclined floor

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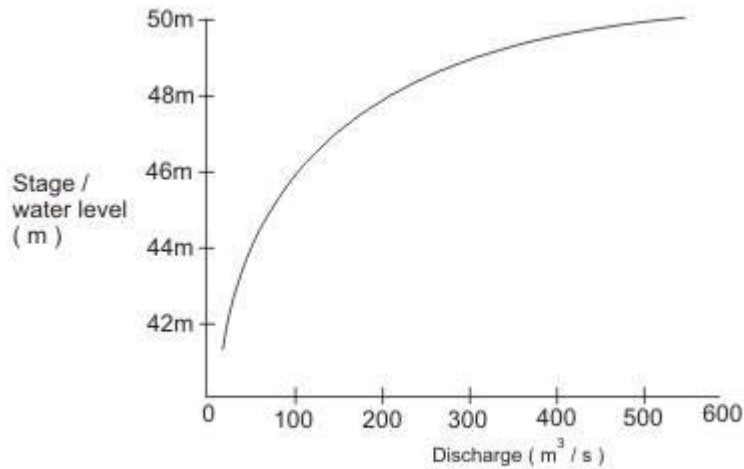
FIGURE 49. A typical stage-discharge curve for a river

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).
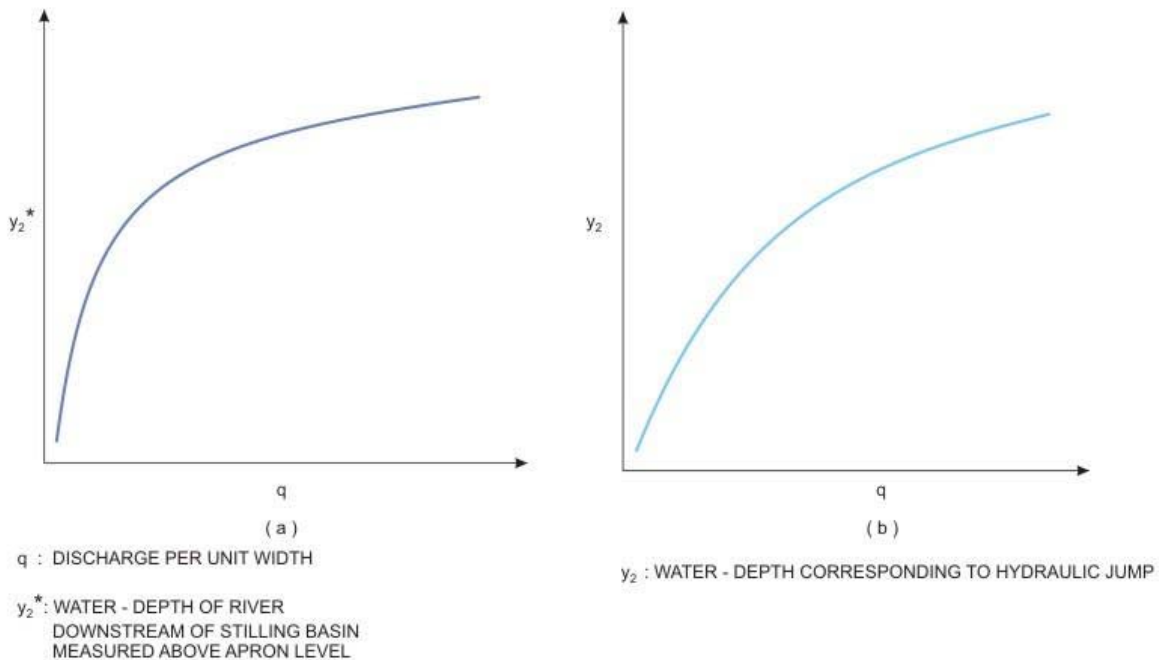

q : DISCHARGE PER UNIT WIDTH

y_2^*: WATER - DEPTH OF RIVER
        DOWNSTREAM OF STILLING BASIN
        MEASURED ABOVE APRON LEVEL

y_2 : WATER - DEPTH CORRESPONDING TO HYDRAULIC JUMP

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.
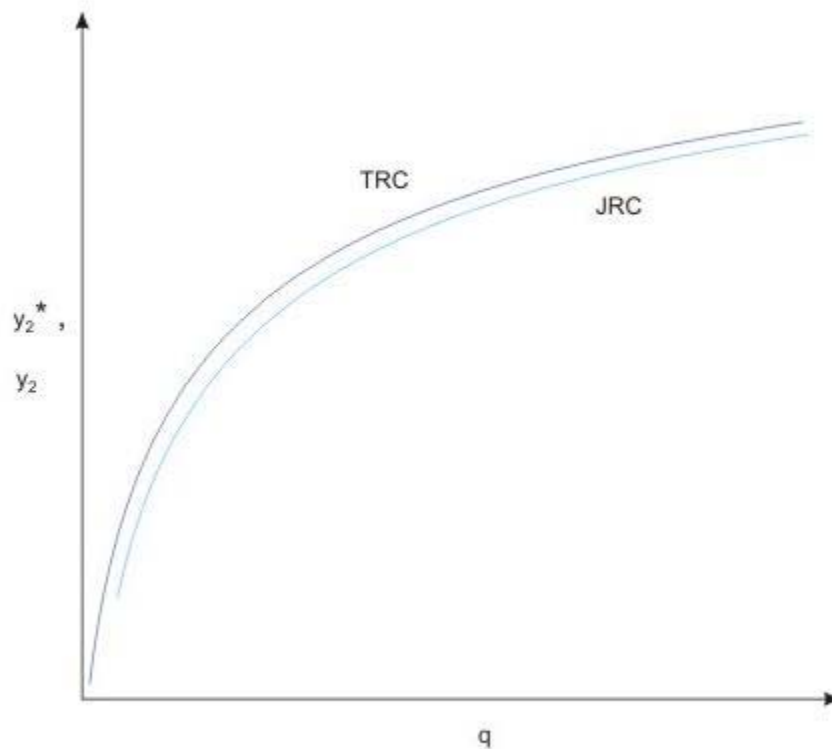

