# 3. SPILLWAYS

## 3.1 Introduction

Spillways are provided for storage dams to release surplus or flood water, which cannot be contained in the allotted storage space, and at diversion dams to bypass flows exceeding those, which are turned into the diversion system.

There are several spillway designs. The choice of design is a function of the nature of the site, the type of dam and the overall economics of the scheme. The importance of a safe spillway cannot be overemphasized; many failures of dams have been caused by spillway of insufficient capacity. Ample capacity is of paramount importance for earth fill and rock fill dams, which are likely to be overtopped, whereas concrete dams may be able to withstand moderate overtopping. Usually, increase in cost is not directly proportional to increase in capacity. Very often, the cost of a spillway of ample capacity will be only moderately higher than that of one which is obviously too small.

A spillway may be located either within the body of the dam or at one end of the dam or entirely away from the dam as an independent structure.

### 3.1.1Essential Requirements of a Spillway

The essential requirements of a spillway are:

i) The spillway must have sufficient capacity;

ii) It must be hydraulically and structurally adequate;

iii) It must be so located that it provides safe disposal of water, i.e. spillway discharge will not erode or undermine the d/s of the dam;

iv) The bounding surfaces of the spillway must be erosion resistant to withstand the high scouring velocities created by the drop from the reservoir surface to the tail water.

v) Some device will be required for dissipation of energy on the d/s side of the spillway.

1.1 **Spillway Capacity**

The required capacity of a spillway, i.e. the maximum outflow rate through the spillway, may be determined by ***flood routing*** and requires the following data:

i) Inflow hydrograph (plot of rate of inflow vs. time)

ii) Reservoir capacity curve (plot of reservoir storage Vs water surface elevation)

iii) Discharge curve (plot of rate of outflow Vs reservoir water surface elevation).

By flood routing, corresponding to a particular inflow hydrograph, the maximum outflow rate and maximum rise in the water surface may be determined.

However, the required capacity of a spillway depends on the following factors:

i) The inflow flood;

ii) The available storage capacity;

iii) The discharge capacity of other outlet works;

iv) Whether the spillway is gated or ungated;

v) The possible damages if a spillway of adequate capacity is not provided.

The selection of the inflow flood for the spillway design depends on the degree of protection that ought to be provided to the dam, which, in turn, depends on the type of dam, its location, and consequences of failure of the dam.

## 3.1.2 Components Of a Spillway

The following are the main components of a spillway:

**i)** **Control Structure:** Major component, which regulates and controls the outflow from the reservoir. It prevents outflow from a reservoir below a fixed level and allows the flow when the water surface in the reservoir rises above the level. In most of the cases, the control section consists of a weir, which may be sharp crested, ogee, or broad crested. Gates may also be provided on the crest of the control structure to regulate the flow of water from the reservoir.

**ii)** **Discharge channel(or waterway, or conveyance structure):** Its main function is to convey the water safely from the reservoir downward to the river. Located next to the control structure. The conveyance structure may be the d/s face of the spillway, an open channel excavated along the ground surface, a closed conduit placed through or under the dam, or a tunnel excavated through an abutment.

iii) **Terminal structure or energy dissipator:** Provided to dissipate the high energy of flow from spillway before the flow is returned to the river. It is provided on the downstream of the spillway.

iv) **Entrance or approach channel and outlet channel:** Entrance

Channels may be required to draw water from the reservoir and convey it to the control structure. Similarly outlet channels may be required to convey the spillway flow from the terminal structure to the river channel below the dam. The entrance and outlet channels are notrequired where a spillway draws water directly from the reservoir and delivers it directly back into the river; e.g. overflow spillway. However, in the case of spillways placed through abutments or through saddles or ridges, the entrance and outlet channels may be required.

### 3.1.3 Types Of Spillway

Spillways may be classified:

1. **According to their function** (or based on the time when the spillway comes into operation) as

(a) **Service (or main) spillways:** Designed for frequent use in conveying flood releases from the reservoir to a watercourse downstream from a dam. It is designed to pass the entire design flood.

(b) **Auxiliary Spillways: - Designed** for infrequent use and may sustain limited damages when used. Some damages of the structure from passage of infrequent flood is permissible. It is provided as a supplement to the main spillway and its crest is so located that it comes into operation only after the floods for which the main spillway is designed are exceeded. It is provided in conjunction with the main spillway. The total capacity of the spillway is then equal to the sum of the capacities of the main and auxiliary spillways.

(c) **Emergency spillways: -** Designed to provide a reserve protection against overtopping of a dam and are intended for use under extreme conditions, such as mis-operation or malfunction of a service spillway or other emergency conditions. Under normal reservoir operation, emergency spillways are never required to function. The control crest is, therefore, placed at or above the designed maximum reservoir water surface.

Some of the situations, which may lead to emergency, are:

a) an enforced shut down of outlet works,
b) a malfunctioning of spillway gates,
c) The necessity for bypassing the regular spillway because of damage or failure of some part of that structure.

2. **According to Mode of Control** as:
   (a) Free (or uncontrolled) spillways,
   (b) Gated (or controlled) spillways.
3. **Based on prominent features pertaining to the various components of the spillway** (or according to hydraulic criteria) as:
   (a) Free over fall or straight drop spillway,
   (b) Overflow or ogee spillway,
   (c) Chute or open channel or Trough spillway,
   (d) Side channel spillway,
   (e) Siphon spillway,
   (f) Shaft or Morning Glory spillway,
   (g) Conduit or tunnel spillway.
   (h) Labyrinth spillway
   (i) Cascade or stepped spillways

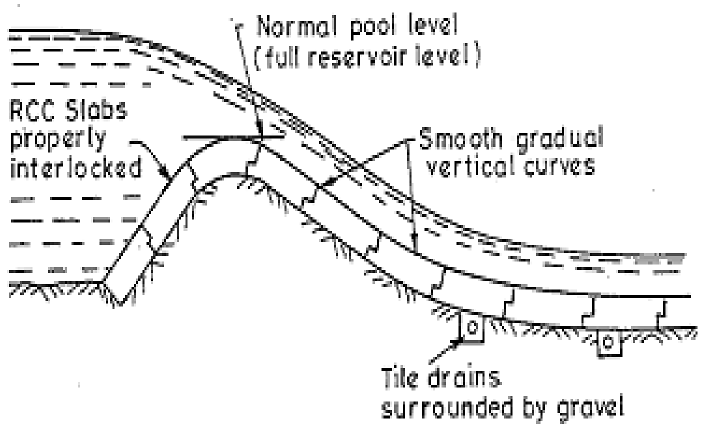



Fig. 21.18. (*d*) Section through a Chute Spillway.

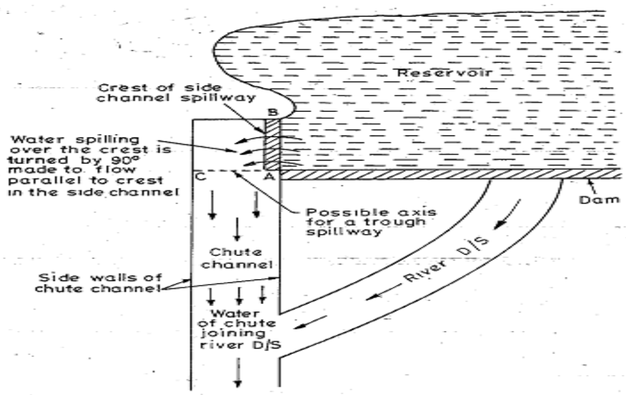



Fig. 21.22. (*a*) Simplified line sketch of a Side Channel Spillway.

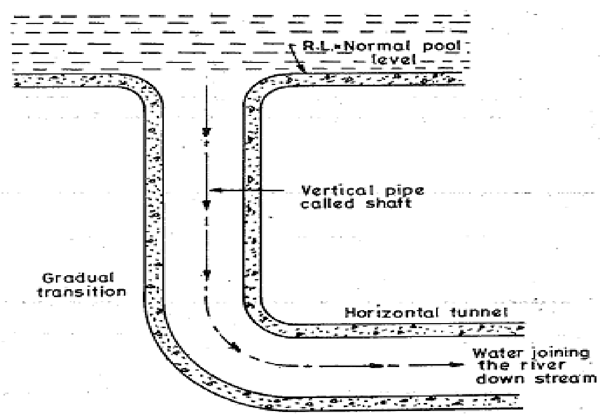



Fig. 21.23. Shaft Spillway.

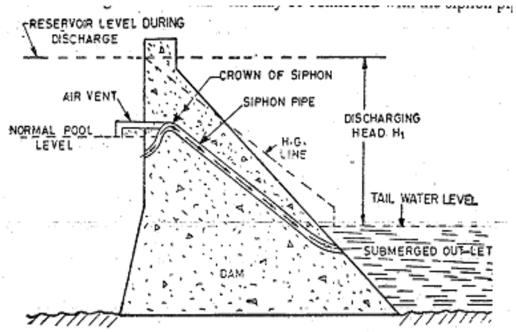


Fig. 21.25. Siphon pipe installed within the gravity dam.

### 3.1.4 Free Over fall Or Straight Drop Spillway

This is the simplest type of spillway, which is constructed in the form of low height weir having d/s face either vertical or nearly vertical. Water drops freely from the crest, and the underside of the falling nappe is ventilated sufficiently to prevent a pulsating, fluctuating, jet. Occasionally, the crest is extended in the form of an overhanging lip to direct the small discharge away from the face of the overfall section.

Since vacuum gets created in the underside portion of the falling jet, sufficient ventilation of the nappe is required in order to avoid pulsating and fluctuating effects of the jet.

If no artificial protection is provided on the d/s side of the over-fall section, the falling jet usually causes the scouring of the stream bed and will form a deep plunge pool.

The free over fall spillway is suitable for thin arch dams and for those dams with nearly vertical downstream face and would permit free fall of water. Free over-fall spillways are used where the hydraulic drops from head pool to TW are not in excess of about 6m.

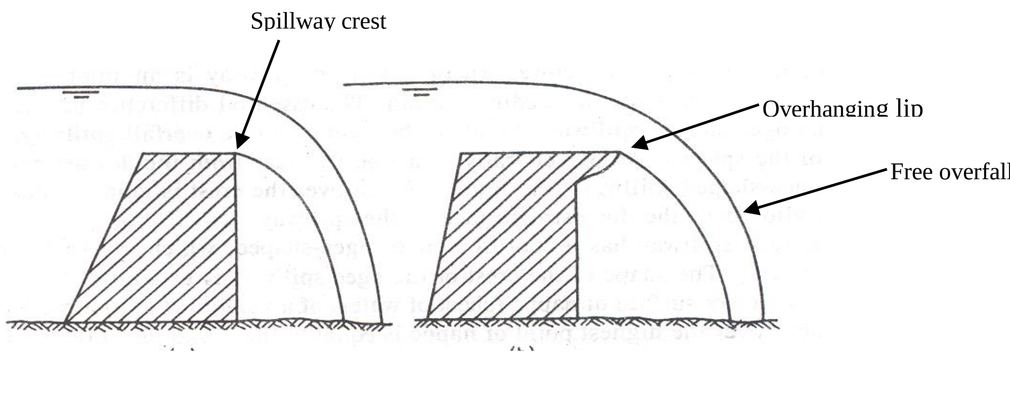



**Fig 1.1 Straight drop spillway**

### 3.1.5 Overflow (or Ogee) Spillways

Overflow spillways are by far the most widely adopted. They are mainly used on masonry or concrete dams, and if used with earth fill and need a separate concrete structure.

An overflow spillway is an improvement upon the free overfall spillway. The essential difference between the free overfall spillway and the overflow spillway is that in the case of the former the water flowing over the crest of the spillway drops as a free jet clearly away from the downstream face of the spillway, while in the case of the latter the water is guided smoothly over the crest of the spillway and is made to glide over the downstream face of the spillway.

#### 3.1.5.1 Crest Shape Of Overflow Spillway

The shape of the crest or the upper curve of the ogee profile of this spillway is made to conform closely to the profile of the lower surface of the nappe (or lower nappe) or sheet of water flowing over a ventilated sharp-crested weir when discharging at a head equal to the design head of the spillway.

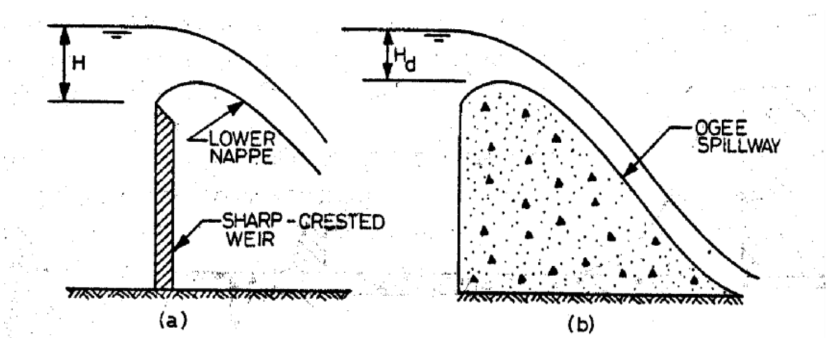



**Fig 1.2** Crest shape of overflow spillway

At the design head ($H = H_d$) the water flowing over the crest of the spillway will remain in contact with the surface of the spillway as it glides over it and optimum discharge will occur. In this case no pressure is exerted on the spillway by the flowing water, as there will be atmospheric pressure along the contact surface between the flowing water and the spillway.

At head less than the design head ($H < H_d$) the overflowing water will remain in contact with the surface. The natural trajectory of the nappe falls below the profile of the spillway crest, then there will therefore be positive gage pressures over the crest, as the nappe tends to be depressed. In this case, as the spillway is supporting a sheet of flowing water backwater effect will be created and the discharge will be reduced.