FIGURE 51. TRC & JRC coinciding

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.
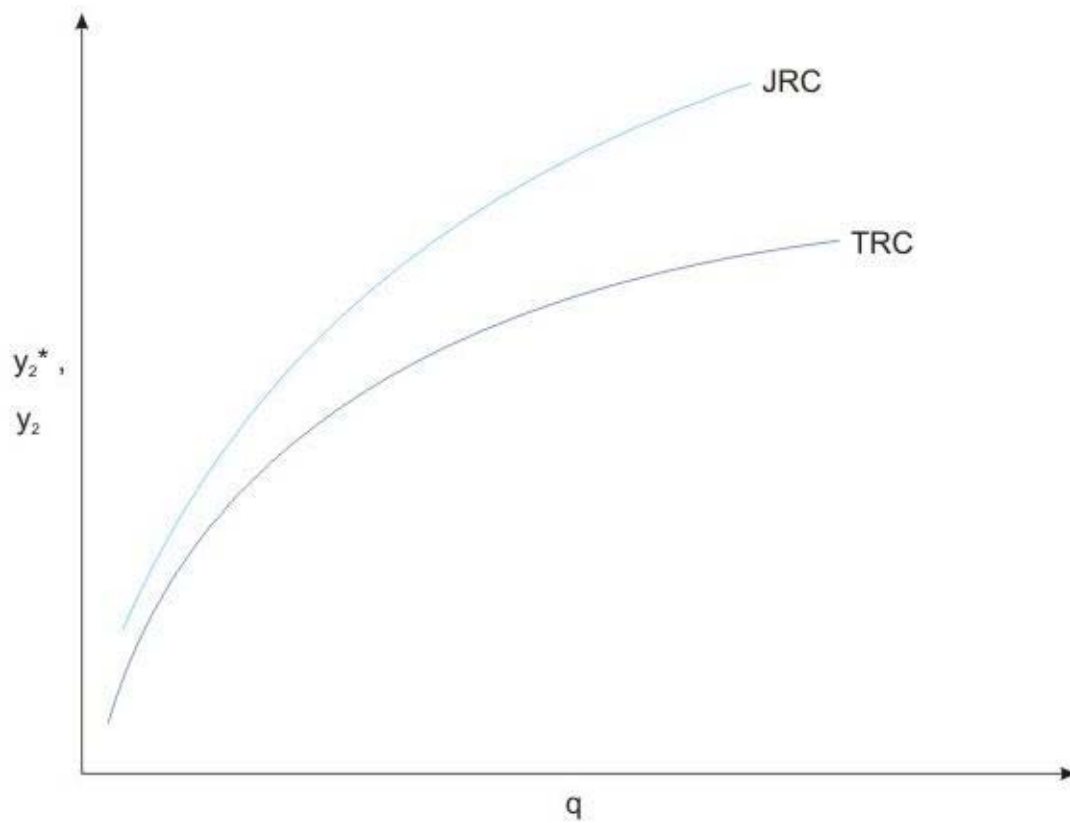
FIGURE 52. TRC below JRC for all discharges