At a head greater than the design head ($H < H_d$), the nappe trajectory is higher than the crest profile, and the overflowing water tends to break contact with the spillway surface and zone of separation will be formed in which negative or suction pressure will be produced. The effect of negative pressure will be to increase the effective head and thereby increase the discharge. This may result in cavitation. However, in practice, this pressure reduction is not normally a serious problem unless $H > 1.5\ H_d$. Indeed recent work suggests that separation will not occur until H approaches $3\ H_d$.

### 3.1.5.2 Design Of Crest Of Ogee Spillway

The shape of the nappe shaped profile depends upon the head, the inclination of the upstream face of the spillway and the height of the spillway above the streambed or the bed of the entrance channel (which influences the velocity of approach to the crest of the spillway).

Several standard ogee shapes have been developed by U.S. Army Corps of Engineers at their Waterways Experimental Station (WES). Such shapes are known as 'WES' standard spillway shapes. The downstream profile can be represented by:

$$\mathbf{X^n = KH_d^{n-1} y} \tag{6.1}$$

Where: x, y = Co-ordinates of the points on the crest profile with the origin at the highest point of the crest called APEX.

$H_d$ = Design head excluding head due to velocity of approach,

K, n = Constants depending on the slope of the upstream face.

The crest equation gives the crest shape downstream from the origin of coordinates. This equation is applicable to positive values of x and y.

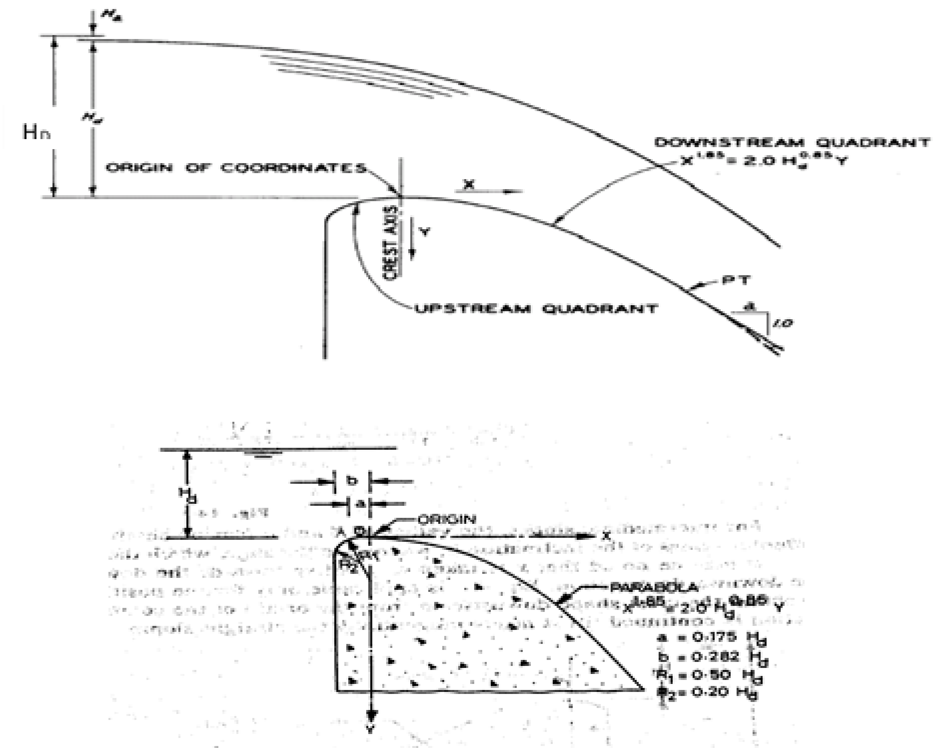



**Fig 6.3**WES- standard spillway shape (vertical upstream face)

The following table gives values of K, n and other constants and crest equations

| U/s Face slope | K | n | $\frac{a}{H_d}$ | $\frac{b}{H_d}$ | $\frac{R_1}{H_d}$ | $\frac{R_2}{H_d}$ | Crest Equation |
|---|---|---|---|---|---|---|---|
| Vertical | 2.000 | 1.850 | 0.175 | 0.282 | 0.200 | 0.500 | $x^{1.85} = 2H_d^{0.85}y$ |
| 1H: 3v | 1.936 | 1.836 | 0.139 | 0.237 | 0.210 | 0.680 | $x^{1.836} = 1.936H_d^{0.836}y$ |
| 2H: 3v | 1.936 | 1.810 | 0.115 | 0.214 | 0.220 | 0.480 | $x^{1.810} = 1.939H_d^{0.810}y$ |
| 3H: 3v | 1.873 | 1.776 | 0.000 | 0.119 | $\propto$ (Straight line) | 0.450 | $x^{1.776} = 1.873H_d^{0.776}y$ |

According to U.S. Army Corps of Engineers, the u/s curve of the ogee spillway (u/s of origin, though in the form of compound circular curve) having a vertical u/s face, should have the following equation:

$$\mathbf{y = \frac{0.724(x+0.27H_d)^{1.85}}{H_d^{0.85}} + 0.126H_d - 0.4315H_d^{0.375}(x+0.27H_d)^{0.625}} \qquad (6.2)$$

Where the upstream profile extends up to $\mathbf{x = -027H_d}$

The corresponding y value is equal to **0.126 Hd**.

The curved profile of the crest section is continued tangentially along the straight sloping surface, which forms the d/s face of the spillway. The location of the point of tangency (P.T) depends on the slope of the straight portion of the d/s face of the spillway, which in turn depends on the stability requirements and on the features of the stilling basin at toe of the spillway. The slope of the straight portion varies between 1V: 0.6H to 1V: 0.8H. At the end of the sloping surface a curved bucket is provided to create a smooth transition of flow from the spillway to the outlet channel or the river on the d/s side and prevent scoring.

The approximate radius R of the bucket may be obtained from (empirical)

$$R = 10^{(v+6.4H+4.88)/(3.6H+19.52)} \qquad (6.3)$$

V= velocity of flow at to e of spillway [m/s

H = head excluding head due to velocity of approach (m)

Neglecting energy loss over the spillway, velocity of flow v at the toe will be

$$\mathbf{V = \sqrt{2g(Z + Ha - y}} \qquad (6.4)$$

Where Z = the fall, m

$H_a$ = head due to velocity of approach, m

y = depth of flow at the toe, m

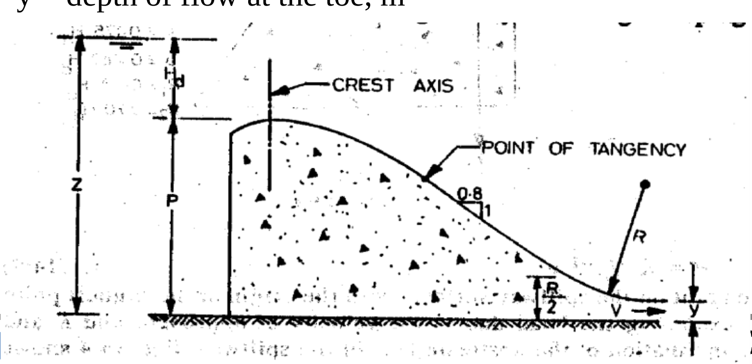


**Figure 1.4** Profile of an ogee spillway

### 1.3 Discharge of Overflow Spillway

The discharge over an overflow spillway is given by

$$Q = CL_e H_D^{3/2} \tag{6.5}$$

Where Q = discharge, $m^3/s$

C = coefficient of discharge

$L_e$ = effective length of crest of spillway, m

$H_D$ = total head over the crest including that due to velocity of approach.