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)
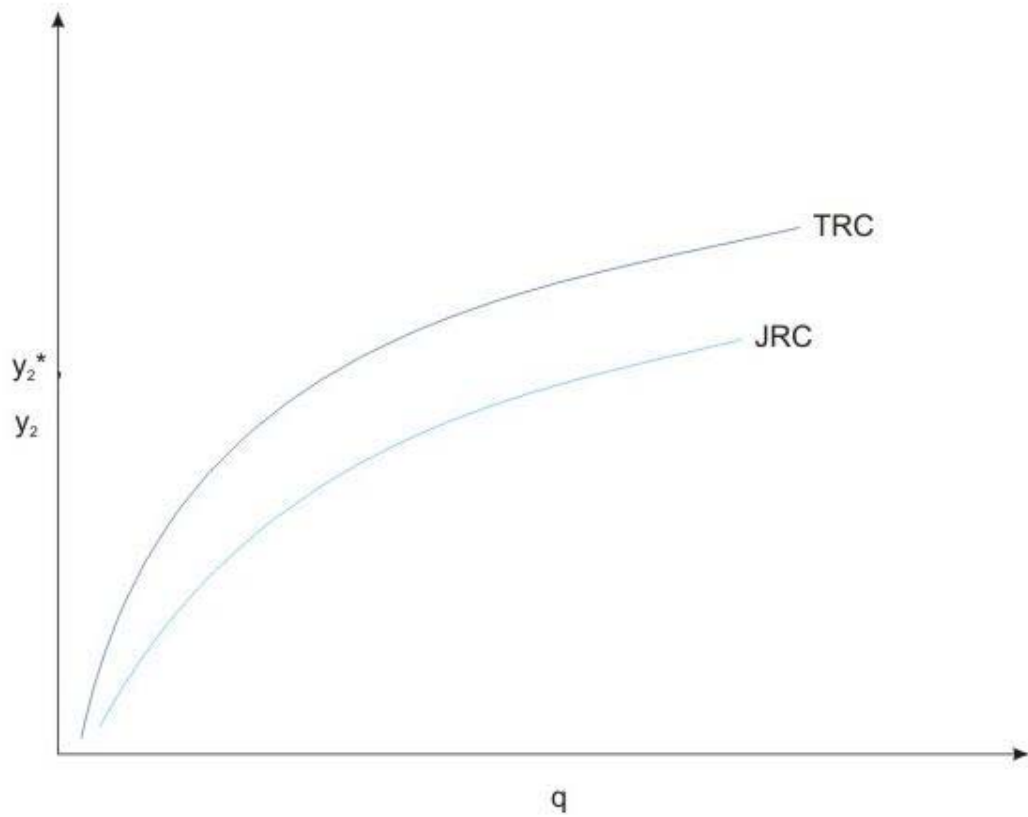
FIGURE 53. TRC above JRC for all discharges

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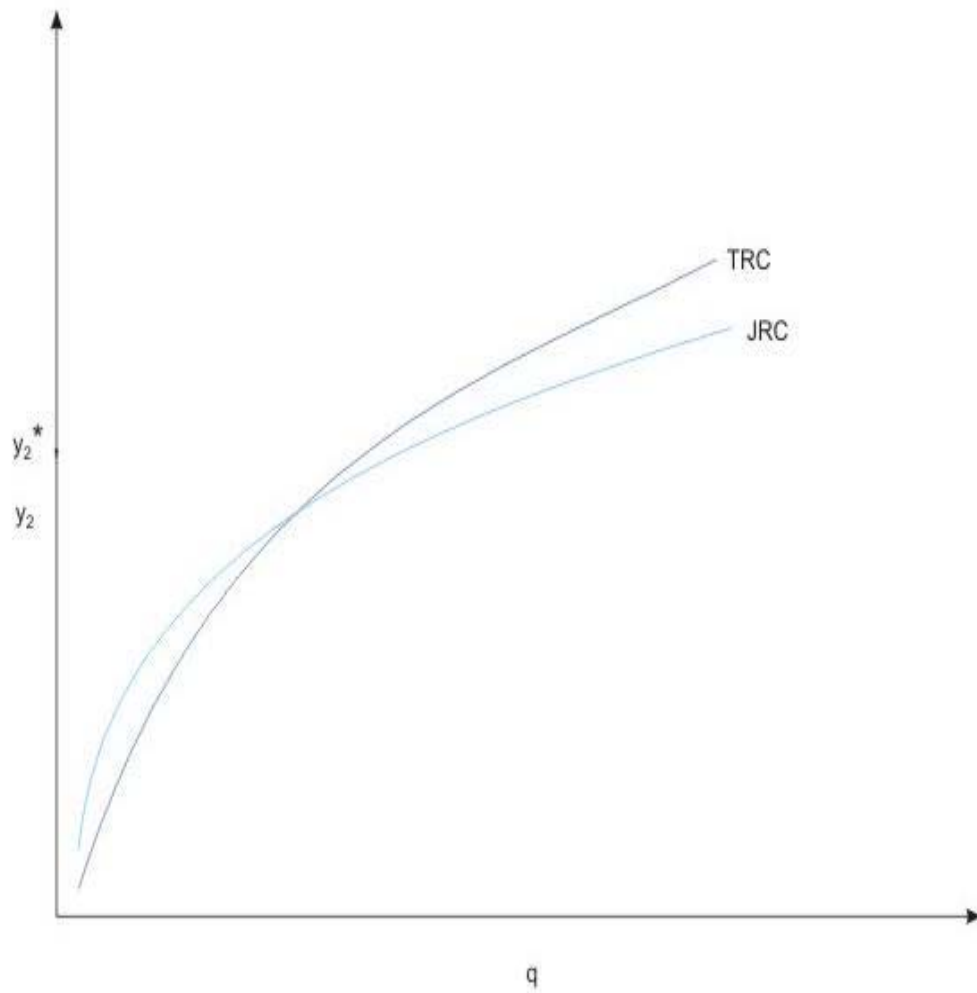
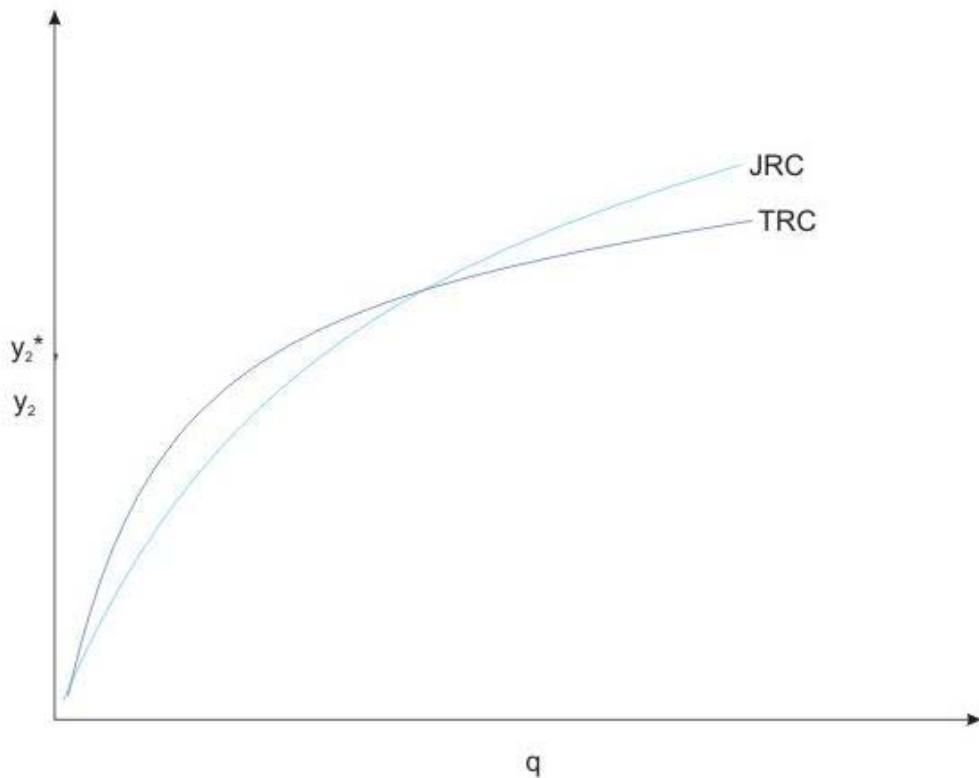FIGURE 54. TRC below JRC for low discharges and above for high discharges