$H_D = H_d + H_a$

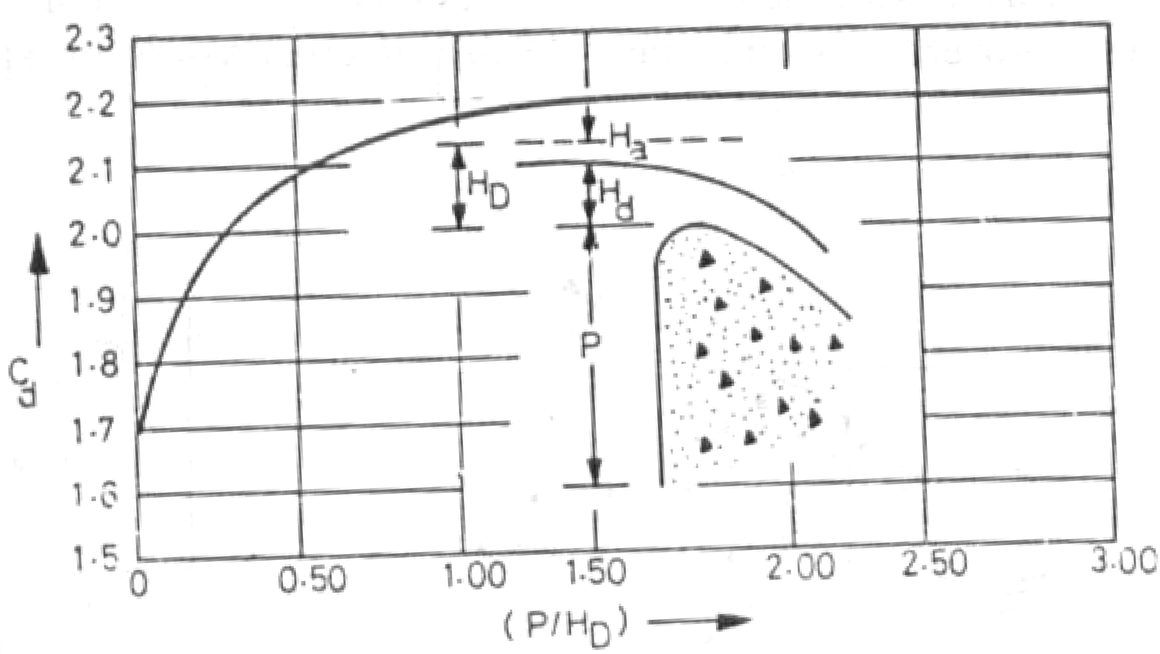
For high ogee spillway $H_a$ is very small, and $H_D \approx H_d$

#### (i) Coefficient of discharge, C, of Overflow spillway

An overflow spillway has a relatively high coefficient of discharge the maximum value of which may be about 2.2 if no negative or suction pressure is allowed to develop. Its value depends on the following factors:

a) Depth of approach, p
b) Heads differing from design head
c) Upstream face slope
d) Downstream apron interference and downstream submergence

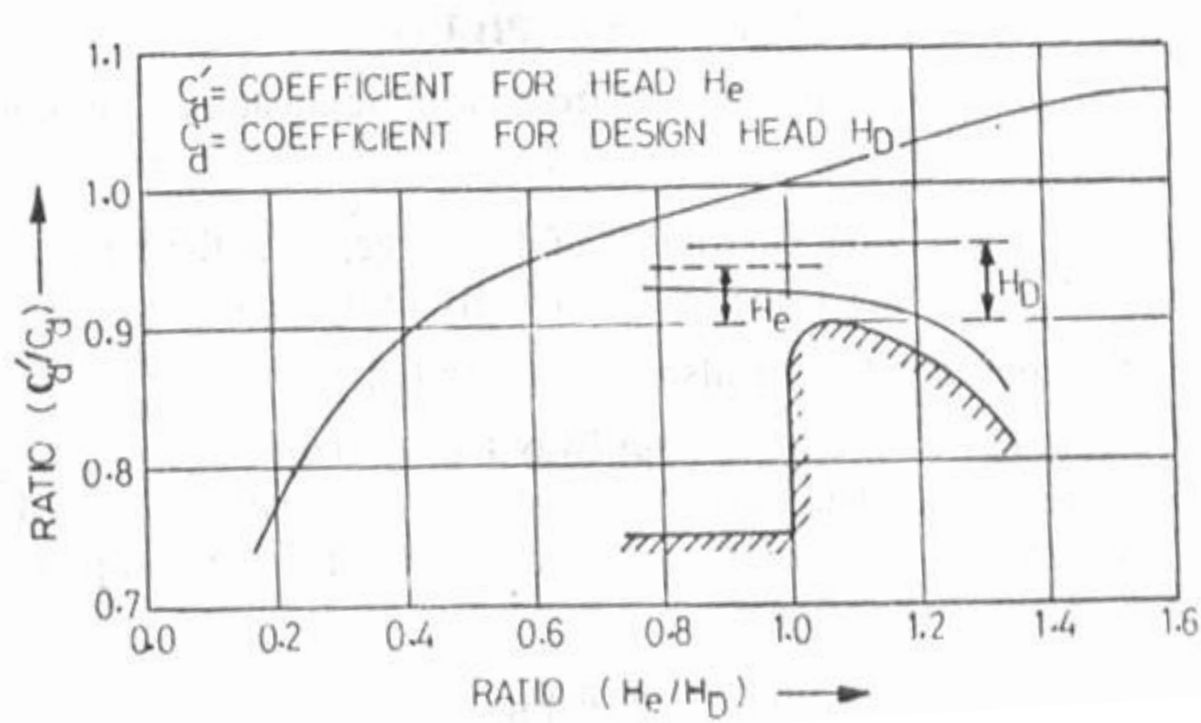
**a)Effect of Depth of Approach:** With increase in the height of spillway the velocity of approach decreases and the coefficient of discharge increase. Model tests have shown that the effect of approach velocity is negligible when the height of the spillway above the streambed is equal to or greater than 1.33 $H_d$ ($P \geq 1.33\ H_d$) where $H_d$ is the design head excluding the head due to velocity of approach




A plot of C versus $P/H_D$ is shown below, where $H_D$ is the design head including head due to velocity of approach (i.e. $H_D = H_d + H_a$). It may be observed from this plot that there is a marked increase in the value of C till the height of the spillway (P) becomes equal to twice the design head $H_D$. With further increase in P there is no much increase in the value of C.