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)
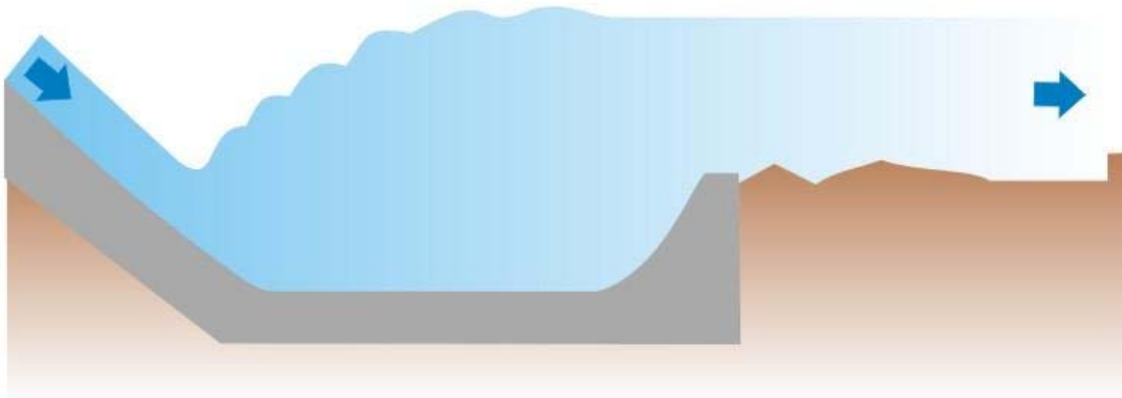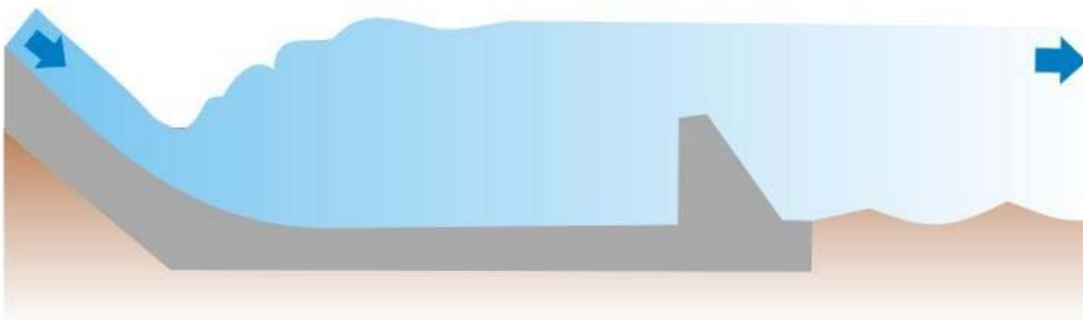
FIGURE 56. Depressed floor of stilling basin apron