**Fig 1.5** Plot of coefficient of discharge versus ($P/H_D$)

**(b) Effect of heads differing from the design head:** The plot of (C/C’) versus ($H_e/H_D$) for a spillway of height P above stream bed greater than 1.33 $H_d$, where C is




coefficient of discharge corresponding to the actual head of flow H and C’ is the coefficient of discharge corresponding to the design head $H_D$. It may be observed from this plot that with increase in the value of ($H_e/H_D$) the value of (C/C’) increases. In other words, with increase in the head H the coefficient of discharge increases. However, for $H_e < H_D$, C < C’; and for $H_e > H_D$, C > C’.

Fig 1.6 plot of C/C' Vs ($H_e/H_D$)

Since for heads of flow higher than the design head higher will be the coefficient of discharge, if the spillway crest is designed by assuming a lower design head, for most of the range of heads of flow higher coefficient of discharge will be obtained.

However, the design head should not be less than about 80% of the maximum head in order to avoid the possibility of cavitation.

Model tests have shown that for P > 1.33 $H_d$ the head due to velocity of approach is negligible and when the total head of flow is equal to the design head, i.e. $H_e = H_D$, the coefficient of discharge is equal to 2.2.

When the actual operating head is less than the design head, the prevailing coefficient of discharge, C, tends to reduce, and is given by

$$C = C'\left(\frac{H_e}{H_D}\right)^{0.12} \tag{6.6}$$

Where $H_D$ = design head including velocity head and C' = 2.2

**(c) Effect of upstream face slope:** For small values of the ratio ($P/H_D$) a spillway with sloping upstream face has a higher coefficient of discharge than a spillway with vertical upstream face. However, for large values of the ratio ($P/H_D$) the coefficient of discharge for spillways with sloping upstream face tends to decrease.

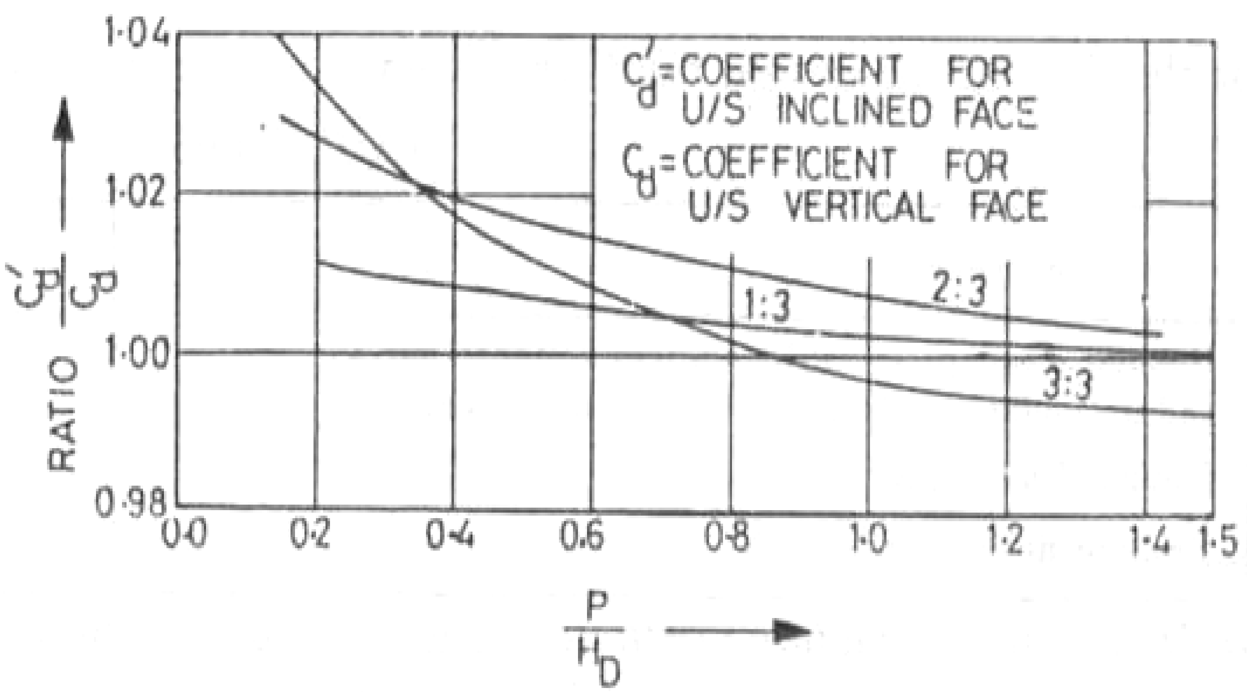



**Fig 1.7** Coefficient of discharge for various u/s slopes

**(d) Downstream apron interface and submergence effects:** The coefficient of discharge is reduced due to submergence. When the tailwater level is such that the top of the weir is covered by it, such that the weir cannot discharge freely; the weir is then said to be submerged weir. Where the hydraulic jump occurs, the coefficient of discharge may decrease due to backpressure effect of the downstream apron and is independent of the submergence effect. When the value of $\frac{h_d + d}{H_D}$ exceeds 1.7, the downstream apron is found to have negligible effect on the coefficient of discharge. But there may be a decrease in C due to tail water submergence.

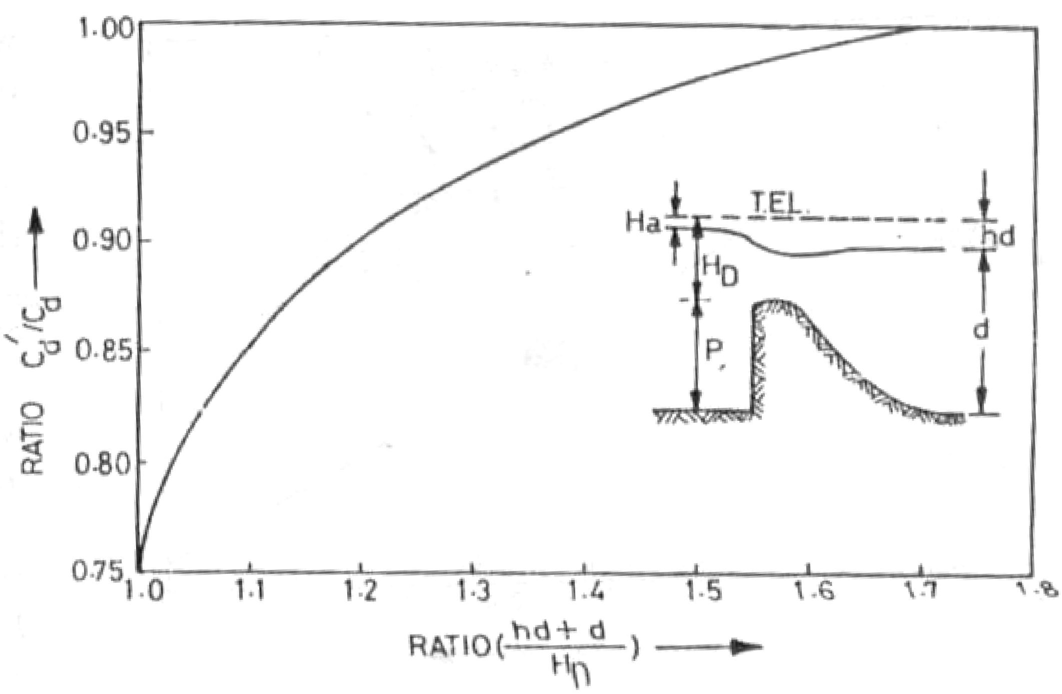



**Fig 1.8** Maximum TW depth for a non-submerged weir

**(e) Effective Length Of Crest Of Overflow Spillway:** The effective length of an overflow spillway is given by

$$L_e = L - 2 (NK_p + Ka) H_D \quad (6.7)$$

Where $L_e$ = effective length of crest

L =net length of crest which is equal to the sum of the clear spans of the gate bays between piers

$H_D$ = total head on crest including velocity head

N = number of Piers

$K_P$ = Pier contraction coefficient

$K_a$ = abutment contraction coefficient

The pier contraction coefficient, $K_p$ depends on

i) Shape and location of pier nose;

ii) Thickness of pier;

iii) Velocity of approach; and

iv) Ratio of actual head to design head.

For flow at design head the average values of $K_p$ may be assumed as follows:

**Pier coefficients, $K_p$:**

1. Square nosed piers with corners rounded on a radius equal to about 0.1 of pier thickness → $K_p$ =0.02
2. Round-nosed piers → $K_p$ = 0.01
3. Pointed nose piers → $K_p$ = 0.00

**The abutment contraction coefficient $K_a$ depends on:**

i) Shape of abutment;

ii) Angle between upstream approach wall and axis of flow;

iii) Approach velocity; and

iv) Ratio of actual head to design head

For flow at design head, average value of $K_a$ may be assumed as follows:

**Abutment coefficients, $K_a$:**

1. Square abutment with head wall at $90^0$ to the direction of flow → $K_a$= 0.20
2. Rounded abutment with head wall at $90^0$ to the direction of flow, when $0.5\ H_d \geq r \geq 0.15\ H_d$ → $K_a$ = 0.10
3. Rounded abutments where $r > 0.5\ H_d$ and headwall is placed not more than $45^0$ to the direction of flow → $K_a$ = 0.00.

Where r = radius of abutment rounding

$H_d$ = design head.

**Example 21.1.** *Design a suitable section for the overflow portion of a concrete gravity dam having the downstream face sloping at a slope of 0.7 H : 1 V. The design discharge for the spillway is 8,000 cumecs. The height of the spillway crest is kept at RL 204.0 m. The average river bed level at the site is 100.0 m. The spillway length consists of 6 spans having a clear width of 10 m each. Thickness of each pier may be taken to be 2.5 m.*

**Solution.** Since the given spillway looks like a high weir, the coefficient of discharge may be assumed to be 2.2.

Now $$Q = C \cdot L_e H_e^{3/2}$$

where $L_e = L - 2\,[N K_p + K_a]\, H_e$

Let us first work out the approximate value of $H_e$ for a value of

$$L_e \approx L = \text{clear waterway} = 6 \times 10 = 60 \text{ m.}$$

$\therefore$ $$8{,}000 = 2.2 \times 60\, H_e^{3/2}$$

or $$H_e^{3/2} = \frac{8{,}000}{2.2 \times 60} = 60.6$$

or $$H_e = (60.6)^{2/3} = 15.5 \text{ m.}$$

The height of the spillway above the river bed (see Fig. 21.15)

$$= h = 204 - 100 = 104.0 \text{ m}$$

Since $\frac{h}{H_d}$, *i.e.* $\frac{104}{15.5} > 1.33$,

it is a high spillway, the effect of velocity head can, therefore, be neglected.

Since $\frac{h_d + d}{H_e} = \frac{H_e + h}{H_e} = \frac{15.5 + 104}{15.5} > 1.7$ ;

the discharge coefficient is not affected by tail water conditions, and the spillway remains a high spillway.

**U/s Slope.** The upstream face of the dam and spillway is proposed to be kept vertical. However, a batter of 1 : 10 will be provided from stability considerations in the lower part. This batter is small and will not have any effect on the coefficient of discharge.

**Effective length of spillway** ($L_e$) can now be worked out as

$$L_e = L - 2\,[N.K_p + K_a]\,H_e$$

Assuming that 90° cut water nose piers and rounded abutments shall be provided, we have

$$K_p = 0.01$$

and $$K_a = 0.1$$

No. of piers $= N = 5$.

Also assuming that the actual value of $H_e$ is slightly more than the approximate value worked out (*i.e.* 15.5 m), say, let it be 16.3 m, we have

$\therefore \quad L_e = 60 - 2\,[5 \times 0.01 + 0.1] \times 16.3 = \mathbf{55.1\ m.}$

Hence $\quad Q = 2.2 \times 55.1 \times H_e^{3/2}$

or $\quad 8{,}000 = 2.2 \times 55.1 \times H_e^{3/2}$

or $\quad H_e^{3/2} = \dfrac{8{,}000}{2.2 \times 55.1} \cong 66.0$

or $\quad H_e = (66.0)^{2/3} = \mathbf{16.4\ m} \cong 16.3$ (assumed)

Hence, the assumed $H_e$ for calculating $L_e$ is all right. The crest profile will be designed for $H_d = 16.4$ m (neglecting velocity head).

**Note.** The velocity head ($H_a$) can also be calculated as follows :

$$\text{Velocity of approach} = V_a = \frac{8{,}000}{(60 + 5 \times 2.5)\,(104 + 16.4)}$$

$$= \frac{8{,}000}{72.5 \times 120.4} = 0.917 \text{ m/sec.}$$

$$H_a = \text{Velocity Head} = \frac{V_a^2}{2g} = \frac{(0.917)^2}{2 \times 9.81} = 0.043 \text{ m.}$$

This is very small and was, therefore, neglected.

*Downstream profile.* The W.E.S. d/s profile for a vertical u/s face is given by equation (21.2) as :

$$x^{1.85} = 2 \cdot H_d^{0.35} \cdot y$$

$$y = \frac{x^{1.85}}{2\,(H_d)^{0.85}} = \frac{x^{1.85}}{2 \times (1.64)^{0.85}}$$

or $$y = \frac{x^{1.85}}{2 \times 10.8}$$

or $$y = \frac{x^{1.85}}{21.6}$$

Before we determine the various co-ordinates of the d/s profile, we shall first determine the tangent point.