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).
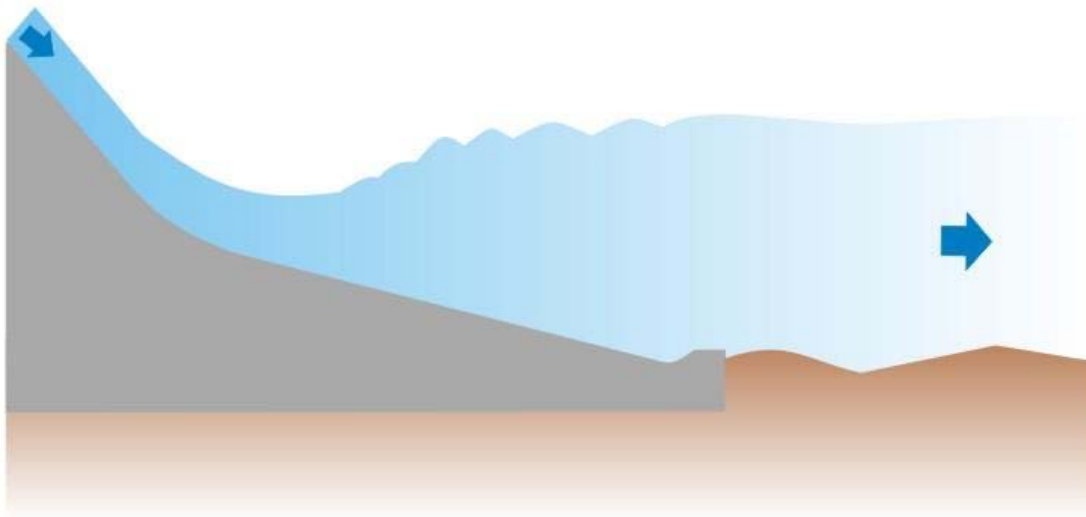
FIGURE 58. Inclined stilling basin

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.

4.8.14   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.
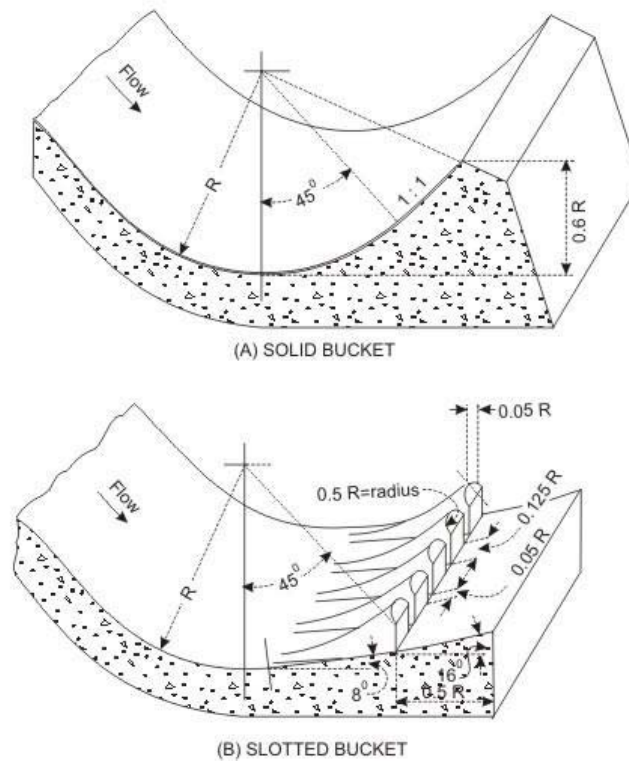
FIGURE 59. Roller buckets ; (A) Solid ; (b) Slotted

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

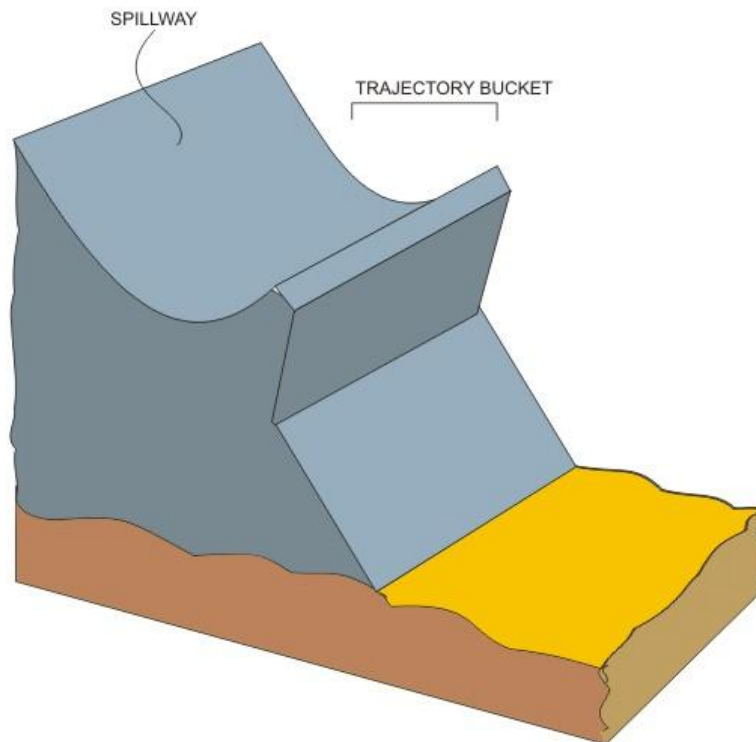An upturn solid bucket is used when the tailwater depth is much in excess of sequent depth and in which dissipation of considerable portion of energy occurs as a result of formation of two complementary elliptical rollers, one in bucket proper, called the surface roller, which is anticlockwise (if the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