The d/s slope of the dam is given to be 0.7 $H$ : 1 $V$.

Hence, $\frac{dy}{dx} = \frac{1}{0.7}$

Differentiating the equation of the d/s profile w.r.t. to $x$, we get

$$\frac{dy}{dx} = \frac{1.85x^{1.85-1}}{21.6} = \frac{1}{0.7}$$

or $$x^{0.85} = \frac{21.6}{1.85 \times 0.7} = 16.7$$

or $$x = 22.4 \text{ m.}$$

$$\therefore \quad y = \frac{(22.4)^{1.85}}{21.6} = 14.6 \text{ m.}$$

The co-ordinates from $x = 0$ to $x = 22.4$ m are worked out in Table

| $x$ metres | $y = \frac{x^{1.85}}{21.6}$ metres |
|---|---|
| 1 | 0.046 |
| 2 | 0.166 |
| 3 | 0.354 |
| 4 | 0.60 |
| 5 | 0.905 |
| 6 | 1.274 |
| 7 | 1.710 |
| 8 | 2.162 |
| 9 | 2.684 |
| 10 | 3.240 |
| 12 | 4.575 |
| 14 | 6.020 |
| 16 | 7.88 |
| 18 | 9.74 |
| 20 | 11.85 |
| 22 | 14.35 |
| 22.4 | 14.60 |

$$y = \frac{0.724\,(x + 0.27\,H_d)^{1.85}}{H_d^{0.85}} + 0.126\,H_d - 0.4315\,H_d^{0.375}\,(x + 0.27\,H_d)^{0.625}$$

Using $H_d = 16.4$ m, we get

$$y = \frac{0.724\,[x + 0.27 \times 16.4]^{1.85}}{(16.4)^{0.85}} + 0.126\,(16.4)$$
$$- 0.4315\,(16.4)^{0.375}\,(x + 0.27 \times 16.4)^{0.625}$$

or $$y = 0.07\,(x + 4.44)^{1.85} + 2.07 - 1.234\,(x + 4.432)^{0.625} \quad \text{...(21.8)}$$

This curve should go upto $x = -0.27\,H_d$

or $$x = -0.27 \times 16.4 = -4.443 \text{ m}.$$

Various values of $x$ such as, $x = -0.5, x = -1.0, x = -2.0, x = -3.0, x = -4.0, x = -4.443$ are substituted in equation (21.8) and corresponding values of $y$ are worked out, as given below in Table 21.6.

**Table 21.6**

| *x in metres* | *y in metres* |
|---|---|
| − 0.5 | 0.020 |
| − 1.0 | 0.063 |
| − 2.0 | 0.27 |
| − 3.0 | 0.65 |
| − 4.0 | 1.34 |
| − 4.443 | 2.07 |

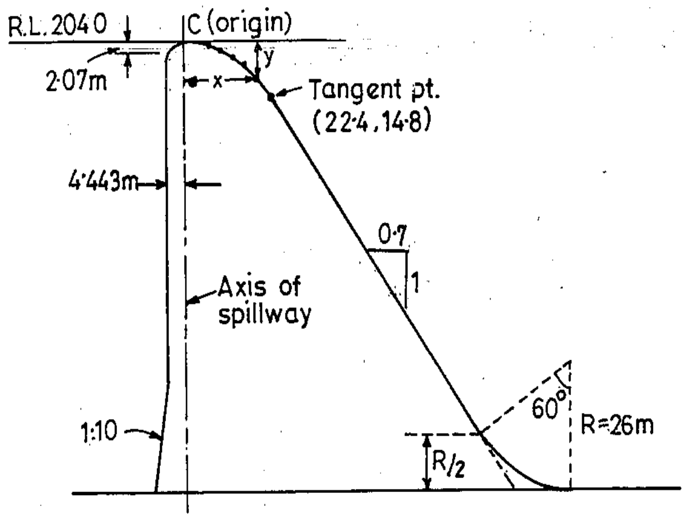
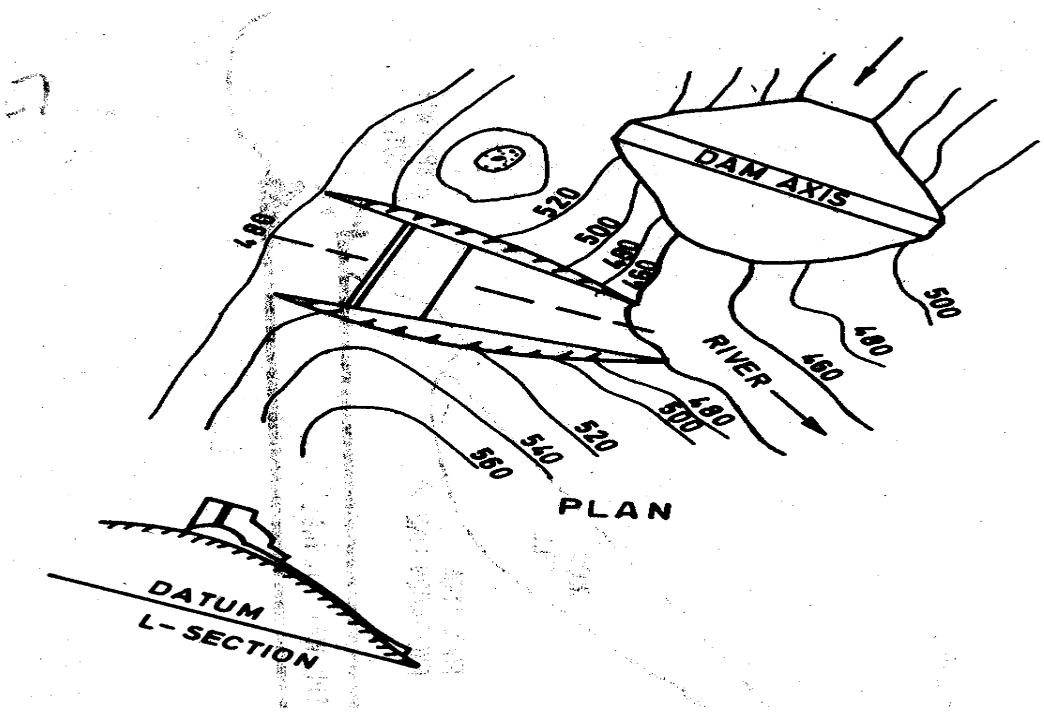



Fig. 21.17

The profile of the spillway has been determined and plotted in Fig. 21.17. A reverse curve at the toe with a radius equal to $\frac{h}{4} = \frac{104}{4} = 26$ m can be drawn at angle 60°, as shown in Fig. 21.17. Aeration pipes (say 25 mm pipes at 3 m c/c) can be installed along the spillway face below the gate lip, so as to prevent the development of negative

## Chute Spillway or Trough Spillway

An ogee spillway is mostly suitable for concrete Gravity dam when the spillway is located within the body of dam. For Earth & Rock-fill dam, a separate spillway is generally constructed in a flank or saddle, away from main valley. Sometimes even for gravity dams a separate spillway is required because of the narrowness of the valley. In such circumstances a separate spillway may have to be provided. The trough spillway or chute spillway is the simplest type of spillway which can be easily provided independently and at low costs. It is lighter & adoptable to any type of foundation and hence provided easily on Earth & Rock-Fill dam. It is also called at times **Waste Weir.** If it is constructed in continuation of the dam at one end, it may be called a **Flank weir**. If it is constructed in a natural saddle in the bank of the river separated from the main dam by a high ridge it is called a **Saddle Weir**.