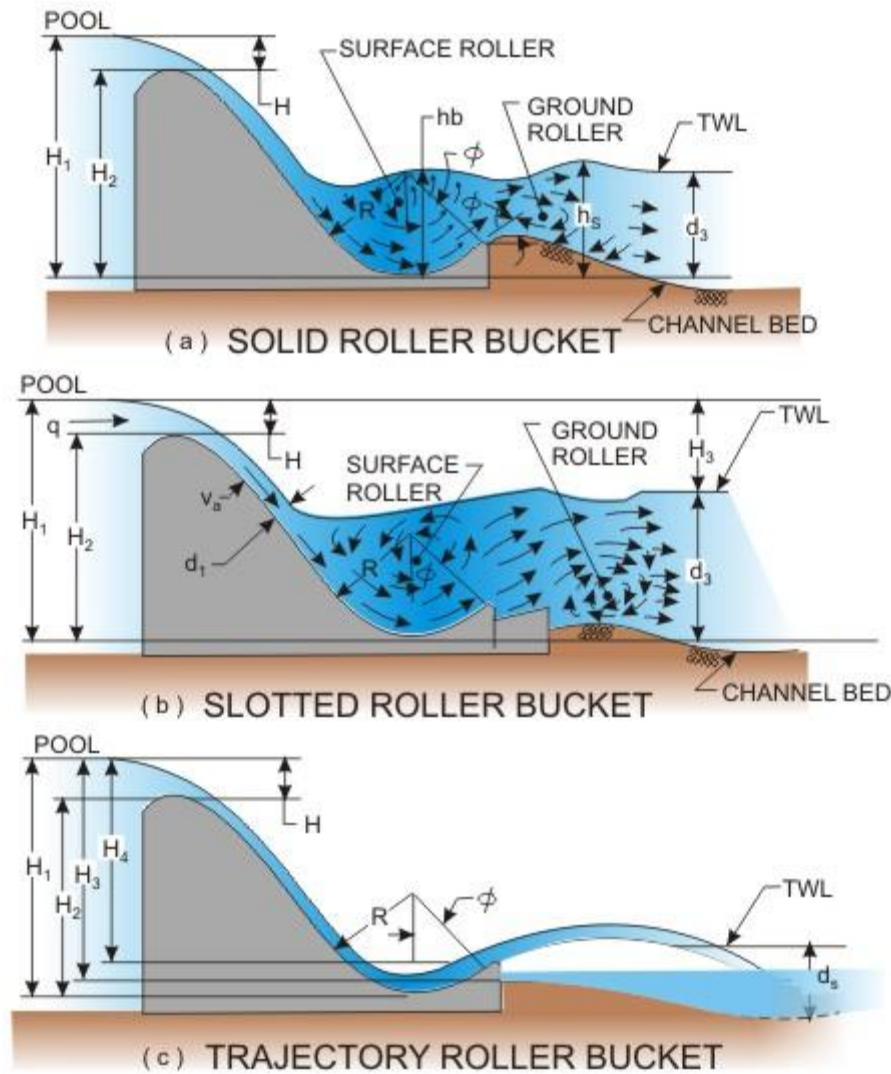The various parameters are shown in Figure 61 (a).

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)
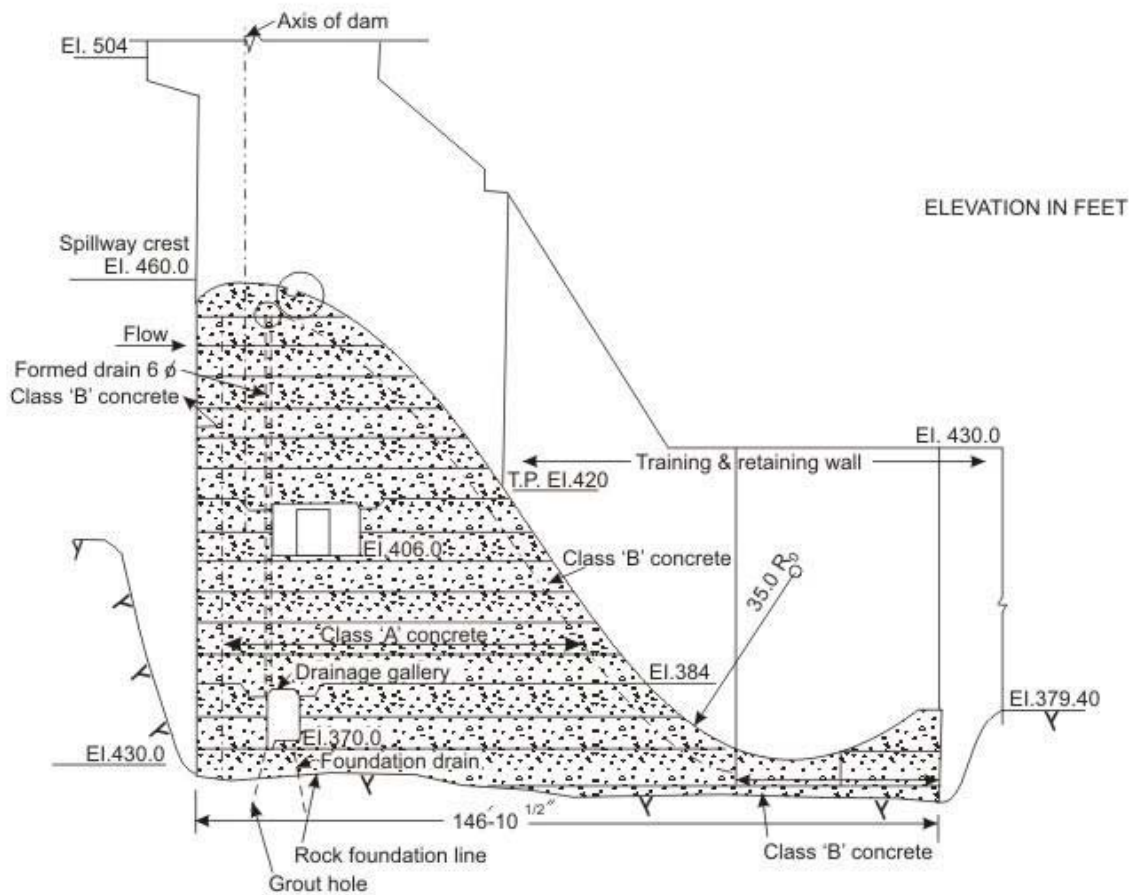