A chute spillway essentially consists of a steeply sloping open channel placed along a dam abutment or through a flank or saddle. It leads the water from the reservoir to the downstream channel below. The base of the channel is usually made of reinforced concrete slabs 25 to 50 cm thick. Light reinforcement of about 0.25% of concrete area is provided in the top of the slab.in both directions. The chute is some times of constant width but is usually narrowed for economy and then widened near the end to reduce the discharging velocity. Expansion joints are usually provided in the chutes at intervals of about 9to 12m in either direction. The expansion joint s should be made water tight so as to avoid any under seepage and its troublesome effects. Under drains are also provided, so as to drain the water which may seep through the trough bottom and side walls. These drains may be in the form of a perforated steel pipes. clay tiles or rock filled trenches.

Slope of chute can conform to available topography leading to minimum excavation, but the slope should be steep enough to maintain supercritical flow to avoid unstable flow conditions.When a vertical curve is provided at a point where chute slope changes it must be gradual & designed to avoid any separation of flow.

## Control Structure or a Low ogee weir

As the trough spillway is provided in a flank or saddle the height of spillway depends upon the natural level of bottom of flank.

If NPL > Natural level of bottom of flank,

Construct low ogee weir height h = NPL- natural level of bottom of flank.

If NPL < Natural level of bottom of flank, then excavate and provide a flat crest at NPL

## Chute slope

Water spilling over the control structure (i.e., Ogee weir) flows through the chute channel Minimum slope of the chute channel should correspond to a supercritical flow for as long a distance as possible. After that slope is made as steep as possible without endangering the stability or without getting into heavy excavations.

**Side walls (called Training Walls)**

Height not to allow any spilling over it. Height = Free Nappe + Free Board

Free Board = $0.61 + 0.4\ V_m\ (D_m)^{1/3}$

Where $V_m$ =mean velocity in the chute

$D_m$ =mean depth of water in the chute

Walls in the vicinity of ogee weir should be made vertical in the later portion it can be vertical or sloping.

## Design of small ogee weir required as control structure for chute spillway

Equation for D/S profile with crest of ogee taken as origin is given as

$$X^N = a\,(H_e)^{N-1}Y$$

**Table showing equations for D/S profile of low ogee weir**

| Value of $H_a/H_e$ | Range $h/H_e$ | a | N |
|---|---|---|---|
| **0.00** | **>1.0** | **1.852** | **1.780** |
| **0.08** | **1.00 – 0.58** | **1.869** | **1.750** |
| **0.12** | **0.58 –0.30** | **1.905** | **1.747** |

Coordinates of U/S profile should merge in a slope of 1:1

| $X/H_e$ | $Y/H_e$ | $Y/H_e$ | $Y/H_e$ |
|---|---|---|---|
| $X/H_e$ | $H_a/H_e$ =0.00 | $H_a/H_e$ =0.08 | $H_a/H_e$ =0.12 |
| -0.020 | 0.0004 | 0.0004 | 0.0004 |
| -0.060 | 0.0036 | 0.0035 | 0.0035 |
| -0.10 | 0.013 | 0.0101 | 0.0099 |
| -0.012 | 0.015 | 0.015 | 0.0147 |
| -0.140 | 0.0207 | 0.028 | 0.0199 |
| -0.150 | 0.0239 | 0.0235 | 0.0231 |
| -0.160 | 0.0275 | 0.0270 | 0.0265 |
| -0.175 | 0.0333 | 0.0328 | 0.0325 |
| -0.190 | 0.0399 | 0.0395 | 0.0390 |
| -0.195 | 0.0424 | 0.042 | ------- |
| -0.200 | 0.0450 | ------ | ----- |

Radius of curve at toe = $2H_e$

Design of Vertical curve of Chute

Avoid sharp convex and concave vertical curves, Provide flat curves where ever required.

Concave Curve   Provided when the chute floor changes from Steeper slope to less steep.

Concave curves should be of large radius to minimize the dynamic force on the floor.

Force created due to centrifugal action $R > (2\gamma\ d\ V^2/pg)$

where V is the velocity , d is the depth of flow; p is the permissible intensity of dynamic pressure exerted on the floor. R> 10d except at the toe of crest where R could be 5d. Curve is made tangential to the u/s and d/s slope.

Convex Curve : Provided when the chute floor changes from Steep slope to steeper. Convex Curve starts tangentially from the end of u/s sloping floor. It should be flat enough to maintain positive pressure on the floor and thus avoid tendency of separation of flow from floor. The convex curve is usually parabolic as given by equation

$Y = \{x \tan\theta\} + \{x^2 / 4K(d + h_V) \cos^2 \theta\}$

$\theta$ is the angle of u/s floor just at the beginning of the curve

K is a factor of safety > 1.5

**Horizontal curves**

Horizontal curves may also be required if the alignment is not straight but takes a curve as it may not be possible to have a straight trough. Curves should be quite gentle and in order to account for super elevation in the curved portion of the trough bed should be provided with a cross slope.

**Approach channel of chute spillway**

An entrance channel called approach channel trapezoidal shaped with side slope 1:1 to lead the reservoir water up to control structure (low ogee weir). Friction head loss in discharge channel = $n^2V^2L/(R)^{4/3}$

One can calculate velocity and depth at different sections by applying that specific energy above a certain datum remains constant and only losses are friction, turbulence, transition and impact.

Entire chute spillway:

(i) Entrance channel

(ii) Control structure

(iii) Chute channel or discharge carrier

(iv) Energy dissipation arrangement at the bottom in the form of stilling basin.

Cutoff

Cutoff at upper end of spillway to reduce uplift pressure on paving. Cutoff at D/S end of paving to prevent under cutting of paving. Further at U/S end of each panel a cutoff is provided to prevent creeping of panels resulting from expansion & contraction due to changes in temperature as well as to prevent flow of water from one panel to other along the underside of the paving. A typical cutoff of this type is shown.

Drainage

Drainage is necessary to prevent uplift from ground water or the water that finds its way through the paving through the operation of spillway. If paving is on rock foundation, drainage system consists of gravel filled trenches under the paving, with some times an open tile drain imbedded in the gravel. The drains are either relieved at intervals through the paving or collected into one or more trunk drain which carry the entire flow to an outlet at lower end of the trough.