FIGURE 62. Maithon Dam spillway

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).
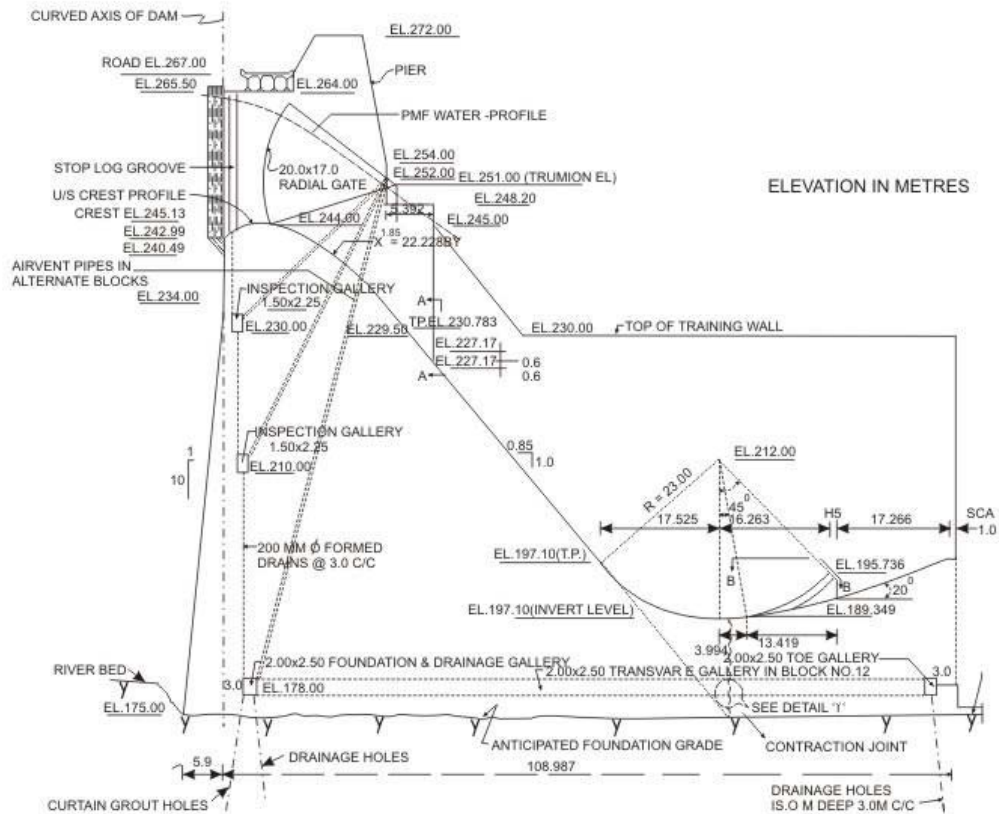
FIGURE 63. Indira Sagar Dam Spillway

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).
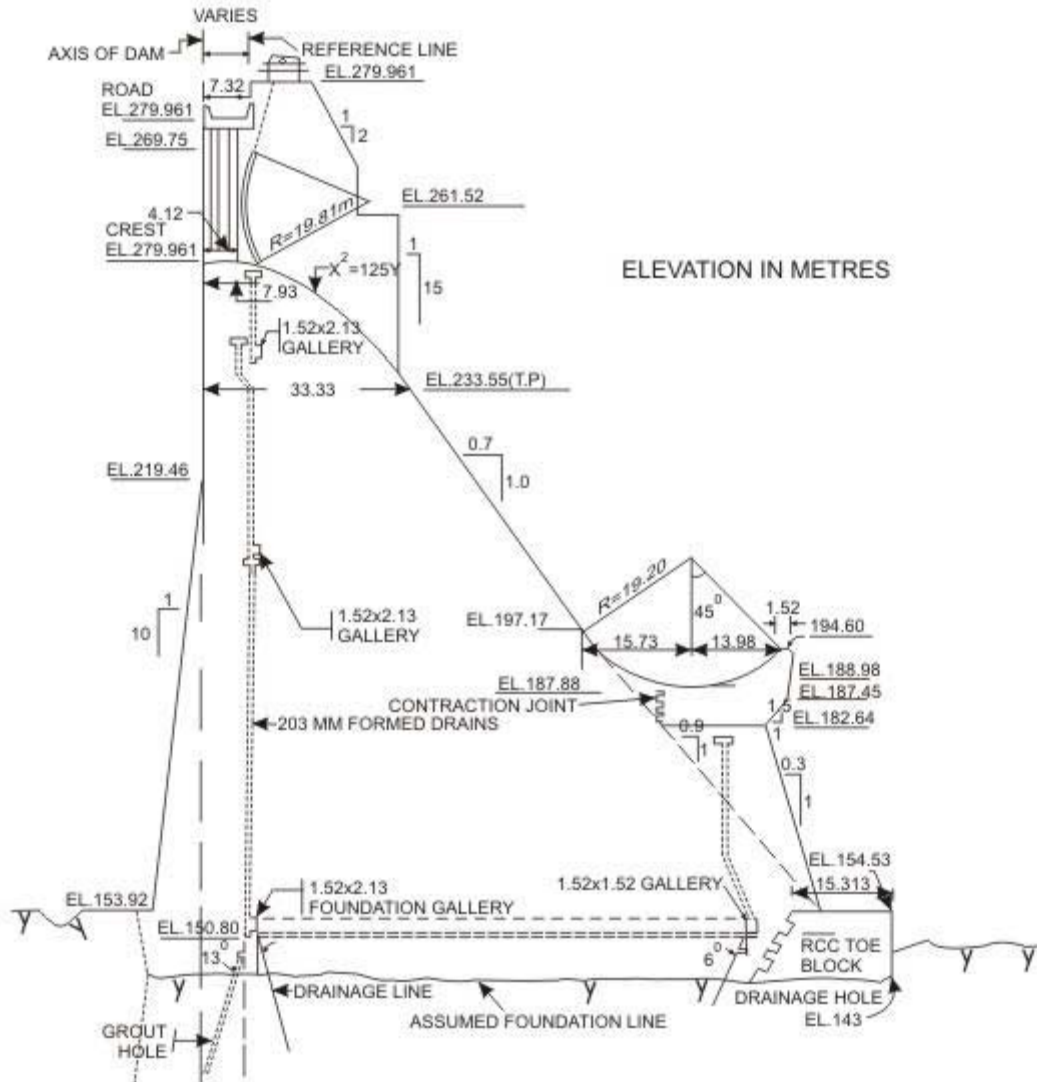
FIGURE 64. Srisailam Dam Spillway

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"

4.8.15   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.
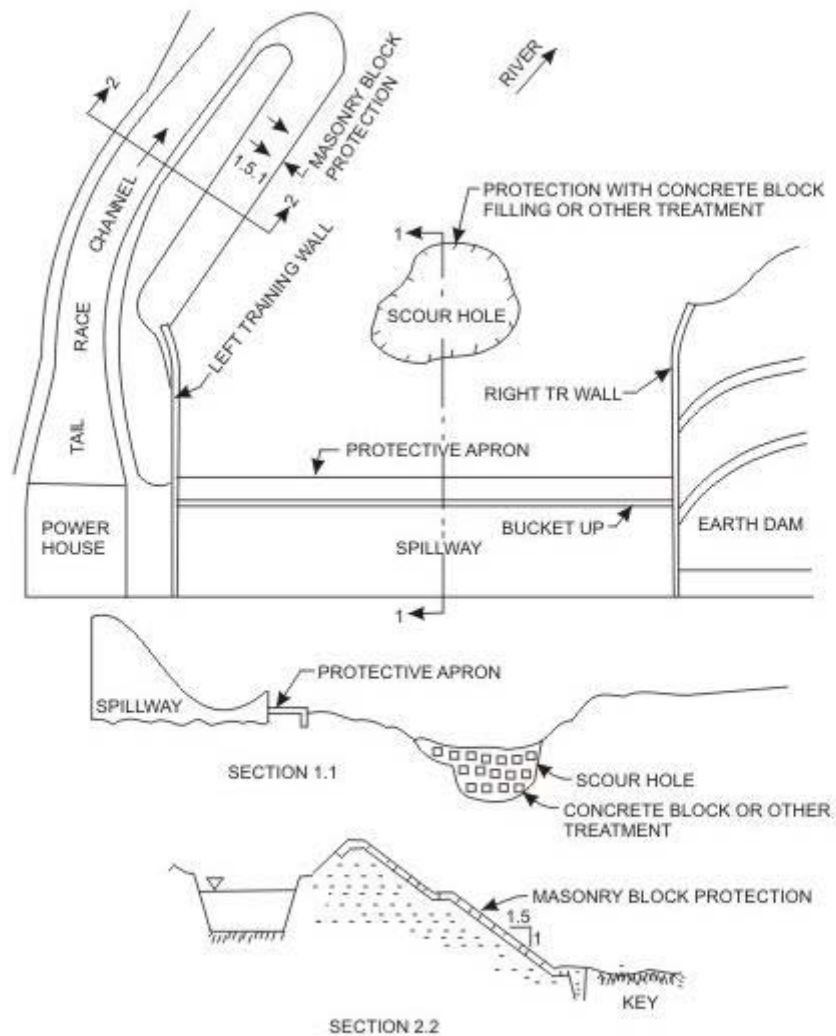
FIGURE 65. Different types of protection works downstream of a typical spillway project
(Courtesy: IS 13195)

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.